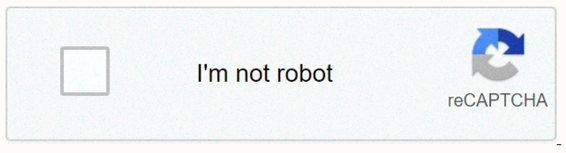

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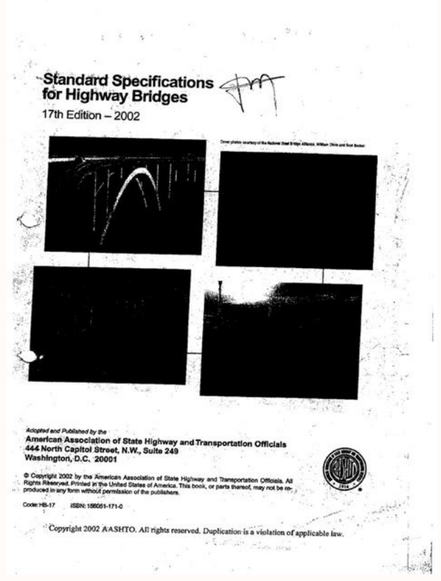
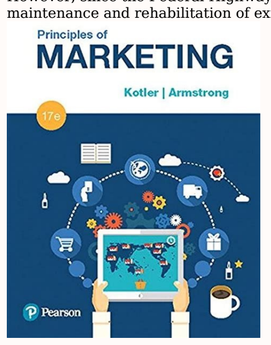
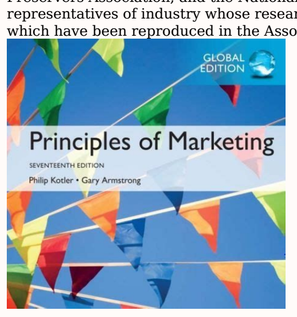
During the period from 1921, until printed in 1931, the specifications were gradually developed, and as the several divisions were approved from time to time, they were made available in mimeographed form for use of the State Highway Departments and other organizations. A complete specification was available in 1926 and it was revised in 1928. Though not in printed form, the specifications were valuable to the bridge engineering profession during the period of development. The first edition of the Standard Specifications was published in 1931, and it was followed by the 1935,1941, 1944,1949,1953,1957, 1961,1965,1969,1973,1977, 1983, 1989, 1992, and 1996 revised editions.



The present seventeenth edition constitutes a revision of the 1996 specifications, including those changes adopted since the publication of the sixteenth edition and those through 2002. In the past, Interim Specifications were usually published in the middle of the calendar year, and a revised edition of this book was generally published every 4 years. However, since the Federal Highway Administration and the States have established a goal that the LRFD standards be used on all new bridge designs after 2007, only edits related to technical errors in the seventeenth edition will be made hereafter. These Standard Specifications are applicable to new structure designs prior to 2007 and for the maintenance and rehabilitation of existing structures.



Future revisions will have the same status as standards of the American Association of State Highway and Transportation Officials (AASHTO) and are approved by at least two-thirds of the Subcommittee on Bridges and Structures. These revisions are voted on by the Association Member Departments prior to the publication of a new edition of this book, and if approved by at least two-thirds of the members, they are included in a new edition as standards of the Association. Members of the Association are the 50 State Highway or Transportation Departments, the District of Columbia, and Puerto Rico. Each member has one vote. The U.S. Department of Transportation is a nonvoting member. Future revisions will be displayed on AASHTO's website via a link from the title's book code listing, HB-17, in the Bookstore of www.transportation.org. An e-mail notification will also be sent to previous purchasers notifying them that a revision is available for download. Please check the site periodically to ensure that you have the most up-to-date and accurate information. The Standard Spec$cations for Highway Bridges are intended to serve as a standard or guide for the preparation of State specifications and for reference by bridge engineers. Primarily, the specifications set forth minimum requirements which are consistent with current practice, and certain modifications may be necessary to suit local conditions. explorateur de fichier android pour mac They apply to ordinary highway bridges and supplemental specifications may be required for unusual types and for bridges with spans longer than 500 feet. Specifications of the American Society for Testing and Materials (ASTM), the American Welding Society, the American Wood Preservers Association, and the National Forest Products Association are referred to, or are recognized. Numerous research bulletins are noted for references. The American Association of State Highway and Transportation Officials wishes to express its sincere appreciation to the above organizations, as well as to those universities and representatives of industry whose research efforts and consultations have been most helpful in continual improvement of these specifications. Extensive references have been made to the Standard for Transportation Materials and Methods of Sampling and Testing also published by AASHTO, including equivalent ASTM specifications which have been reproduced in the Association's Standard Specifications by permission of the American Society for Testing and Materials.



(c) The D/t ratio of we& shall not exceed D & (2 - 3) (d) If subject to combined axial force and bending, the D/t ratio of webs shall not exceed but need not be less than 2,400b I- & or Section 3 LOADS Part A TYPES OF LOADS 3.1 NOTATIONS A a B = maximum expected acceleration of bedrock at the site = length of shoa span of slab (Article 3.24.6) = buoyancy (Article 3.22) b = width of pier or diameter of pile (Article 3.18.2.2.4) b = length of long span of slab (Article 3.24.6) C = combined response coefficient C = stiffness parameter = K(W/L) (Article 3.23.4.3) C = centrifugal force in percent of live load (Article 3.10.1) CF = centrifugal force (Article 3.22) C, = coefficient for nose inclination (Article 3.18.2.2.1) CM = steel bending stress coefficient (Article 3.25.1.5) C, = steel shear stress coefficient (Article 3.25.1.5) D = parameter used in determination of load fraction of wheel load (Article 3.23.4.3) D = degree of curve (Article 3.10.1) D = dead load (Article 3.22) D.F. = fraction of wheel load applied to beam (Article 3.28.1) DL = contributing dead load E = width of slab over which a wheel load is distributed (Article 3.24.3) E = earth pressure (Article 3.22) EQ = equivalent static horizontal force applied at the center of gravity of the structure E, = modulus of elasticity of concrete (Article 3.26.3) E, = modulus of elasticity of steel (Article 3.26.3) E, = modulus of elasticity of wood (Article 3.26.3) F = horizontal ice force on pier (Article 3.18.2.2.1) F, = allowable bending stress (Article 3.25.1.3) F, = allowable shear stress (Article 3.25.1.3) g = 32.2 ft.lse~.~ I = impact fraction (Article 3.8.2) I = gross flexural moment of inertia of the precast member (Article 3.23.4.3) ICE = ice pressure (Article 3.22) J = gross Saint-Venant torsional constant of the precast member (Article 3.23.4.3) K = stream flow force constant (Article 3.18.1) K = stiffness constant (Article 3.23.4) K = wheel load distribution constant for timber flooring (Article 3.25.1.3) k = live load distribution constant for spread box girders (Article 3.28.1) L = loaded length of span (Article 3.8.2) L = loaded length of sidewalk (Article 3.14.1.1) 17 -- 18 L HIGHWAY BRIDGES = live load (Article 3.22) L = span length (Article 3.23.4) LF = longitudinal force from live load (Article 3.22) MD = moment capacity of dowel (Article 3.25.1.4) M, = primary bending moment (Article 3.25.1.3) M, = total transferred secondary moment (Article 3.25.1.4) NB = number of beams (Article 3.28.1) NL = number of traffic lanes (Article 3.23.4) n = number of dowels (Article 3.25.1.4) P = live load on sidewalk (Article 3.14.1.1) P = stream flow pressure (Article 3.18.1) P = total uniform force required to cause unit horizontal deflection of whole structure P = load on one rear wheel of truck (Article 3.24.3) P = wheel load (Article 3.24.5) P = design wheel load (Article 3.25.1.3) PI5 = 12,000 pounds (Article 3.24.3) P2, = 16,000 pounds (Article 3.24.3) p = effective ice strength (Article 3.18.2.2.1) p = proportion of load carried by short span (Article 3.24.6.1) R = radius of curve (Article 3.10.1) R = normalized rock response R = rib shortening (Article 3.22) RD = shear capacity of dowel (Article 3.25.1.4) R, = primary shear (Article 3.25.1.3) R, = total secondary shear transferred (Article 3.25.1.4) S = design speed (Article 3.10.1) S = soil amplification spectral ratio S = shrinkage (Article 3.22) S = average stringer spacing (Article 3.23.2.3.1) S = spacing of beams (Article 3.23.3) S = width of precast member (Article 3.23.4.3) S = effective span length (Article 3.24.1) S = span length (Article 3.24.8.2) S = beam spacing (Article 3.28.1) s = effective deck span (Article 3.25.1.3) SF = stream flow (Article 3.22) T = period of vibration T = temperature (Article 3.22) t = thickness of ice (Article 3.18.2.2.4) t = deck thickness (Article 3.25.1.3) V = variable spacing of truck axles (Figure 3.7.7A) V = velocity of water (Article 3.18.1) W = combined weight on the first two axles of a standard HS Truck (Figure 3.7.7A) W = width of sidewalk (Article 3.14.1.1) W = wind load on structure (Article 3.22) W = total dead weight of the structure We = width of exterior girder (Article 3.23.2.3.2) W = overall width of bridge (Article 3.23.4.3) W = roadway width between curbs (Article 3.28.1) WL = wind load on live load (Article 3.22) w = width of pier or diameter of circular-shaft pier at the level of ice action (Article 3.18.2.2.1) X = distance from load to point of support (Article 3.24.5.1) x = subscript denoting direction perpendicular to longitudinal stringers (Article 3.25.1.3) 3.1 - 3.1 DIVISION I-DESIGN 19 Z = reduction for ductility and risk assessment = (with appropriate script) coefficient applied to actual loads for service load and load factor designs (Article 3.22) = load factor (Article 3.22) y up, = proportional limit stress perpendicular to grain (Article 3.25.1.4) = load combination coefficient for buoyancy (Article 3.22.1) p PB PC = load combination coefficient for centrifugal force (Article 3.22.1) pD = load combination coefficient for dead load (Article 3.22.1) PE = load combination coefficient for earth pressure (Article 3.22.1) PEQ = load combination coefficient for earthquake (Article 3.22.1) PICE= load combination coefficient for ice (Article 3.22.1) pL = load combination coefficient for live load (Article 3.22.1) = load combination coefficient for rib shortening, shrinkage, and temperature (Article 3.22.1) = load combination coefficient for stream flow (Article 3.22.1) = load combination coefficient for wind (Article 3.22.1) PWL= load combination coefficient for wind on live load (Article 3.22.1) p = Poisson's ratio (Article 3.23.4.3) PR Ps Pw 3.2 GENERAL 3.2.1 Structures shall be designed to carry the following loads and forces: Dead load. Live load. Impact or dynamic effect of the live load. Wind loads. Other forces, when they exist, as follows: Longitudinal forces; centrifugal force; thermal forces; earth pressure; buoyancy; shrinkage stresses; rib shortening; erection stresses; ice and current pressure; and earthquake stresses. tactical_bodyweight_workout.pdf Provision shall be made for the transfer of forces between the superstructure and substructure to reflect the effect of friction at expansion bearings or shear resistance at elastomeric bearings. 3.2.2 Members shall be proportioned either with reference to service loads and allowable stresses as provided in Service Load Design (Allowable Stress Design) or, alternatively, with reference to load factors and factored strength as provided in Strength Design (Load Factor Design). 3-23 When stress sheets are required*a diagram or notation of the assumed loads shall be shown and the stresses due to the various loads shall be shown separately. 3.2.4 Where required by design conditions, the concrete placing sequence shall be indicated on the plans or in the special provisions. 3.2.5 The loading combinations shall be in accordance with Article 3.22. 3.2.6 When a bridge is skewed, the loads and forces carried by the bridge through the deck system to pin connections and hangers should be resolved into vertical, lateral, and longitudinal force components to be considered in the design. 3.3 DEADLOAD 3.3.1 The dead load shall consist of the weight of the entire structure, including the roadway, sidewalks, car tracks, pipes, conduits, cables, and other public utility services. 3.3.2 The snow and ice load is considered to be offset by an accompanying decrease in live load and impact and shall not be included except under special conditions. 3.3.2.1 If differential settlement is anticipated in a structure, consideration should be given to stresses resulting from this settlement. 3.3.3 If a separate wearing surface is to be placed when the bridge is constructed, or is expected to be placed in the future, adequate allowance shall be made for its weight in the design dead load. Otherwise, provision for a future wearing surface is not required. barska 3- 9x32 plinker- 22 riflescope manual 3.3.4 Special consideration shall be given to the necessity for a separate wearing surface for those regions where the use of chains on tires or studded snow tires can be anticipated. 20 HIGHWAY BRIDGES 3.3.5 - 3.3.5 Where the abrasion of concrete is not expected, the traffic may bear directly on the concrete slab. If considered desirable, Y 4 inch or more may be added to the slab for a wearing surface. 3.3.6 The following weights are to be used in computing the dead load: #/Cu.ft. Steel or cast steel ..... 490 Cast iron ..... 450 Aluminum alloys . . . the_time_machine_book.pdf . . warranty card sample format word .

. . . . . una realidad aparte castaneda pdf . . . . . .
. . . wondershare pdf editor kuyhaa 175 Timber (treated or untreated) ..... 50 Concrete, plain or reinforced ..... 150 Compacted sand, earth, gravel, or ballast ..... house_tree_person_questions.pdf 120 Loose sand, earth, and gravel ..... 100 Macadam or gravel, rolled ..... 140 Cinder filling ..... 60 Pavement, other than wood block ..... mid_year_self_performance_review_examples.pdf 150 Railway rails, guardrails, and fastenings (per linear foot of track) ..... 200 Stone masonry ..... acceptance big book page 449 pdf 170 Asphalt plank, 1 in. thick ..... 9 lb. sq. ft. traffic lanes, spaced across the entire bridge roadway width measured between curbs. 3.6.3 Fractional parts of design lanes shall not be used, but roadway widths from 20 to 24 feet shall have two design lanes each equal to one-half the roadway width. 3.6.4 The traffic lanes shall be placed in such numbers and positions on the roadway, and the loads shall be placed in such positions wifhin their individual lanes, so as to produce the maximum stress in the memher under 3.7 HIGHWAY LOADS 3,7.1 Standard Truck and Lane Loads* 3.7.1.1 The highway live loadings on the roadways of bridges or incidental structures shall consist of standard trucks or lane loads that are equivalent to truck trains. Two systems of loading are provided, the H loadings and the HS loadings-the HS loadings being heavier than the corresponding H loadings.

STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES

SIXTEENTH EDITION

1996

Adopted by the
American Association of State Highway
and Transportation Officials

3.4 LIVE LOAD The live load shall consist of the weight of the applied moving load of vehicles, cars, and pedestrians. 3.5 OVERLOAD PROVISIONS 3.5.1 For all loadings less than H 20, provision shall be made for an i n f r e q u e n t G y load by applying Loading Combination IA (see Article 3.22), with the live load assumed to be H or HS truck and to occupy a single lane without concurrent loading in any other lane. The overload shall apply to all parts of the structure affected, except the roadway deck, or roadway deck plates and stiffening ribs in the case of orthotropic bridge superstructures. 3.7.1.2 Each lane load shall consist of a uniform load per linear foot of traffic lane combined with a single concentrated load (or two concentrated loads in the case of continuous spans-see Article 3.11.3), so placed on the span as to produce maximum stress. The concentrated load and uniform load shall be considered as uniformly distributed over a 10-foot width on a line normal to the center line of the lane. financial accounting solved mcqs pdf
3.7.1.3 For the computation of moments and shears, different concentrated loads shall be used as indicated in Figure 3.7.6B. The lighter concentrated loads shall be used when the stresses are primarily bending stresses, and the heavier concentrated loads shall be used when the stresses are primarily shearing stresses. acs analytical chemistry exam study guide pdf 3.5.2 Structures may be analyzed for an overload that is selected by the operating agency in accordance with Loading Combination Group IB in Article 3.22. 3.6 TRAFFIC LANES 3.6.1 The lane loading or standard truck shall be assumed to occupy a width of 10 feet. 3.6.2 These loads shall be placed in 12-foot wide design *Note: The system of lane loads defined here (and illustrated in Figure 3.7.6.B) was developed in order to give a simpler method of calculating moments and shears than that based on wheel loads of the truck.
Appendix B shows the truck train loadings of the 1935 Specifications of AASHO and the corresponding lane loadings. In 1944. the HS series of trucks was develoued. These auuroximatethe effect of h e corresponding 1935 truck precehed and followed by a train of trucks weighing three-fourths as much as the basic truck. 3.7.2 DIVISION I-DESIGN 3.7.2 Classes of Loading There are four standard classes of highway loading: H 20, H 15, HS 20, and HS 15. Loading H 15 is 75% of Loading H 20. geekvape aegis x user manual Loading HS 15 is 75% of Loading HS 20. If loadings other than those designated are desired, they shall be obtained by proportionately changing the weights shown for both the standard truck and the corresponding lane loads. 3.7.3 Designation of Loadings The policy of affixing the year to loadings to identify them was instituted with the publication of the 1944 Edition in the following manner: H 15 Loading, 1944 Edition shall be designated..... H 20 Loading, 1944 Edition shall be designated..... ancient words piano chords pdf H 15-S 12 Loading, 1944 Edition shall be designated ..... HS 15-44 H 20-S 16 Loading, 1944 Edition shall be designated ..... HS 20-44 The affix shall remain unchanged until such time as the loading specification is revised.
The same policy for identification shall be applied, for future reference, to loadings previously adopted by AASHTO. 21 gross weight in tons of the tractor truck. The variable axle spacing has been introduced in order that the spacing of axles may approximate more closely the tractor trailers now in use. The variable spacing also provides a more satisfactory loading for continuous spans, in that heavy axle loads may be so placed on adjoining spans as to produce maximum negative moments. 3.8 IMPACT 3.8.1 Application Highway Live Loads shall be increased for those structural elements in Group A, below, to allow for dynamic, vibratory and impact effects. Impact allowances shall not be applied to items in Group B. how to read vernier caliper and micrometer screw gauge pdf It is intended that impact be included as part of the loads transferred from superstructure to substructure, but shall not be included in loads transferred to footings nor to those parts of piles or columns that are below ground. 3.8.1.1 Group A-Impact shall be included. (1) Superstructure, including legs of rigid frames. (2) Piers, (with or without bearings regardless of type) excluding footings and those portions below the ground line. (3) The portions above the ground line of concrete or steel piles that support the superstructure. 3.7.4 Minimum Loading 3.8.1.2 Group B-Impact shall not be included. Bridges supporting Interstate highways or other highways which carry, or which may cany, heavy truck traffic, shall be designed for HS 20-44 Loading or an Alternate Military Loading of two axles four feet apart with each axle weighing 24,000 pounds, whichever produces the greatest stress. The H loadings consist of a two-axle truck or the corresponding lane loading as illustrated in Figures 3.7.6A and 3.7.6B. The H loadings are designated H followed by a number indicating the gross weight in tons of the standard truck. (1) Abutments, retaining walls, piles except as specified in Article 3.8.1.1 (3).
(2) Foundation pressures and footings. (3) Timber structures.
(4) Sidewalk loads. (5) Culverts and structures having 3 feet or more cover.
3.8.2 Impact Formula 3.8.2.1 The amount of the impact allowance or increment is expressed as a fraction of the live load stress, and shall be determined by the formula: 3.7.6 HS Loading The HS loadings consist of a tractor truck with semitrailer or the corresponding lane load as illustrated in Figures 3.7.7A and 3.7.6B. The HS loadings are designated by the letters HS followed by a number indicating the in which, I = impact fraction (maximum 30 percent); HIGHWAY BRIDGES 22 3.8.2.1 I I H 20-44 H 15-44 8,000 LBS. 6,000 LBS.
32,000 L B S .
~ 24,000 LBS.
14'-0 31 2; - W = TOTAL WEIGHT O F - j 3 TRUCK AND LOAD 12 CLEARANCE AND LOAD LANE WIDTH FIGURE 3.7.6A Standard H Trucks *Inthe design of timber floors and orthotropicsteel decks (excluding transverse beams) for H 20 Loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown. **For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Article 3.24.2.) DMSION I-DESIGN 3.8.2.1 18,000 LBS. FOR MOMENT* 26,000 LBS. FOR SHEAR 'ONCENTRATED I cUNIFORM 23 LOAD 640 LBS. 12093117629.pdf PER LINEAR FOOT OF LOAD LANE H20-44 LOADING HS20-44 LOADING L 1 13,500 LBS. FOR MOMENP 19,500 LBS. FOR SHEAR UNIFORM LOAD 480 LBS. PER LINEAR FOOT OF LOAD LANE CONCENT~A~E~ H15-44 LOADING HS15-44 LOADING FIGURE 3.7.6B Lane Loading *For the loading of continuous spans involving lane loading refer to Article 3.11.3 which provides for an additional concentrated load. L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member. 3.8.2.3 For culverts with cover 0'0" to 1'-0" inc. I = 30% 1'-1" to 2'-0" inc.
I = 20% 2'-1" to 2'-11" inc. I = 10% 3.8.2.2 For uniformity of application,in this formula, the loaded length, L, shall be as follows: (a) For roadway floors: the design span length. (b) For transverse members, such as floor beams: the span length of member center to center of supports. (c) For computing truck load moments: the span length, or for cantilever arms the length from the moment center to the farthermost axle. (d) For shear due to truck loads: the length of the loaded portion of span from the point under consideration to the far reaction; except, for cantilever arms, use a 30% impact factor. (e) For continuous spans: the length of span under consideration for positive moment, and the average of two adjacent loaded spans for negative moment. 3.9 LONGITUDINAL FORCES Provision shall be made for the effect of a longitudinal force of 5% of the live load in all lanes carrying traffic headed in the same direction. All lanes shall be loaded for bridges likely to become one directional in the future. The load used, without impact, shall be the lane load plus the concentrated load for moment specified in Article 3.7, with reduction for multiple-loaded lanes as specified in Article 3.12. The center of gravity of the longitudinal force shall be assumed to be located 6 feet above the floor slab and to be transmitted to the substructure through the superstructure.
3.9 HIGHWAY BRIDGES 24 HS20-44 HS15-44 8,000 LBS. 6.000 LBS. 32,000 LBS.* 24,000 LBS. 32,000 LBS* 24,000 LBS. W = COMBINED WEIGHT ON THE FIRST TWO AXLES WHICH IS THE SAME AS FOR THE CORRESPONDING H TRUCK. V = VARIABLE SPACING - 14 FEET TO 30 FEET INCLUSIVE. SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM STRESSES. CLEARANCE AND LOAD LANE WIDTH 10'-0" FIGURE 3.7.7A Standard HS Trucks *In the design of timber floors and orthotropic steel decks (excluding transverse beams) for H 20 Loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown. **For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Article 3.24.2.) 3.10 DIVISION I-DESIGN 25 3.10 CENTRIFUGAL FORCES 3.11.3 Lane Loads on Continuous Spans 3.10.1 Structures on curves shall be designed for a horizontal radial force equal to the following percentage of the live load, without impact, in all traffic lanes: For the determination of maximum negative moment in the design of continuous spans, the lane load shown in Figure 3.7.6B shall be modified by the addition of a second, equal weight concentrated load placed in one other span in the series in such position to produce the maximum effect. For maximum positive moment, only one concentrated load shall be used per lane, combined with as many spans loaded uniformly as are required to produce maximum moment. where, C = the centrifugal force in percent of the live load, without impact; S = the design speed in miles per hour; D = the degree of curve; R = the radius of the curve in feet. 3.10.2 The effects of superelevation shall be taken into account. 3.10.3 The centrifugal force shall be applied 6 feet above the roadway surface, measured along the center line of the roadway. The design speed shall be determined with regard to the amount of superelevation provided in the roadway. The traffic lanes shall be loaded in accordance with the provisions of Article 3.7 with one standard truck on each design traffic lane placed in position for maximum loading. 3.10.4 Lane loads shall not be used in the computation of centrifugal forces. 3.10.5 When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the centrifugal forces acting on the live load. 3.11 APPLICATION OF LIVE LOAD In computing stresses, each 10-foot lane load or single standard truck shall be considered as a unit, and fractions of load lane widths or trucks shall not be used. 3.11.2 Number and Position of Traffic Lane Units The number and position of the lane load or truck loads shall be as specified in Article 3.7 and, whether lane or truck loads, shall be such as to produce maximum stress, subject to the reduction specified in Article 3.12.
+ 3.11.4 Loading for Maximum Stress 3.11.4.1 On both simple and continuous spans, the type of loading, whether lane load or truck load, to be used shall be the loading which produces the maximum stress. The moment and shear tables given in Appendix A show which types of loading controls for simple spans. 3.11.4.2 For continuous spans, the lane loading shall be continuous or discontinuous; only one standard H or HS truck per lane shall be considered on the structure. 3.12 REDUCTION IN LOAD INTENSITY 3.12.1 Where maximum stresses are produced in any member by loading a number of traffic lanes simultaneously, the following percentages of the live loads may be used in view of the improbability of coincident maximum loading: Percent One or two lanes .....lo0 Three lanes . . . . . .
. .
. . . . . .90 Four lanes or more .....75 3.12.2 The reduction in load intensity specified in Article 3.12.1 shall not be applicable when distribution factors from Table 3.23.1 are used to determine moments in longitudinal beams. 3.12.3 The reduction in intensity of loads on transverse members such as floor beams shall be determined as in the case of main trusses or girders, using the number of traffic lanes across the width of roadway that must be loaded to produce maximum stresses in the floor beam. 26 HIGHWAY BRIDGES 3.13 -- 3.13 ELECTRIC RAILWAY LOADS 3.14.2.2 Where sidewalk, curb, and traffic rail form an integral system, the traffic railing loading shall be applied and stresses in curbs computed accordingly. If highway bridges carry electric railway traffic, the railway loads shall be determined from the class of traffic which the bridge may be expected to carry. The possibility that the bridge may be required to carry railroad freight cars shall be given consideration. 3.14.3 Railing Loading 3.14 SIDEWALK, CURB, AND RAILING LOADING 3.15 WIND LOADS 3.14.1 Sidewalk Loading 3.14.1.1 Sidewalk floors, stringers, and their irnmediate supports shall be designed for a live load of 85 pounds per square foot of sidewalk area. Girders, trusses, arches, and other members shall be designed for the following sidewalk live loads: Spans 0 to 25 feet in length . . . . . .85 l b . / f ~ ~ Spans 26 to 100 feet in length .
. . . . . .60 lb./ft.2 Spans over 100 feet in length according to the formula For Railing Loads, see Article 2.7.1.3. The wind load shall consist of moving uniformly distributed loads applied to the exposed area of the structure.
The exposed area shall be the sum of the areas of all members, including floor system and railing, as seen in elevation at 90 degrees to the longitudinal axis of the structure. The forces and loads given herein are for a base wind velocity of 100 miles per hour. For Group 11 and Group V loadings, but not for Group 111 and Group VI loadings, they may be reduced or increased in the ratio of the square of the design wind velocity to the square of the base wind velocity provided that the maximum probable wind velocity can be ascertained with reasonable accuracy, or provided that there are permanent features of the terrain which make such changes safe and advisable. If a change in the design wind velocity is made, the design wind velocity shall be shown on the plans. in which P = live load per square foot, max. 60-lb.
per sq. ft. L = loaded length of sidewalk in feet. W = width of sidewalk in feet. 3.14.1.2 In calculating stresses in structures that Support cantilevered sidewalks, the sidewalk shall be fully loaded on only one side of the structure if this condition produces maximum stress. 3.14.1.3 Bridges for pedestrian andlor bicycle tr shall be designed for a live load of 85 PSE c 3.149194 Where bicycle or pedestrian bridges are expected to be used by maintenance vehicles, special design consideration should be made for these loads. 3.14.2 Curb Loading 3.14.2.1 Curbs shall be designed to resist a lateral force of not less than 500 pounds per linear foot of curb, applied at the top of the curb, or at an elevation 10 inches above the floor if the curb is higher than 10 inches. 3.15.1 Superstructure Design 3.15.1.1 Group I1 and Group V Loadings 3.15.1.1.1 A wind load of the following intensity shall be applied horizontally at right angles to the longitudinal axis of the structure: For trusses and arches .....75 pounds per square foot For girders and beams .....50 pounds per square foot 3.15.1.1.2 The total force shall not be less than 300 pounds per linear foot in the plane of the windward chord and 150 pounds per linear foot in the plane of the leeward chord on truss spans, and not less than 300 pounds per linear foot on girder spans. 3.15.1.2 Group 111 and Group VI Loadings Group I11 and Group VI loadings shall comprise the loads used for Group I1 and Group V loadings reduced by 70% and a load of 100 pounds per linear foot applied at right angles to the longitudinal axis of the structure and 6 feet above the deck as a wind load on a moving live load. - 3.15.1.2 DIVISION I-DESIGN When a reinforced concrete floor slab or a steel grid deck is keyed to or attached to its supporting members, it may be assumed that the deck resists, within its plane, the shear resulting from the wind load on the moving live load.
3.15.2 Substructure Design Forces transmitted to the substructure by the superstructure and forces applied directly to the subsmcture by wind loads shall be as follows: 27 This load shall be applied at a point 6 feet above the deck. 3.15.2.1.3 For the usual girder and slab bridges having maximum span lengths of 125 feet, the following wind loading may be used in lieu of the more precise loading specified above: W (wind load on structure) 50 pounds per square foot, transverse 12 pounds per square foot, longitudinal Both forces shall be applied simultaneously. 3.15.2.1 Forces from Superstructure 3.15.2.1.1 The transverse and longitudinal forces transmitted by the superstructure to the substructure for various angles of wind direction shall be as set forth in the following table. The skew angle is measured from the perpendicular to the longitudinal axis and the assumed wind direction shall be that which produces the maximum stress in the substructure. The transverse and longitudinal forces shall be applied simultaneously at the elevation of the center of gravity of the exposed area of the superstructure. Skew Angle of Wind Degrees Trusses Lateral Longitudinal Load Load PSF PSF 0 15 30 45 60 75 70 65 47 24 0 12 28 41 50 Girders Lateral Longitudinal Load Load PSF PSF 50 44 41 33 17 0 6 12 16 19 The loads listed above shall be used in Group I1 and Group V loadings as given in Article 3.22. 3.15.2.1.2 For Group III and Group VI loadings, these loads may be reduced by 70% and a load per linear foot added as a wind load on a moving live load, as given in the following table: Skew Angle of Wmd Degrees 0 15 30 45 60 Lateral Load Ib.lft. Longitudinal Load lb./ft. 100 88 82 66 34 0 12 24 32 38 WL (wind load on live load) 100 pounds per linear foot, transverse 40 pounds per linear foot, longitudinal Both forces shall be applied simultaneously. 3.15.2.2 Forces Applied Directly to the Substructure The transverse and longitudinal forces to be applied directly to the substructure for a 100-mile per hour wind shall be calculated from an assumed wind force of 40 pounds per square foot. For wind directions assumed skewed to the substructure, this force shall be resolved into components perpendicular to the end and front elevations of the substructure. The component perpendicular to the end elevation shall act on the exposed substructure area as seen in end elevation and the component perpendicular to the front elevation shall act on the exposed areas and shall be applied simultaneously with the wind loads from the superstructure.
The above loads are for Group11 and Group V loadings and may be reduced by 70% for Group 111 and Group VI loadings, as indicated in Article 3.22. 3.15.3 Overturning Forces The effect of forces tending to overturn structures shall be calculated under Groups 11, III, V, and VI of Article 3.22 assuming that the wind direction is at right angles to the longitudinal axis of the structure. In addition, an upward force shall be applied at the windward quarter point of the transverse superstructure width.
This force shall be 20 pounds per square foot of deck and sidewalk plan area for Group 11 and Group V combinations and 6 pounds per square foot for Group 111 and Group VI combinations. 28 HIGHWAY BRIDGES 3.16 THERMAL FORCES ity distribution and thus a triangular pressure distribution, shall be calculated by the formula: Provision shall be made for stresses or movements resulting from variations in temperature. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be computed from an assumed temperature at the time of erection. Due consideration shall be given to the lag between air temperature and the interior temperature of massive concrete members or structures. The range of temperature shall generally be as follows: Metal structures: Moderate climate, from 0 to 120°E Cold climate, from - 30 to 120% Temperature Rise Temperature Fall 30°F 35°F 40°F 45°F Concrete structures: Moderate climate . . . . . .
. Cold climate .
. . . . .
. .
. . .
. . 3.16 3.17 UPLIFT 3.17.1 Provision shall be made for adequate attachment of the superstructure to the substructure by ensuring that the calculated uplift at any support is resisted by tension members engaging a mass of masonry equal to the largest force obtained under one of the following conditions: (a) 100% of the calculated uplift caused by any loading or combination of loadings in which the live plus impact loading is increased by 100%. (b) 150% of the calculated uplift at working load level. 3.17.2 Anchor bolts subject to tension or other elements of the structure stressed under the above conditions shall be designed at 150% of the allowable basic stress. 3.18 FORCES FROM STREAM CURRENT AND FLOATING ICE, AND DRIFT CONDITIONS where, Pa, = average stream pressure, in pounds per square foot, Va, = average velocity of water in feet per second, computed by dividing the flow rate by the flow area, K = a constant, being 1.4 for all piers subjected to drift build-up and square-ended piers, 0.7 for circular piers, and 0.5 for angle-ended piers where the angle is 30 degrees or less. The maximum stream flow pressure, P-, shall be equal to twice the average stream flow pressure, Pa,,, computed by Equation 3-4.
Stream flow pressure shall be a triangular distribution with P, located at the top of water elevation and a zero pressure located at the flow line. 3.18.1.1.2 The stream flow forces shall be computed by the product of the stream flow pressure, taking into account the pressure distribution, and the exposed pier area. In cases where the corresponding top of water elevation is above the low beam elevation, stream flow loading on the superstructure shall be investigated.The stream flow pressure acting on the superstructuremay be taken as P, with a uniform distribution. 3.18.1.2 Pressure Components When the direction of stream flow is other than normal to the exposed surface area, or when bank migration or a change of stream bed meander is anticipated, the effects of the directional components of stream flow pressure shall be investigated. 3.18.1.3 Drift Lodged Against Pier All piers and other portions of structures that are subject to the force of flowing water, floating ice, or drift shall be designed to resist the maximum stresses induced thereby. 3.18.1 Force of Stream Current on Piers 3.18.1.1 Stream Pressure 3.18.1.1.1 The effect of flowing water on piers and drift build-up, assuming a second-degree parabolic veloc- Where a significant amount of drift lodged against a pier is anticipated, the effects of this drift buildup shall be considered in the design of the bridge opening and the bridge components. The overall dimensions of the drift buildup shall reflect the selected pier locations, site conditions, and known drift supply upstream. When it is anticipated that the flow area will be significantly blocked by drift buildup, increases in high water elevations, stream velocities, stream flow pressures, and the potential increases in scour depths shall be investigated. 3.18.2 29 DIVISION I-DESIGN 3.18.2 Force of Ice on Piers 3.18.2.2.3 The following values of effective ice strength appropriate to various situations may be used as a guide. 3.18.2.1 General Ice forces on piers shall be selected, having regard to site conditions and the mode of ice action to be expected. Consideration shall be given to the following modes: (a) Dynamic ice pressure due to moving ice-sheets and ice-floes carried by streamflow, wind, or currents. (b) Static ice pressure due to thermal movements of continuous stationary ice-sheets on large bodies of water. (c) Static pressure resulting from ice-jams.
(d) Static uplift or vertical loads resulting from adhering ice in waters of fluctuating level. 3.18.2.2 Dynamic Ice Force 3.18.2.2.1 Horizontal forces resulting from the pressure of moving ice shall be calculated by the formula: F=C,p.t.w (3-5) (a) In the order of 100 psi where breakup occurs at melting temperatures and where the ice runs as small "cakes" and is substantially disintegrated in its structure. (b) In the order of 200 psi where breakup occurs at melting temperatures,but the ice moves in large pieces and is internally sound.
(c) In the order of 300 psi where at breakup there is an initial movement of the ice sheet as a whole or where large sheets of sound ice may strike the piers. (d) In the order of 400 psi where breakup or major ice movement may occur with ice temperatures significantly below the melting point. 3.18.2.2.4 The preceding values for effective ice strength are intended for use with piers of substantial mass and dimensions. The values shall be modified as necessary for variations in pier width or pile diameter, and design ice thickness by multiplying by the appropriate coefficient obtained from the following table: where, b/t Coefficient F C, p t = horizontal ice force on pier in pounds; = coefficient for nose inclination from table; = effective ice strength in pounds per square inch; = thickness of ice in contact with pier in inches; 0.5 w = 1.8 1.3 1.1 1 .O 0.9 0.8 width of pier or diameter of circular-shaft pier at the level of ice action in inches. Inclination of Nose to vertical cn 1.O 1.5 2.0 3.0 4.0 or greater where, b = width of pier or diameter of pile; t = design ice thickness. 3.18.2.2.2 The effective ice strength p shall normally be taken in the range of 100 to 400 pounds per square inch on the assumption that crushing or splitting of the ice takes place on contact with the pier. The value used shall be based on an assessment of the probable condition of the ice at time of movement, on previous local experience, and on assessment of existing structure performance. Relevant ice conditions include the expected temperature of the ice at time of movement, the size of moving sheets and floes, and the velocity at contact. Due consideration shall be given to the probability of extreme rather than average conditions at the site in question. 3.18.2.2.5 Piers should be placed with their longitudinal axis parallel to the principal direction of ice action. The force calculated by the formula shall then be taken to act along the direction of the longitudinal axis. A force transverse to the longitudinal axis and amounting to not less than 15%of the longitudinal force shall be considered to act simultaneously. 3.18.2.2.6 Where the longitudinal axis of a pier cannot be placed parallel to the principal direction of ice action, or where the direction of ice action may shift, the total force on the pier shall be computed by the formula and resolved into vector components. In such conditions, 30 HIGHWAY BRIDGES 3.18.2.2.6 - forces transverse to the longitudinal axis shall in no case be taken as less than 20% of the total force. holes and crushed rock, pipe drains or gravel drains, or by perforated drains. 3.18.2.2.7 In the case of slender and flexible piers, consideration should be given to the vibrating nature of dynamic ice forces and to the possibility of high momentary pressures and structural resonance. 3.21 EARTHQUAKES 3.18.2.3 Static Ice Pressure Ice pressure on piers frozen into ice sheets on large bodies of water shall receive special consideration where there is reason to believe that the ice sheets are subject to significant thermal movements relative to the piers. 3.19 BUOYANCY Buoyancy shall be considered where it affects the design of either substructure, including piling, or the superstructure. 3.20 EARTH PRESSURE 3.20.1 Structures which retain fills shall be proportioned to withstand pressure as given by Coulomb's Equation or by other expressions given in Section 5, "Retaining Walls"; provided, however, that no structure shall be designed for less than an equivalent fluid weight (mass) of 30 pounds per cubic foot. In regions where earthquakes may be anticipated, structures shall be designed to resist earthquake motions by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure in accordance with Division I-A-Seismic Design. Part B COMBINATIONS OF LOADS 3.22 COMBINATIONS OF LOADS 3.22.1 The following Groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned to withstand safely all group combinations of these forces that are applicable to the particular site or type. Group loading combinations for Service Load Design and Load Factor Design are given by: where, 3.20.2 For rigid frames a maximum of one-half of the moment caused by earth pressure (lateral) may be used to reduce the positive moment in the beams, in the top slab, or in the top and bottom slab, as the case may be. 3.20.3 When highway traffic can come within a horizontal distance from the top of the structure equal to onehalf its height, the pressure shall have added to it a live load surcharge pressure equal to not less than 2 feet of earth. 3.20.4 Where an adequately designed reinforced concrete approach slab supported at one end by the bridge is provided, no live load surcharge need be considered. 3.20.5 All designs shall provide for the thorough drainage of the back-filling material by means of weep N y p = group number; = load factor, see Table 3.22.1A; = coefficient, see Table 3.22.1A; = dead load; = live load; = live load impact; D L I E = earth pressure; B = buoyancy; W = wind load on structure; WL = wind load on live load-100 pounds per linear foot; LF = longitudinal force from live load; CF = centrifugal force; R = rib shortening; S = shrinkage; T = temperature; EQ = earthquake; SF = stream flow pressure; ICE = ice pressure.
3.22.1 -- DIVISION I-DESIGN 31 - -- - - TABLE 3.22.1A Table of Coefficients y and P Culvert -- Culvert (L (L + I), - Live load plus impact for AASHTO Highway H or HS loading + I), - Live load plus impact consistent with the overload criteria of the operation agency. * 1.25 may be used for design of outside roadway beam when combination of sidewalk live load as well as traffic live load plus impact governs the design, but the capacity of the section should not be less than required for highway traffic live load only using a beta factor of 1.67. 1.00 may be used for design of deck slab with combination of loads as described in Article 3.24.2.2. Unit Stress (Operating Rating) ** Percentage = Maximum Allowable Basic Unit Stress For culvert loading specifications, see Article 6.2. p, For Load Factor Design BE = BE = 0.5 for lateral earth pressure when checking positive moments in rigid frames. This complies with Article 3.20. = 1.0 for vertical earth pressure = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity . .For Column = 1.0 when checking member for maximum axial . . . . .
.Design load and minimum moment = 1.0 for flexural and tension members = 1.0 for Rigid Culverts = 1.5 for Flexible Culverts pE BD For Service Load Design % (Column 14) Percentage of Basic Unit Stress No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only. pE = 1.oO for vertical and lateral loads on all other structures.
= 1.0 and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs). See Article 3.20. BD pD PE BE 1.3 for lateral earth pressure for retaining walls and rigid frames excluding rigid culverts. For lateral at-rest earth pressures, BE = 1.15 ... . For Group X loading (culverts) the PE factor shall be applied to vertical and horizontal loads. 32 -- HIGHWAY BRIDGES - -- 3.22.2 For service load design, the percentage of the basic unit stress for the various groups is given in Table 3.22.1A. The loads and forces in each group shall be taken as appropriate from Articles 3.3 to 3.21. The maximum section required shall be used. -- - - - - - - 3.22.2 -- - culations of horizontal shear in rectangular timber beams shall be in accordance with Article 13.3. 3.23.2 Bending Moments in Stringers and Longitudinal Beams** 3.23.2.1 General 3.22.3 For load factor design, the gamma and beta factors given in Table 3.22.1A shall be used for designing structural members and found:ations by the load factor concept. 3.22.4 When long span structures are being designed by load factor design, the gamma and beta factors specified for Load Factor Design represent general conditions and should be increased if, in the Engineer's judgment, expected loads, service conditions, or materials of cons~ructionare different from those anticipated by the specifications. 3.22.5 Structures may be analyzed for an overload that is selected by the operating agency. Size and configuration of the overload, loading combinations, and load distribution will be consistent with procedures defined in permit policy of that agency. The load shall be applied in Group IB as defined in Table 3.22.1A. For all loadings less than H 20, Group IA loading combination shall be used (see Article 3.5). Part C DISTRIBUTION OF LOADS 3.23 DISTRIBUTION OF LOADS TO STRINGERS, LONGITUDINAL BEAMS, AND FLOOR BEAMS* 3.23.1 Position of Loads for Shear 3.23.1.1 In calculating end shears and end reactions in transverse floor beams and longitudinal beams and stringers, no longitudinal distribution of the wheel load shall be assumed for the wheel or axle load adjacent to the transverse floor beam or the end of the longitudinal beam or stringer at which the stress is being determined. 3.23.1.2 Lateral distribution of the wheel loads at ends of the beams or stringers shall be that produced by assuming the flooring to act as a simple span between stringers or beams. For wheels or axles in other positions on the span, the distribution for shear shall be determined by the method prescribed for moment, except that the cal*Provisions in this Article shall not apply to orthotropic deck bridges. In calculating bending moments in longitudinal beams or stringers, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution shall be determined as follows.
3.23.2.2 Interior Stringers and Beams The live load bending moment for each interior stringer shall be d ~ t m n h e by d applying to the stringer the fraction of a wheel load (both front and rear) determined in Table 3.23.1. 3.23.2.3 Outside Roadway Stringers and Beams 3.23.2.3.1 Steel-Timber-Concrete T-Beams 3.23.2.3.1.1 The dead load supported by the outside roadway stringer or beam shall be that portion of the floor slab carried by the stringer or beam. Curbs, railings, and wearing surface, if placed after the slab has cured, may be distributed equally to all roadway stringers or beams. 3.23.2.3.1.2 The live load bending moment for outside roadway stringers or beams shall be determined by applying to the stringer or beam the reaction of the wheel load obtained by assuming the flooring to act as a simple span between stringers or beams. 3.23.2.3.1.3 When the outside roadway beam or stringer supports the sidewalk live load as well as traffic live load and impact and the structure is to be designed by the service load method, the allowable stress in the beam or stringer may be increased by 25%for the combination of dead load, sidewalk live load, traffic live load, and impact, providing the beam is of no less carrying capacity than would be required if there were no sidewalks. When the combination of sidewalk live load and traffic live load plus impact governs the design and the structure is to be designed by the load factor method, 1.25 may be used as the beta factor in place of 1.67.
3.23.2.3.1.4 In no case shall an exterior stringer have less carrying capacity than an interior stringer. **In view of the complexity of the theoretical analysis involved in the distribution of wheel loads to stringers, the empirical method herein described is authorized for the design of normal highway bridges. DIVISION I-DESIGN TABLE 3.23.1 Distribution of Wheel Loads in Longitudinal Beams Kind of Floor Timber:" Plankb Nail laminatedc 4" thick or multiple layeF floors ove; 5" thick Nail laminatedc 6" or more thick Glued laminatede Panels on glued laminated stringers 4" thick 6" or more thick On steel stringers 4" thick 6" or more thick Concrete: On steel I-Beam stringersg and prestressed concrete girders On concrete T-Beams On timber stringers Concrete box girdersh On steel box girders On prestressed concrete spread box Beams Steel grid: (Less than 4" thick) (4" or more) Steel bridge cormgat& plank' (2" min. depth) Bridge Designed for One 'Ikaffic Lane Bridge Designed for Two or more 'Itaffic Lanes Sl4.0 S13.75 S15.0 If S exceeds 5' use footnote f. S14.25 If S exceeds 6.5' use footnote f. S14.5 S16.0 If S exceeds 6' use footnote f. S14.0 Sl5.0 If S exceeds 7.5' use footnote f. S14.5 S15.25 If S exceeds 5.5' use footnote f. Sl4.0 Sl4.5 If S exceeds 7' use footnote f. Sfl.0 If S exceeds 10' use footnote f. S15.5 If S exceeds 14' use footnote f.
Sl6.5 If S exceeds 6' use foomote f.
Sl6.0 If S exceeds 10' use footnote f. Sl6.0 If S exceeds 6' use footnote f. Sl5.0 If S exceeds 10' use footnote f. S18.0 If S exceeds 12' use footnote f. See Article 10.39.2. sn.0 If S exceeds 16' use footnote f. See Article 3.28. S14.5 S16.0 If S exceeds 6' use footnote f. Sl4.0 Sl5.0 If S exceeds 10.5' use foomote f. S15.5 S14.5 s = average stringer spacing in feet. aTimberdimensions shown are for nominal thickness. bPlankfloors consist of pieces of lumber laid edge to edge with the wide faces bearing on the supports (see Article 16.3.11-Division II). cNaillaminated floors consist of pieces of lumber laid face to face with the narrow edges bearing on the s u ~ ~ r teach s , piece being - nailed to the preceding pi&e (see Akcle 16.3.ii-~ivisionII). dMultiplelayer floors consist of two or more layers of planks, each layer being laid at an angle to the other (seeArticle 16.3.11-Division 11). eGlued laminated panel floors consist of vertically glued laminated members with the narrow edges of the laminations bearing on the supports (see Article 16.3.13-Division II). 'In this case the load on each stringer shall be the reaction of the wheel loads, assuming the flooring between the stringers to act as a simple beam. g"Design of I-Beam Bridges" by N. M. Newmark-Proceedings, ASCE, March 1948.
hThesidewalk live load (see Article 3.14) shall be omitted for interior and exterior box girders designed in accordance with the wheel load distribution indicated herein. 'Distribution factors for Steel Bridge Corrugated Plank set forth above are based substantially on the following reference: Journal of Washington Academy of Sciences, Vol. 67, No. 2, 1977 "Wheel Load Distribution of Steel Bridge Plank," by Conrad P. Heins, Professor of Civil Engineering, University of Maryland. These distribution factors were developed based on studies using 6" X 2" steel cormgated plank. The factors should yield safe results for other corrugated configurations provided primary bending stiffness is the same as or greater than the 6" X 2" corrugated plank used in the studies. 3.23.2.3.1.5 In the case of a span with concrete floor supported by 4 or more steel stringers, the fraction of the wheel load shall not be less than: where, S = 6 feet or less and is the distance in feet between outside and adjacent interior stringers, and where, S is more than 6 feet and less than 14 feet.
When S is 14 feet or more, use footnote f, Table 3.23.1. 3.23.2.3.2 Concrete Box Girders 3.23.2.3.2.1 The dead load supported by the exterior girder shall be determined in the same manner as for steel, timber, or concrete T-beams, as given in Article 3.23.2.3.1. 3.23.2.3.2.2 The factor for the wheel load distribution to the exterior girder shall be WJ7,where We is the width of exterior girder which shall be taken as the top slab width, measured from the midpoint between girders to the outside edge of the slab. The-cantilever dimension of any slab extending beyond the exterior girder shall preferably not exceed half the girder spacing.
3.23.2.3.3 Total Capacity of Stringers and Beums The combined design load capacity of all the beams stringers in a 'pan shall not be less than required support the total live and dead load in the span. - . - HIGHWAY BRIDGES 34 3.23.3 - - TABLE 3.233.1 Distribution of Wheel Loads in Ransverse Beams 3.23.3 Bending Moments in Floor Beams (Transverse) 3.23.3.1 In calculating bending moments in floor beams, no transverse distribution of the wheel loads shall be assumed. 3.23.3.2 If longitudinal stringers are omitted and the floor is supported directly on floor beams, the beams shall be designed for loads determined in accordance with Table 3.23.3.1. Kind of Floor Nail laminatedC or glued'laminatede, 4 inches in thickness, or multiple laye8 floors more than 5 inches thick 3.23.4 Precast Concrete Beams Used in Multi-Beam Decks Nail laminatedc or glued laminatede, 3.23.4.1 A multi-beam bridge is constructed with precast reinforced or prestressed concrete beams that are placed side by side on the supports. The interaction between the beams is developed by continuous longitudinal shear keys used in combination with transverse tie assemblies which may, or may not, be prestressed, such as bolts, rods, or prestressing strands, or other mechanical means. Full-depth rigid end diaphragms are needed to ensure proper load distribution for channel, single- and multi-stemmed tee beams. Concrete 6 inches or more in thickness Steel grid (less than 4 inches thick) Fraction of Wheel Load to Each Floor Beam S 4.5 sf 5 S 4.5 Steel grid (4 inches or more) Steel bridge corrugated plank (2 inches minimum depth) S 5.5 Note: S = spacing of floor beams in feet.
3.23.4.2 In calculating bending moments in multibeam precast concrete bridges, conventional or prestressed, no longitudinal distribution of wheel load shall be assumed. 3.23.4.3 The live load bending moment for each section shall be determined by applying to the beam the fraction of a wheel load (both

shall be 1 foot from the face of the rail. b = the overall width of the box, d = the overall depth of the box, t = the thickness of either web, tf = the thickness of either flange. The formula assumes that both flanges are the same thickness and uses the thickness of only one flange. The same is true of the webs.
For preliminary design, the following values of K may be used: 3.24.2.2 In designing sidewalks, slabs and supporting members, a wheel load located on the sidewalk shall be 1 foot from the face of the rail. In service load design, the combined dead, live, and impact stresses for this loading shall be not greater than 150% of the allowable stresses. In load factor design, 1.0 may be used as the beta factor in place of 1.67 for the design of deck slabs.
Wheel loads shall not be applied on sidewalks protected by a traffic bamer. 3.24.3 Bending Moment Bridge Multi-beam Beam Type Non-voided rectangular beams Rectangular beams with circular voids Box section beams Channel, single- and multi-stemmed tee beams K 0.7 0.8 1 .O The bending moment per foot width of slab shall be calculated according to methods given under Cases A and 2.2 3.24 DISTRIBUTION OF LOADS AND DESIGN OF CONCRETE SLABSQ 3.24.1 Span Lengths (See Article 8.8) 3.24.1.1 F~~simple spans the span length shall be the distance center to center but need not exceed clear span plus thickness of slab. *The slab distribution set forth herein is based substantially on the "Westergaard" theory. The following references are furnished concerning the subject of slab design. Public Roads, March 1930,"Computation of Stresses in Bridge Slabs Due to Wheel Loads," bv H. M. westereaard. University of ~llinois,~ulletin No. 363, "Solutions,for Certain Rectangular Slabs Continuous over Flexible Supports," by Vernon P. Jensen; ~ulletin304. "A Distribution Procedure forthe Analvsis of Slabs Continuous Over'Flexible Beams," by Nathan M. ~ e w k r k BuUetin ; 315, "Moments in Simple Span Bridge Slabs with Stiffened Edges," by Vernon P. Jensen; and Bulletin 346, "Highway Slab Bridges with Curbs; Laboratory Tests and Proposed Design Method." 36 HIGHWAY BRIDGES B, unless more exact methods are used considering tire contact area. The tire contact area needed for exact methods is given in Article 3.30. In Cases A and B: span length, in feet, as defined under "Span Lengths" Articles 3.24.1 and 8.8; E = width of slab in feet over which a wheel load is distributed; P = load on one rear wheel of truck (PISor PZO); PIS = 12,000 pounds for H 15 loading; Pzo = 16,000 pounds for H 20 loading. S = effective 3.24.3.1 Case A-Main Reinforcement Perpendicular to Traffic (Spans 2 to 24 Feet Inclusive) The live load moment for simple spans shall be determined by the following formulas (impact not included): HS 20 Loading: + ( ) P, = Moment in foot - pounds (3 - 15) per foot - width of slab HS 15 Loading: (I' PI, = Moment in foot - pounds (3 - 16) perfoot-widthofslab In slabs continuous over three or more supports, a continuity factor of 0.8 shall be applied to the above formulas for both positive and negative moment. 3.24.3.2 Case B-Main Reinforcement Parallel to Traffic For wheel loads, the distribution width, E, shall be (4 0.06s) but shall not exceed 7.0 feet. Lane loads are distributed over a width of 2E. Longitudinally reinforced slabs shall be designed for the appropriate HS loading. For simple spans, the maximum live load moment per foot width of slab, without impact, is closely approximated by the following formulas: + HS 20 Loading: Spans up to and including 50 feet: LLM = 900s foot-pounds Spans 50 feet to 100 feet: LLM = 1,000 (1.30s-20.0) foot-pounds 3.24.3 HS 15 Loading: Use 3/4 of the values obtained from the formulas for HS 20 Loading Moments in continuous spans shall be determined by suitable analysis using the truck or appropriate lane loading. 3.24.4 Shear and Bond Slabs designed for bending moment in accordance with Article 3.24.3 shall be considered satisfactory in bond and shear.
3.24.5 Cantilever Slabs 3.24.5.1 Truck Loads Under the following formulas for distribution of loads on cantilever slabs, the slab is designed to support the load independently of the effects of any edge support along the end of the cantilever. The distribution given includes the effect of wheels on parallel elements.
3.24.5.1.1 Case A-Reinforcement Perpendicular to Trafjic Each wheel on the element perpendicular to traffic shall be distributed over a width according to the following formula: The moment per foot of slab shall be (PIE) X footpounds, in which X is the distance in feet from load to point of support. 3.24.5.1.2 Case B-Reinforcement Parallel to Trafjic The distribution width for each wheel load on the element parallel to traffic shall be as follows: E = 0.35X + 3.2, but shall not exceed 7.0 feet (3-18) The moment per foot of slab shall be (PIE) X footpounds. 3.24.5.2 Railing Loads Railing loads shall be applied in accordance with Article 2.7. The effective length of slab resisting post loadings shall be equal to E = 0.8X 3.75 feet where no parapet + 3.24.5.2 -- - DIVISION I-DESIGN - . - - -- + is used and equal to E = 0.8X 5.0 feet where a parapet is used, where X is the distance in feet from the center of the post to the point under investigation. Railing and wheel loads shall not be applied simultaneously. 3.24.6 Slabs Supported on Four Sides 3.24.6.1 For slabs supported along four edges and reinforced in both directions, the proportion of the load carried by the short span of the slab shall be given by the following equations: For uniformly distributed load, p = b4 (3-19) a 4 + b4 b3 (3-20) For concentrated load at center, p = a3 + b3 where, p = proportion of load carried by short span; a = length of short span of slab; b = length of long span of slab. 3.24.6.2 Where the length of the slab exceeds 1% times its width, the entire load shall be carried by the transverse reinforcement. 3.24.6.3 The distribution width, E, for the load taken by either span shall be determined as provided for other slabs. The moments obtained shall be used in designing the center half of the short and long slabs. The reinforcement steel in the outer quarters of both short and long spans may be reduced by 50%. In the design of the supporting beams, consideration shall be given to the fact that the loads delivered to the supporting beams are not uniformly distributed along the beams. 3.24.7 Median Slabs Raised median slabs shall be designed in accordance with the provisions of this article with truck loadings so placed as to produce maximum stresses. Combined dead, live, and impact stresses shall not be greater than 150% of the allowable stresses. Flush median slabs shall be designed without overstress. 3.24.8 Longitudinal Edge Beams 3.24.8.1 Edge beams shall be provided for all slabs having main reinforcement parallel to traffic. The beam may consist of a slab section additionally reinforced, a 37 -- - - - beam integral with and dee~erthanthe slab, or an integral reinforced section of slab and curb. 3.24.8.2 The edge beam of a simple span shall be designed to resist a live load moment of 0.10 PS, where, P = wheel load in pounds PI5or PzO; S = span length in feet.
3.24.8.3 For continuous spans, the moment may be reduced by 20% unless a greater reduction results from a more exact analysis. 3.24.9 Unsupported Transverse Edges The design assumptions of this article do not providefor the effect of loads near unsupported edges.
Therefore, at the ends of the bridge and at intermediate points where the continuity of the slab is broken, the edges shall be supported by diaphragms or other suitable means. The diaphragms shall be designed to resist the full moment and shear produced by the wheel loads which can come on them. 3.24.10 Distribution Reinforcement 3.24.10.1 To provide for the lateral distribution of the concentrated live loads, reinforcement shall be placed transverse to the main steel reinforcement in the bottoms of all slabs except culvert or bridge slabs where the depth of fill over the slab exceeds 2 feet.
3.24.10.2 The amount of distribution reinforcement shall be the percentage of the main reinforcement steel required for positive moment as given by the following formulas: For main reinforcement parallel to traffic, 100 Percentage = -Maximum 50% & (3 - 21) For main reinforcement perpendicular to traffic, Percentage Maximum where, S = the effective span length in feet. 3.24.10.3 For main reinforcement perpendicular to traffic, the specified amount of distribution reinforcement shall be used in the middle half of the slab span, and not less than 50% of the specified amount shall be used in the outer of the slab span. 38 - HIGHWAY BRIDGES - - - - 3.25 DISTRIBUTION OF WHEEL LOADS ON TIMBER FLOORING 3.25 ---- - - - - support. The maximum moment is for a wheel position assumed to be centered between the supports. For the calculation of bending moments in timber flooring each wheel load shall be distributed as follows. 3.25.1 Transverse Flooring 3.25.1.1 In the direction of flooring span, the wheel load shall be distributed over the width of tire as given in Article 3.30. Normal to the direction of flooring span, the wheel load shall be distributed as follows: Plank floor: the width of plank, but not less than 10 inches. Non-interconnected* nail laminated panel floor: 15 inches, but not to exceed panel width. Non-interconnected glued laminated panel floor: 15 inches plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated panel floor, with adequate shear transfer between panels*": 15 inches plus thickness of floor, but not to exceed panel width. Interconnected* glued laminated panel floor, with adequate shear transfer between panels**, not less than 6 inches thick: 15 inches plus twice thickness of floor, but not to exceed panel width. 3.25.1.2 For transverse flooring the span shall be taken as the clear distance between stringers plus one-half the width of one stringer, but shall not exceed the clear span plus the floor thickness. 3.25.1.3 One design method for interconnected glued laminated panel floors is as follows: For glued laminated panel decks using vertically laminated lumber with the panel placed in a transverse direction to the stringers and with panels interconnected using steel dowels, the determination of the deck thickness shall be based on the following equations for maximum unit primary moment and shear.? The maximum shear is for a wheel position assumed to be 15 inches or less from the center line of the Thus, 3R t = -[- whichever is greater 2F" (3 - 26) where, M, = primary bending moment in inch-pounds per inch; R, = primary shear in pounds per inch; x = denotes direction perpendicular to longitudinal stringers; P = design wheel load in pounds; s = effective deck span in inches; 4 r - d-h,.-I7 - &I-:. .iii .---L-- L - - - J -- L L ~ L L I G ~ ~ , u L b 1 i ~ a ,u a a ~ u w11 L -- lllwl~l~~ w ll l i shear, whichever controls; K = design constant depending on design load as follows: H15 Fb K=0.47 = allowable bending stress, in pounds per square inch, based on load applied parallel to the wide face of the laminations (see Tables 13.2.2Aand B); F, = allowable shear stress, in pounds per square inch, based on load applied parallel to the wide face of the laminations (see Tables 13.2.2Aand B).
3.25.1.4 The determination of the minimum size and spacing required of the steel dowels required to transfer the load between panels shall be based on the following equation: where, *The terms interconnected and non-interconnected refer to the joints between the individual nail laminated or glued laminated panels. n **This shear transfer may be accomplished using mechanical fasteners, splines, or dowels along the panel joint or other suitable means. , a = proportional limit stress perpendicular to grain ?The equations are developed for deck panel spans equal to or greater than the width of the tire (as specified in Article 3.30), but not greater than 200 inches.
= number of steel dowels required for the given spans; - R, = (for Douglas fir or Southern pine, use 1,000 psi); total secondary shear transferred, in pounds, determined by the relationship: R,= 6Ps / 1,000 for s 150 inches (3 - 28) or, - R, =(s- 20) for s > 50 inches 2s (3 - 29) My= total secondary moment transferred, in inchpound, determined by the relationship, - Ps M y=(s - 10) for s I 5 0 inches (3 - 30) PS (s - 30) M y=-for s > 50 inches 20 (s - 10) (3- 31) 1,600 R D and M, = shear and moment capacities, respectively, as given in the following table: 3.25.2.2 Normal to the direction of the span the wheel load shall be distributed as follows: Plank floor: 20 inches; Non-interconnected nail laminated floor: width of tire plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated floor, with adequate shear transfer between panels*, not less than 6 inches thick: width of tire plus twice thickness of floor. 3.25.23 For longitudinal flooring the span shall be taken as the clear distance between floor beams plus onehalf the width of one beam but shall not exceed-the clear span plus the floor thickness. 3.253 Longitudinal Glued Laminated Timber Decks 3.253.1 Shear Moment Diameter Capacity Capacity o f h l RD MD in. lb. in.-lb. Steel Stress Coefficients Total Dowel Lengih Raguind in. ---Yin.' Yh3 Cn CM Bending Moment In calculating bending moments in glued laminated timber longitudinal decks, no longitudinal distribution of wheel loads shall be assumed. The lateral distribution shall be determined as follows. The live load bending moment for each panel shall be determined by applying to the panel the fraction of a wheel load determined from the following equations: TWO OR MORE TRAFFIC LANES Load Fraction = 3.25.1.5 In addition, the dowels shall be checked to ensure that the allowable stress of the steel is not exceeded using the following equation: wp L 3.75 +28 orwp whichever is 5.00' greater. ONE TRAFFIC LANE where, = minimum yield point of steel pins in pounds per square inch (see Table - - 10.32.1A); n, R,, My= as previously defined; CR,CM = steel stress coefficients as given in preceding table.
Load Fraction = u 3.25.2 Plank and Nail Laminated Longitudinal Flooring 3.25.2.1 In the direction of the span, the wheel load shall be distributed over 10 inches. wp 4.25 +28 or 3, whichever is 5.50 greater. where, Wp = Width of Panel; in feet (3.5 5 Wp 5 4.5) L =Length of span for simple span bridges and the length of the shortest span for continuous bridges in feet. *Thisshear transfer may be accomplished using mechanical fasteners, splines, or dowels along the panel joint or spreader beams located at inte~valsalong the panels or other suitable means. 3.2532 Shear When calculating the end shears and end reactions for each panel, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution of the wheel load at the supports shall be that determined by the equation: Wheel Load Fraction per Panel W = 2but not less than 1. 4.00 For wheel loads in other positions on the span, the lateral distribution for shear shall be determined by the method prescribed for moment. shall be distributed over a transverse width of 5 feet for bending moment and a width of 4 feet for shear. 3.26.1.2 For composite T-beams of wood and concrete, as described in Article 20.19.2, Division 11, the effective flange width shall not exceed that given in Article 10.38.3. Shear connectors shall be capable of resisting both vertical and horizontal movement. 3.26.2 Distribution of Bending Moments in Continuous Spans 3.26.2.1 Both positive and negative moments shall be distributed in accordance with the following table: Maximum Bending Moments-Percent of Simple S ~ a Moment n 3.2533 Deflections The maximum deflection may be calculated by applying to the panel the wheel load fraction determined by the method prescribed for moment. 3.253.4 Stiffener Arrangement The transverse stiffeners shall be adequately attached to each panel, at points near the panel edges, with either steel plates, thru-bolts, C-clips or aluminum brackets. The stiffener spacing required will depend upon the spacing needed in order to prevent differential panel movement; however, a stiffener shall be placed at mid-span with additional stiffeners placed at intervals not to exceed 10 feet. The stiffness factor EI of the stiffenershall not be less than 80,000 kip-in2. 3.25.4 Continuous Flooring If the flooring is continuous over more than two spans, the maximum bending moment shall be assumed as being 80% of that obtained for a simple span. 3.26 DISTRIBUTION OF WHEEL LOADS AND DESIGN OF COMPOSITE WOODCONCRETE MEMBERS 3.26.1 Distribution of Concentrated Loads for Bending Moment and Shear 3.26.1.1 For freely supported or continuous slab spans of composite wood-concrete construction, as described in Article 16.3.14, Division 11, the wheel loads Span Maximum Live Maximum uniform Dead Load Moments Load Moma~ts Wood Composite Gmcentrated Uniform Subdeck Slab load Load Pas. Neg. Pos. Neg. P06. Neg. Pos. Neg. 50 70 hterior End 2-spa 65 50 60 70 55 70 60 45 60 75 75 85 85 25 30 30 75 85 80 55 65 75 'Continuous beam of 2 equal spans. 3.26.2.2 Impact should be considered in computing stresses for concrete and steel, but neglected for wood. 3.26.3 Design The analysis and design of composite wood-concrete members shall be based on assumptions that account for the different mechanical properties of the components.
A suitable procedure may be based on the elastic properties of the materials as follows: E, = 1 for slab in which the net concrete thickness is less than half the overall depth of the composite section Ec = 2 for slab in which the net concrete thickness is E, at least half the overall depth of the composite section E, - = 18.75 (for Douglas fir and Southern pine) L in which, E, = modulus of elasticity of concrete; E, = modulus of elasticity of wood; E, = modulus of elasticity of steel. 3.27 .- -- -- DIVISION I- -- 3.27 DISTRIBUTION OF WHE~LLOADSON~ STEEL GRID FLOORS* 3.27.1 General 3.27.1.1 The grid floor shall be designed as continuous, but simple span moments may be used and reduced as provided in Article 3.24. 3.27.1.2 The following rules for distribution of loads assume that the grid floor is composed of main elements that span between girders, stringers, or cross beams, and secondary elements that are capable of transferring load between the main elements. 3.27.1.3 Reinforcementfor secondary elements shall consist of bars or shapes welded to the main steel. --- ---- -- - 3.27.3.3 Edges of open grid steel floors shall be supported by suitable means as required. These supports may be longitudinal or transverse, or both, as may be required to support all edges properly, 3.27.3.4 When investigating for fatigue, the minimum cycles of maximum stress shall be used. 3.28 DISTRIBUTION OF LOADS FOR BENDING MOMENT IN SPREAD BOX GIRDERS*" 3.28.1 Interior Beams The live load bending moment for each interior beam in a spread box beam superstructure shall be determined by applying to the beam the fraction (D.F.) of the wheel load (both front and rear) determined by the following equation: 3.27.2 Floors Filled with Concrete 3.27.2.1 The distribution and bending moment shall be as specified for concrete slabs, Article 3.24. The following items specified in that article shall also apply to concrete filled steel grid floors: Longitudinal edge beams Unsupported transverse edges Span lengths 3.27.2.2 The strength of the composite steel and concrete slab shall be determined by means of the "transformed area" method. The allowable stresses shall be as set forthinArticles 8.15.2,8.16.1, and 10.32. 3.27.3 Open Floors 3.27.3.1 A wheel load shall be distributed, normal to the main elements, over a width equal to 1v4 inches per ton of axle load plus twice the distance center to center of main elements. The portion of the load assigned to each main element shall be applied uniformly over a length equal to the rear tire width (20 inches for H 20, 15 inches for H 15). where, NL = number of design tr&~c lanes (Article 3.6); NB = number of beams (4 5 NB5 10); S = beam spacing in feet (6.57 5 S 5 11.00); L = span length in feet; k = 0.07 W - NL (O.lONL - 0.26) - 0.20NB - 0.12; (3-34) W = numeric value of the roadway width between curbs expressed in feet (32 5 W 5 66). 3.28.2 Exterior Beams The live load bending moment in the exterior beams shall be determined by applying to the beams the reaction of the wheel loads obtained by assuming the flooring to act as a simple span (of length S) between beams, but shall not be less than 2NJNB. 3.29 MOMENTS, SHEARS, AND REACTIONS 3.27.3.2 The strength of the section shall be determined by the moment of inertia method. The allowable stresses shall be as set forth in Article 10.32. Maximum moments, shears, and reactions are given in tables, Appendix A, for H 15, H 20, HS 15, and HS 20 loadings. They are calculated for the standard truck or the lane loading applied to a single lane on freely supported spans. It is indicated in the table whether the standard truck or the lane loadings produces the maximum stress. *Provisions in this article shall not apply to orthotropic bridge superstructures. **The provisions of Article 3.12, Reduction in Load intensity, were not applied in the development of the provisions presented in Articles 3.28.1 and 3.28.2. HIGHWAY BRIDGES 42 --- -- . - -- - -- - - 3.30 TIRE CONTACT AREA The tire contact area for the Alternate Military Loading or HS 20-44 shall be assumed as a rectangle with a length in the direction of traffic of 10 inches, and a width of tire of 20 inches. For other design vehicles, the tire contact should be determined by the engineer. - 3.30 -- - Section 4 FOUNDATIONS Part A GENERAL REQUIREMENTS AND MATERIALS 4.1 GENERAL Foundations shall be designed to support all live and dead loads, and earth and water pressure loadings in accordance with the general principles specified in this section. The design shall be made either with reference to service loads and allowable stresses as provided in SERVICE LOAD DESIGN or, alternatively, with reference to load factors, and factored strength as provided in STRENGTH DESIGN. 4.2.2.2 Settlement The settlement of foundations may be determined using procedures described in Articles 4.4,4.5, or 4.6 for service load design and Articles 4.11, 4.12, or 4.13 for strength design, or other generally accepted methodologies.
Such methods are based on soil and rock parameters measured directly or inferred from the results of in situ and/or laboratory tests. 4.2.2.3 Overall Stability 4.2 FOUNDATION TYPE AND CAPACITY 4.2.1 Selection of Foundation Type Selection of foundation type shall be based on an assessment of the magnitude and direction of loading, depth to suitable bearing materials, evidence of previous flooding, potential for liquefaction, undermining or scour, swelling potential, frost depth and ease and cost of construction. 4.2.2 Foundation Capacity Foundations shall be designed to provide adequate structural capacity, adequate foundation bearing capacity with acceptable settlements, and acceptable overall stability of slopes adjacent to the foundations. The tolerable level of structural deformation is controlled by the type and span of the superstructure. 4.2.2.1 Bearing Capacity The bearing capacity of foundations may be estimated using procedures described in Articles 4.4,4.5, or 4.6 for service load design and Articles 4.11, 4.12, or 4.13 for strength design, or other generally accepted theories. Such theories are based on soil and rock parameters measured by in situ and/or laboratory tests. The bearing capacity may also be determined using load tests. The overall stability of slopes in the vicinity of foundations shall be considered as part of the design of foundations. 4.2.3 Soil, Rock, and Other Problem Conditions Geologic and environmental conditions can influence the performance of foundations and may require special consideration during design. To the extent possible, the presence and influence of such conditions shall be evaluated as part of the subsurface exploration program. A representative, but not exclusive, listing of problem conditions requiring special consideration is presented in Table 4.2.3A for general guidance. 4.3 SUBSURFACE EXPLORATION AND TESTING PROGRAMS The elements of the subsurface exploration and testing programs shall be the responsibility of the designer based on the specific requirements of the project and his or her experience with local geologic conditions. 4.3.1 General Requirements As a minimum, the subsurface exploration and testing programs shall define the following, where applicable: Soil strata -Depth, thickness, and variability HIGHWAY BRIDGES TABLE 4.2.3A Problem Conditions Requiring Special Consideration Problem VP~ Soil Description Comments Organic soil; highly plastic clay Sensitive clay Micaceous soil Low strength and high compressibility Potentially large strength loss upon large straining Potentially high compressibility (often saprolitic) Expansive claylsilt; expansive slag Liquefiable soil Potentially large expansion upon wetting Complete strength loss and high deformations due to earthquake loading Potentially large deformations upon wetting (Caliche; Loess) Potentially large expansion upon oxidation Low strength when loaded parallel to bedding Potentially large expansion upon wetting; degrades readily upon exposure to aidwater Expands upon exposure to aidwater Collapsible soil Pyritic soil Laminated rock Expansive shale Pyritic shale Soluble rock Rock Cretaceous shale Weak claystone (Red Beds) Gneissic and Schistose Rock Subsidence Sinkholes/solutioning Condition Negative skin friction1 expansion loading Corrosive environments Permafrostlfrost Capillary water Soluble in flowing and standing water (Limestone, Limerock, Gypsum) Indicator of potentially corrosive ground water Low strength and readily degradable upon exposure to aidwater Highly distorted with irregular weathering profiles and steep discontinuities Typical in areas of underground mining or high ground water extraction Karst topography; typical of areas underlain by carbonate rock strata Additional compressive/uplift load on deep foundations due to settlementhplift of soil Acid mine drainage; degradation of certain soillrock types Typical in northern climates Rise of water level in silts and fine sands leading to strength loss -Identification and classification -Relevant engineering properties (i.e., shear strength, compressibility, stiffness, permeability, expansion or collapse potential, and frost susceptibility) Rock strata -Depth to rock -Identification and classification --Quality (i.e., soundness, hardness, jointing and presence of joint filling, resistance to weathering, if exposed, and solutioning) --Compressive strength (e.g., uniaxial compression, point load index) -Expansion potential Ground water elevation Ground surface elevation Local conditions requiring special consideration Exploration logs shall include soil and rock strata descriptions, penetration resistance for soils (e.g., SPT or q,), and sample recovery and RQD for rock strata. The drilling equipment and method, use of drilling mud, type of SPT hammer (i.e. safety, donut, hydraulic) or cone penetrometer (i.e., mechanical or electrical), and any unusual subsurface conditions such as artesian pressures, boulders or other obstructions, or voids shall also be noted on the exploration logs. 4.3.2 Minimum Depth Where substructure units will be supported on spread footings, the minimum depth of the subsurface exploration shall extend below the anticipated bearing level a minimum of two footing widths for isolated, individual footings where L 5 2B, and four footing widths for footings where L > 5B. For intermediate footing lengths, the minimum depth of exploration may be estimated by linear interpolation as a function of L between depths of 2B and 5B below the bearing level. Greater depths may be required where warranted by local conditions. 4.3.2 --- DIVISION I-DESIGN Where substructure units will be supported on deep foundations, the depth of the subsurface exploration shall extend a minimum of 20 feet below the anticipated pile or shaft tip elevation. Where pile or shaft groups will be used, the subsurface exploration shall extend at least two times the maximum pile group dimension below the anticipated tip elevation, unless the foundations will be end bearing on or in rock. For piles bearing on rock, a minimum of 10 feet of rock core shall be obtained at each exploration location to insure the exploration has not been terminated on a boulder. For shafts supported on or extending into rock, a minimum of 10 feet of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the maximum shaft group dimension for a shaft group, whichever is greater, shall be obtained to insure the exploration has not terminated in a boulder and to determine the physical characteristics of rock within the zone of foundation influence for design. 4.3.3 Minimum Coverage A minimum of one soil boring shall be made for each substructure unit. (See Article 7.1.1 for definition of substructure unit.) For substructure units over 100 feet in width, a minimum of two borings shall be required. 4.3.4 Laboratory Testing Laboratory testing shall be performed as necessary to determine engineering properties including unit weight, shear strength, compressive strength and compressibility.
In the absence of laboratory testing, engineering properties may be estimated based on published test results or local experience. 4.3.5 Scour The probable depth of scour shall be determined by subsurface exploration and hydraulic studies. Refer to Article 1.3.2 and FHWA (1988) for general guidance regarding hydraulic studies and design. Part B SERVICE LOAD DESIGN METHOD ALLOWABLE STRESS DESIGN 4.4 SPREAD FOOTINGS 4.4.1 General 45 4.4.1.2 Footings Supporting Non-Rectangular Columns or Piers Footings supporting circular or regular polygonshaped concrete columns or piers may be designed assuming that the columns or piers act as square members with the same area for location of critical sections for moment, shear, and development of reinforcement. 4.4.1.3 Footings in Fill Footings located in fill are subject to the same bearing capacity, settlement, and dynamic ground stability considerations as footings in natural soil in accordance with Articles 4.4.7.1 through 4.4.7.3. The behavior of both the fill and underlying natural soil shall be considered. 4.4.1.4 Footings in Sloped Portions of Embankments The earth pressure against the back of footings and columns within the sloped portion of an embankment shall be equal to the at-rest earth pressure in accordance with Article 5.5.2. The resistance due to the passive earth pressure of the embankment in front of the footing shall be neglected to a depth equal to a minimum depth of 3 feet, the depth of anticipated scour, freeze thaw action, andlor trench excavation in front of the footing, whichever is greater.
4.4.1.5 Distribution of Bearing Pressure Footings shall be designed to keep the maximum soil and rock pressures within safe bearing values. To prevent unequal settlement, footings shall be designed to keep the bearing pressure as nearly uniform as practical. For footings supported on piles or drilled shafts, the spacing between piles and drilled shafts shall be designed to ensure nearly equal loads on deep foundation elements as may be practical. When footings support more than one column, pier, or wall, distribution of soil pressure shall be consistent with properties of the foundation materials and the structure, and with the principles of geotechnical engineering. 4.4.2 Notations The following notations shall apply for the design of spread footings on soil and rock: 4.4.1.1 Applicability Provisions of this Article shall apply for design of isolated footings, and to combined footings and mats (footings supporting more than one column, pier, or wall). A A' = Contact area of footing (ft2) = Effective footing area for computation of bearing capacity of a footing subjected to eccentric load (ft2); (See Article 4.4.7.1.1.1) HIGHWAY BRIDGES = Base inclination factors (dim); (See Article 4.4.7.1.1.8) = Width of footing (ft); (Minimum plan dimension of footing unless otherwise noted) = Effective width for load eccentric in direction of short side, L unchanged (ft) = Soil cohesion (ksf) = Effective stress soil cohesion (ksf) = Reduced effective stress soil cohesion for punching shear (ksf); (See Article 4.4.7.1) = Adhesion between footing and foundation soil or rock (ksf); (See Article 4.4.7.1.1.3) = Coefficient of consolidation (ft2/yr); (See Article 4.4.7.2.3) = Shear strength of upper cohesive soil layer below footing (ksf); (See Article 4.4.7.1.1.7) = Shear strength of lower cohesive soil layer below footing (ksf); (See Article 4.4.7.1.1.7) = Compression index (dim); (See Article 4.4.7.2.3) = Recompression index (dim); (See Article 4.4.7.2.3) = Compression ratio (dim); (See Article 4.4.7.2.3) = Uniaxial compressive strength of intact rock (ksf) = Recompression ratio (dim); (See Article 4.4.7.2.3) = Coefficient of secondary compression defined as change in height per log cycle of time (dim); (See Article 4.4.7.2.4) = Influence depth for water below footing (ft); (See Article 4.4.7.1.1.6) = Depth to base of footing (ft) = Void ratio (dim); (See Article 4.4.7.2.3) = Void ratio at final vertical effective stress (dim); (See Article 4.4.7.2.3) = Void ratio at initial vertical effective stress (dim); (See Article 4.4.7.2.3) = Void ratio at maximum past vertical effective stress (dim); (See Article 4.4.7.2.3) = Eccentricity of load in the B direction measured from centroid of footing (ft); (See Article 4.4.7.1.1.1) = Eccentricity of load in the L direction measured from centroid of footing (ft); (See Article 4.4.7.1.1.1) = Modulus of intact rock (ksf) = Rock mass modulus (ksf); (See Article 4.4.8.2.2) Es F 4.4.2 = Soil modulus (ksf) = Total force on footing subjected to an in- clined load (k); (See Article 4.4.7.1.1.1) Unconfined compressive strength of concrete (ksf) = Factor of safety against bearing capacity, FS overturning or sliding shear failure (dim) = Depth from footing base to top of second H cohesive soil layer for two-layer cohesive soil profile below footing (ft); (See Article 4.4.7.1.1.7) = Height of compressible soil layer (ft) Hc = Critical thickness of the upper layer of a k t two-layer system beyond which the underlying layer will have little effect on the bearing capacity of footings bearing in the upper layer (ft); (See Article 4.4.7.1.1.7) = Height of longest drainage path in comlld pressible soil layer (ft) = Height of slope (ft); (See Article 4.4.7.1.1.4) Hs = Slope angle from horizontal of ground suri face below footing (deg) = Load inclination factors (dim); (See Article , i,, i 4.4.7.1.1.3) = Influence coefficient to account for rigidity IP and dimensions of footing (dim); (See Article 4.4.8.2.2) = Center-to-center spacing between adjacent e footings (ft) L = Length of footing (ft) = Effective footing length for load eccentric L' in direction of long side, B unchanged (ft) = Length (or width) of footing having positive L1 contact pressure (compression) for footing loaded eccentrically about one axis (ft) = Exponential factor relating B/L or LIB ran tios for inclined loading (dim); (See Article 4.4.7.1.1.3) N = Standard penetration resistance (blowslft) = Standard penetration resistance corrected N1 for effects of overburden pressure (blows/ ft); (See Article 4.4.7.2.2) N,, N,, Nq = Bearing capacity factors based on the value of internal friction of the foundation soil (dim); (See Article 4.4.7.1) = Modified bearing capacity factor to account Nm for layered cohesive soils below footing (dim); (See Article 4.4.7.1.1.7) = Coefficient factor to estimate q,,, for rock Nm (dim); (See Article 4.4.8.1.2) Ns = Stability number (dim); (See Article 4.4.7.1.1.4) fi = DMSION I-DESIGN 47 = Modified bearing capacity factors for ef- = Time factor (dim); (See Article 4.4.7.2.3) fects of footing on or adjacent sloping ground (dim); (See Article 4.4.7.1.1.4) = Tangential component of force on footing = Depth from footing base down to the high- Or> = Maximum resisting force between footing base and foundation soil or rock for sliding failure (k) = Effective overburden pressure at base of footing (ksf) = Normal component of force on footing (k) = Allowable uniform bearing pressure or contact stress (ksf) = Cone penetration resistance (ksf) = Maximum rotting contact pressure (ksf) = Maximum normal component of load supported by foundation soil or rock at ultimate bearing capacity (k) = Minimum magnitude of footing contact pressure (ksf) = Vertical stress at base of loaded area (ksf); (See Article 4.4.7.2.1) = Ultimate bearing capacity for uniform bearing pressure (ksf) = Ultimate bearing capacity of footing supported in the upper layer of a two-layer system assuming the upper layer is infinitely thick (ksf); (See Article 4.4.7.1.1.7) = Ultimate bearing capacity of a fictitious footing of the same size and shape as the actual footing, but supported on surface of the second (lower) layer of a two-layer system (ksf); (See Article 4.4.7.1.1.7) = Resultant of pressure on base of footing (k) = Radius of circular footing or B12 for square footing (ft); (See Article 4.4.8.2.2) = Rock Quality Designation (dim) = Footing shape factors (dim); (See Article 4.4.7.1.1.2) = Undrained shear strength of soil &sf) = Consolidation settlement (ft); (See Article 4.4.7.2.3) = Elastic or immediate settlement (ft); (See Article 4.4.7.2.2) = Secondary settlement (ft); (See Article 4.4.7.2.4) = Total settlement (ft); (See Article 4.4.7.2) = Time to reach specified average degree of consolidation (yr); (See Article 4.4.7.2.3) = Arbitrary time intervals for determination of Ss(yr); (See Article 4.4.7.2.4) est anticipated ground water level (ft); (See Article 4.4.7.1.1.6) = Angle of inclination of the footing base from the horizontal (radian) = Reduction factor (dim); (See Article 4.4.8.2.2) = Length to width ratio of footing (dim) = Punching index = BLl[2(B + L)H] (dim); (See Article 4.4.7.1.1.7) = Factor to account for footing shape and rigidity (dim); (See Article 4.4.7.2.2) = Total unit weight of soil or rock (kcf) = Buoyant unit weight of soil or rock (kcf) = Moist unit weight of soil (kcf) = Angle of friction between footing and foundation soil or rock (deg); (See Article 4.4.7.1.1.3) = Differential settlement between adjacent footings (ft); (See Article 4.4.7.2.5) = Vertical strain (dim); (See Article 4.4.7.2.3) =Vertical strain at final vertical effective stress (dim); (See Article 4.4.7.2.3) = Initial vertical strain (dim); (See Article 4.4.7.2.3) = Vertical strain at maximum past vertical effective stress (dim); (See Article 4.4.7.2.3) = Angle of load eccentricity (deg) = Shear strength ratio (c21c1)for two layered cohesive soil system below footing (dim); (See Article 4.4.7.1.1.7) = Reduction factor to account for three-dimensional effects in settlement analysis (dim); (See Article 4.4.7.2.3) = Poisson's ratio (dim) = Final vertical effective stress in soil at depth interval below footing (ksf); (See Article 4.4.7.2.3) = Initial vertical effective stress in soil at depth interval below footing (ksf); (See Article 4.4.7.2.3) = Maximum past vertical effective stress in soil at depth interval below footing (ksf); (See Article 4.4.7.2.3) = Angle of internal friction (deg) = Effective stress angle of internal friction ((leg) = Reduced effective stress soil friction angle for punching shear (ksf); (See Article 4.4.7.1) 48 HIGHWAY BRIDGES The notations for dimension units include the following: dim = Dimensionless; deg = degree; ft = foot; k = kip; klft = kiplft; ksf = kip/ft2;kcf = kip/ft3;lb = pound; in. = inch; and psi = pound per square inch. The dimensional units provided with each notation are presented for illustration only to demonstrate a dimensionally correct combination of units for the footing capacity procedures presented herein. If other units are used, the dimensional correctness of the equations shall be confirmed.
4.4.2 4.4.4 Soil and Rock Property Selection Soil and rock properties defining the strength and compressibility characteristics of the foundation materials are required for footing design. Foundation stability and settlement analyses for design shall be conducted using soil and rock properties based on the results of field and/or laboratory testing.
4.4.5 Depth 4.4.3 Design Terminology Refer to Figure 4.4.3Afor terminology used in the design of spread footing foundations. 4.4.5.1 Minimum Embedment and Bench Width Footings not otherwise founded on sound, non-degradeable rock surfaces shall be embedded a sufficient FIGURE 4.4.3A Design Terminology for Spread Footing Foundations 4.4.5.1 DIVISION I-DESIGN depth to provide adequate bearing, scour and frost heave protection, or 2 feet to the bottom of footing, whichever is greatest.
For footings constructed on slopes, a minimum horizontal distance of 4 feet, measured at the top of footing, shall be provided between the near face of the footing and the face of the finished slope. 4.4.5.2 Scour Protection Footings supported on soil or degradable rock strata shall be embedded below the maximum computed scour depth or protected with a scour countermeasure. Footings supported on massive, competent rock formations which are highly resistant to scour shall be placed directly on the cleaned rock surface. Where required, additional lateral resistance should be provided by drilling and grouting steel dowels into the rock surface rather than blasting to embed the footing below the rock surface. Footings on piles may be located above the lowest anticipated scour level provided the piles are designed for this condition.Assume that only one-half of the maximum anticipated scour has occurred when designing for earthquake loading. Where footings on piles are subject to damage by boulders or debris during flood scour, adequate protection shall be provided. Footings shall be constructed so as to neither pose an obstacle to water traffic nor be exposed to view during low flow.
4.4.5.3 Footing Excavations Footing excavations below the ground water table, particularly in granular soils having relatively high permeability, shall be made such that the hydraulic gradient in the excavation bottom is not increased to a magnitude that would cause the foundation soils to loosen or soften due to the upward flow of water. Further, footing excavations shall be made such that hydraulic gradients and material removal do not adversely affect adjacent structures. Seepage forces and gradients may be evaluated by fl

DESIGN 65 -- - - - --- TABLE 4.4.8.2.2A Summary of Poisson's Ratio for Intact Rock Modified after Kulhawy (1978) Rock Type No. of Values Granite Gabbro Diabase Basalt Quartzite Marble Gneiss Schist Sandstone Siltstone Shale Limestone Dolostone 22 3 6 11 6 5 11 12 12 3 3 19 5 No. of Rock V~es 22 3 6 11 6 5 11 11 9 3 3 19 5 Poisson's Ratio, v Maximum Minimum Mean Standard Deviation 0.39 0.20 0.38 0.32 0.22 0.40 0.40 0.31 0.46 0.23 0.18 0.33 0.35 0.09 0.16 0.20 0.16 0.08 0.17 0.09 0.02 0.08 0.09 0.03 0.12 0.14 0.20 0.18 0.29 0.23 0.14 0.28 0.22 0.12 0.20 0.18 0.09 0.23 0.29 0.08 0.02 0.06 0.05 0.05 0.08 0.09 0.08 0.11 0.06 0.06 0.06 0.08 TABLE 4.4.8.2.2B Summary of Elastic Moduli for Intact Rock Modified after Kdhawy (1978) Rock 'Me No. of Values Granite Diorite Gabbro Diabase Basalt Quartzite Marble Gneiss Slate Schist Phyllite Sandstone Siltstone Shale Limestone Dolostone 26 3 3 7 12 7 14 13 11 13 3 27 5 30 30 17 ("1.0 x 106psi = 1.44 x 1@ ksf. No. of Rock V~es 26 3 3 7 12 7 13 13 2 12 3 19 5 14 30 16 Elastic Modulus, E, (psi x lo6)(') Standard Maximum Minimum Mean Deviation 14.5 16.2 12.2 15.1 12.2 12.8 10.7 11.9 3.79 10.0 2.51 5.68 4.76 5.60 13.0 11.4 0.93 2.48 9.80 10.0 4.20 5.29 0.58 4.13 0.35 0.86 1.25 0.09 0.38 0.001 0.65 0.83 7.64 7.45 11 .O 12.8 8.14 9.59 6.18 8.86 1.39 4.97 1.71 2.13 2.39 1.42 5.70 4.22 3.55 6.19 0.97 1.78 2.60 2.32 2.49 2.31 0.96 3.18 0.57 1.19 1.65 1.45 3.73 3.44 4.4.9 HIGHWAY BRIDGES 66 TRUS (HOlBS) HENMION ET. AL. - } VLRT STIFF MAR0 UNIAXIAL COMPRESSIVE CLAY STRENGTH Co-osi 110' FIGURE 4.4.8.2.2A Relationship Between Elastic Modulus and Uniaxial Compressive Strength for Intact Rock Modified after Deere (1968) or near a slope by limiting equilibrium methods of analysis which employ the Modified Bishop, simplified Janbu, Spenser or other generally accepted methods of slope stability analysis. Where soil and rock parameters and ground water levels are based on in-situ andlor laboratory tests, the minimum factor of safety shall be 1.3 (or 1.5 where abutments are supported above a slope).

Otherwise, the minimum factor of safety shall be 1.5 (or 1.8 where abutments are supported above a retaining wall). 4.4.10 Dynamic/Seismic Design Refer to Division I-A and Lam and Martin (1986a; 1986b)for guidance regarding the design of footings subjected to dynamic and seismic loads. 4.4.11 Structural Design 4.4.11.1 Loads and Reactions 4.4.11.1.1 Action of Loads and Reactions Footings shall be considered as under the action of downward forces, due to the superimposed loads, resisted by an upward pressure exerted by the foundation materials and distributed over the area of the footings as determined by the eccentricity of the resultant of the downward forces.

Where piles are used under footings, the upward reaction of the foundation shall be considered as a series of concentrated loads applied at the pile centers, each pile being assumed to carry the computed portion of the total footing load. 4.4.11.1.1 DIVISION I-DESIGN 4.4.11.1.2 Isolated and Multiple Footing Reactions When a single isolated footing supports a column, pier or wall, the footing shall be assumed to act as a cantilever. When footings support more than one column, pier, or wall, the footing slab shall be designed for the actual conditions of continuity and restraint. 4.4.11.2 Moments 4.4.11.2.1 Critical Section External moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over the entire area of the footing on one side of that vertical plane. The critical section for bending shall be taken at the face of the column, pier, or wall. In the case of columns that are not square or rectangular, the section shall be taken at the side of the concentric square of equivalent area. For footings under masonry walls, the critical section shall be taken halfway between the middle and edge of the wall. For footings under metallic column bases, the critical section shall be taken halfway between the column face and the edge of the metallic base. 4.4.11.2.2 Distribution of Reinforcement Reinforcement of one-way and two-way square footings shall be distributed uniformly across the entire width of footing. Reinforcement of two-way rectangular footings shall be distributed uniformly across the entire width of footing in the long direction. In the short direction, the portion of the total reinforcement given by Equation 4.4.11.2.2-1 shall be distributed uniformly over a band width (centered on center line of column or pier) equal to the length of the short side of the footing. The remainder of reinforcement required in the short direction shall be distributed uniformly outside the center band width of footing. Reinforcement in band width - 2 -Total reinforcement in short direction (P +1) (4.4.11.2.2 - 1) p is the ratio of the footing length to width. 4.4i11.3 Shear 67 porting a column, pier, or wall. For footings supporting a column or pier with metallic base plates, the critical section shall be measured from the location defined in Article 4.4.11.2. 4.4.11.3.2 Footings on Piles or Drilled Shafs Shear on the critical section shall be in accordance with the following: Entire reaction from any pile or drilled shaft whose center is located d,/2 or more outside the critical section shall be considered as producing shear on that section. Reaction from any pile or drilled shaft whose center is located 412 or more inside the critical section shall be considered as producing no shear on that section. For the intermediate position of pile or drilled shaft centers, the portion of the pile or shaft reaction to be considered as producing shear on the critical section shall be based on linear interpolation between full value at dp/2 outside the section and zero value at 412 inside the section. 4.4.11.4 Development of Reinforcement 4.4.11.4.1 DevelopmentLength Computation of development of reinforcement in footings shall be in accordance with Articles 8.24 through 8.32. 4.4.11.4.2 Critical Section Critical sections for development of reinforcement shall be assumed at the same locations as defined in Article 4.4.11.2 and at all other vertical planes where changes in section or reinforcement occur. See also Article 8.24.1.5. 4.4.11.5 Transfer of Force at Base of Column 4.4.11.5. I Transfer of Force All forces and moments applied at base of column or pier shall be transferred to top of footing by bearing on concrete and by reinforcement. 4.4.11.3.1 Critical Section Computation of shear in footings, and location of critical section, shall be in accordance with Articles 8.15.5.6 or 8.16.6.6. Location of critical section shall be measured from the face of column, pier or wall, for footings sup- 4.4.11.5.2 Lateral Forces Lateral forces shall be transferred to supporting footing in accordance with shear-transfer provisions of Articles 8.15.5.4 or 8.16.6.4. 68 HIGHWAY BRIDGES 4.4.11.5.3 BearingBearing on concrete at contact surface between supporting and supported member shall not exceed concrete bearing strength for either surface as given in Articles 8.15.2 or 8.16.7. 4.4.11.5.4 Reinforcement Reinforcement shall be provided across interface between supporting and supported member either by extending main longitudinal reinforcement into footings or by dowels. Reinforcement across interface shall be sufficient to satisfy all of the following: Reinforcement shall be provided to transfer all force that exceeds concrete bearing strength in supporting or supported member. If required loading conditions include uplift, total tensile force shall be resisted by reinforcement. Area of reinforcement shall not be less than 0.005 times gross area of supported member, with a minimum of four bars.

4.4.1 1.5.5 Dowel Size Diameter of dowels, if used, shall not exceed diameter of longitudinal reinforcement by more than 0.15 inch. 4.4.11.5.6 Development k n g t h For transfer of force by reinforcement, development of reinforcement in supporting and supported member shall be in accordance with Articles 8.24 through 8.32. 4.4.11.5.7 Splicing At footings, No. 14 and 18 main longitudinal reinforcement, in compression only, may be lap spliced with footing dowels to provide the required area, but not less than that required by Article 4.4.11.5.4. Dowels shall not be larger than No. 11 and shall extend into the column a distance of not less than the development length of the No. 14 or 18 bars or the splice length of the dowels, whichever is greater; and into the footing a distance of not less than the development length of the dowels. 4.4.11.6 Unreinforced Concrete Footings 4.4.11.6.1 Design Stress Design stresses in plain concrete footings or pedestals shall be computed assuming a linear stress distribution. For footings and pedestals cast against soil, effective thickness used in computing stresses shall be taken as the 4.4.11.5.3 overall thickness minus 3 inches. Extreme fiber stress in tension shall not exceed that specified in Article 8.15.2.1.1. Bending need not be considered unless projection of footing from face to support member exceeds footing thickness. 4.4.11.6.2 Pedestals The ratio of unsupported height to average least lateral dimension of plain concrete pedestals shall not exceed 3. 4.5 DRnTEN PILES 4.5.1 General The provisions of this article shall apply to the design of axially and laterally loaded driven piles in soil or extending through soil to rock. 4.5.1.1 Application Piling may be considered when footings cannot be founded on rock, or on granular or stiff cohesive soils within a reasonable depth. At locations where soil conditions would normally permit the use of spread footings but the potential for scour exists, piles may be used as a protection against scour. Piles may also be used where an unacceptable amount of settlement of spread footings may occur. 4.5.1.2 Materials Piles may be structural steel sections, steel pipe, precast concrete, cast-in-place concrete, prestressed concrete, timber, or a combination of materials. In every case, materials shall be supplied in accordance with the provisions of this article. 4.5.1.3 Penetration Pile penetration shall be determined based on vertical and lateral load capacities of both the pile and subsurface materials. In general, the design penetration for any pile shall be not less than 10 feet into hard cohesive or dense granular material nor less than 20 feet into soft cohesive or loose granular material. Where the depth to dense material or rock is less than 10 feet, spread footings should be considered. Piles for trestle or pile bents shall meet the above requirements and, additionally, unless refusal is encountered, shall penetrate not less than Y 3 the unsupported length of the pile. - 4.5.1.4 4.5.1.4 Lateral Tip Restraint No piling shall be used to penetrate a soft or loose upper stratum overlying a hard or firm stratum unless the piles penetrate the hard or firm stratum by a sufficient distance to fix the ends against lateral movement of the pile tip. Driving points or shoes may be necessary to accomplish this penetration. 4.5.1.5 Estimated Lengths Estimated pile lengths for each substructure shall be shown on the plans and shall be based upon careful evaluation of available subsurface information, static and lateral capacity calculations, and/or past experience. 4.5.1.6 Estimated and Minimum Tip Elevation Estimated and minimum pile tip elevations for each substructure should be shown on the contract plans. Estimated pile tip elevations shall reflect the elevation where the required ultimate pile capacity can be obtained. Minimum pile tip elevations shall reflect the penetration required to support lateral pile loads (including scour considerations where appropriate) and/or penetration of overlying, unsuitable soil strata.

4.5.1.7 Pies Through Embankment Fill Piles to be driven through embankments shall penetrate a minimurn of 10 feet through original ground unless refusal on bedrock or competent bearing strata occurs at a lesser penetration. Fill used for embankment construction shall be a select material which shall not obstruct pile penetration to the required depth.

The maximum size of any rock particles in the fill shall not exceed 6 inches. Predrilling or spudding pile locations may be required, particularly for displacement piles. 4.5.1.8 69 DIVISION I-DESIGN Test Piles Test piles shall be considered for each substructure unit (See Article 7.1.1 for definition of substructure unit) to determine pile installation characteristics, evaluate pile capacity with depth and to establish contractor pile order lengths. Piles may be tested by static loading, dynamic testing, conducting driveability studies, or a combination thereof, based upon the knowledge of subsurface conditions. The number of test piles required may be increased in non-uniform subsurface conditions. Test piles may not be required where previous experience exists with the same pile type and ultimate pile capacity in similar subsurface conditions. 4.5.2 Pile Q p e s Piles shall be classified as "friction" or "end bearing" or a combination of both according to the manner in which load transfer is developed. 4.5.2.1 Friction Piles A pile shall be considered to be a friction pile if the major portion of support capacity is derived from soil resistance mobilized along the side of the embedded pile. 4.5.2.2 End Bearing Piles A pile shall be considered to be an end bearing pile if the major portion of support capacity is derived from the resistance of the foundation material on which the pile tip rests. 4.5.2.3 Combination Friction and End Bearing Piles Under certain soil conditions and for certain pile materials, the bearing capacity of a pile may be considered as the sum of the resistance mobilized on the embedded shaft and that developed at the pile tip, even though the forces that are mobilized simultaneously are not necessarily maximum values. 4.5.2.4 Batter Piles When the lateral resistance of the soil surrounding the piles is inadequate to counteract the horizontal forces transmitted to the foundation, or when increased rigidity of the entire structure is required, batter piles should be used in the foundation. Where negative skin friction loads are expected, batter piles should be avoided, and an alternate method of providing lateral restraint should be used. 4.5.3 Notations The following notations shall apply for the design of driven pile foundations: A, = Area of pile circumference (ft2) A, = Area of pile tip (ft2) B = Pile diameter or width (ft) f: = Concrete compression strength (ksi) f, = Concrete compression stress due to prestressing after all losses (ksi) FS = Factor of safety (dim) 70 HIGHWAY BRIDGES - - -- - - F, = Yield strength of steel (ksi) L = Pile length (ft) QdI = Design capacity (k) Qs = Ultimate shaft resistance (k) QT = Ultimate tip resistance (k) Quit = Ultimate pile capacity (k) r, = Unit side resistance (ksi) R, = Side resistance (k) rt = Unit tip resistance (ksi) R, = Tip resistance (k) p = Percentage of reinforcement (dim) a, = Allowable stress (ksi) The notations for dimension units include the following: dim = Dimensionless; ft = foot; square feet = ft2; k = kip; ksi = l~iplin.~; and in. = inch. The dimensional units provided with each notation are presented for illustration only to demonstrate a dimensionally correct combination of units for the footing capacity procedures presented herein. If other units are used, the dimensional correctness of the equations shall be confirmed. 4.5.4 Design Terminology Refer to Figure 4.5.4A for terminology used in the design of driven pile foundations. 4.5.5 Selection of Soil and Rock Properties Soil and rock properties defining the strength and compressibility characteristicsof the foundation materials, are required for driven pile design. Refer to Article 4.3 for guidelines for subsurface exploration to obtain soil and rock properties. 4.5.6 Selection of Design Pile Capacity The design pile capacity is the maximum load the pile shall support with tolerable movement. In determining the design pile capacity, the following items shall be considered: Ultimate geotechnical capacity; and Structural capacity of the pile section.

4.5.6.1 Ultimate Geotechnical Capacity The ultimate axial capacity of a driven pile shall be determined from: 4.5.3 - -- The allowable design axial capacity shall be determined from: 4.5.6.1.1 Factors Affecting Axial Capacity In determining the design axial capacity, consideration shall be given to: The difference between the supporting capacity of a single pile and that of a group of piles; The capacity of an underlying strata to support the load of the pile group; The effects of driving piles on adjacent structures or slopes; The possibility of scour and its effect on axial and lateral capacity; The effects of negative skin friction or downdrag loads from consolidating soil and the effects of uplift loads from expansive or swelling soils; The influence of construction techniques such as augering or jetting on capacity; and The influence of fluctuations in the elevation of the ground water table on capacity. 4.5.6.1.2 Axial Capacity in Cohesive Soils The ultimate axial capacity of piles in cohesive soils may be calculated using a total stress method (e.g., Tomlinson, 1957) for undrained loading conditions, or an effective stress method (e.g., Meyerhof, 1976) for drained loading conditions.

The axial capacity may also be calculated from in-situ testing methods such as the cone penetration (e.g., Schmertmann, 1978) or pressuremeter tests (e.g., Baguelin, 1978). 4.5.6.1.3 Axial Capacity in Cohesionless Soils The ultimate axial capacity of piles in cohesionless soils may be calculated using an empirical effective stress method (e.g., Nordlund, 1963) or from in-situ testing methods and analysis such as the cone penetration (e.g., Schmertmann, 1978) or pressuremeter tests (e.g., Baguelin, 1978). 4.5.6.1.4 Axial Capacity on Rock For piles driven to competent rock, the structural capacity in Article 4.5.7 will generally govern the design axial capacity.

For piles driven to weak rock such as shale and mudstone or poor quality weathered rock, a static load test is recommended. Pile relaxation should be considered in certain kinds of rock when performing load tests. 4.5.6.2 DIVISION I-DESIGN 1 71 = SHAFT RESISTANCE R, RT L D B = PILE LENGTH = EMBEDMENT DEPTH = PlLE DlAMETfR A, At rn TIP RESISTANCE CIRCUMFERENTIAL AREA TIP CROSS SECTIONAL AREA FIGURE 4.5.4A Design Terminology for Driven Pile Foundations 4.5.6.2 Factor of Safety Selection The selection of the factor of safety to be applied to the ultimate axial geotechnical capacity shall consider the reliability of the ultimate soil capacity determination and pile installation control. Recommended values for the factor of safety depending upon the degree of construction control specified on the plans are presented in Table 4.5.6.2A.

All factors of safety are based on fulltime observation of pile installation. The design pile capacity shall be specified on the plans so the factor of safety can be adjusted if the specified construction control is altered. 4.5.6.3 Settlement The settlement of axially loaded piles and pile groups at the allowable loads shall be estimated. Elastic analysis, load transfer andlor finite element techniques (e.g., Vesic, 1977 or Poulos and Davis, 1980) may be used. The settlement of the pile or pile group shall not exceed the tolerable movement limits of the structure. 4.5.6.4 Group Pile Loading Group pile capacity should be determined as the product of the group efficiency, number of piles in the group, 72 HIGHWAY BRIDGES TABLE 4.5.6.2A Recommended Factor of Safety on Ultimate Geotechnical Capacity Based on Specified Construction Control Increasing Construction Control Subsurface exploration Static calculation Dynamic formula Wave equation Dynamic measurement and analysis Static load test Factor of safety (')X X X 3.50 X X X X X X X X X X X X X 2.75 2.25 X (32.00 X X 1.90 (')X = Construction Control Specified on Contract Plans. (=)Forany combination of construction control that includes an approved static load test, a factor of safety of 2.0 may be used. and the capacity of a single pile. In general, a group efficiency value of 1.0 should be used except for friction piles in cohesive soils. The efficiency factor for friction piles in cohesive soils with a center-to-center pile spacing less than 3.OB should be 0.7. Center-to-center pile spacings less than 2.5B are not recommended. 4.5.6.5 Lateral Loads on Piles The design of laterally loaded piles is usually governed by lateral movement criteria.

The design of laterally loaded piles shall account for the effects of soillrockstructure interaction between the pile and ground (e.g., Reese, 1984). Methods of analysis evaluating the ultimate capacity or deflection of laterally loaded piles (e.g., Broms, 1964aand 1964b;Singh, et al., 1971) may be used for preliminary design only as a means to evaluate appropriate pile sections. 4.5.6.6 Uplift Loads on Piles The uplift design capacity of single piles and pile groups shall be determined in accordance with Articles 4.5.6.6.1 and 4.5.6.6.2, respectively. Proper provision shall be made for anchorage of the pile into the pile cap. 4.5.6.6.1 Single Pile The uplift design capacity for a single pile shall not exceed one-third of the ultimate frictional capacity determined by a static analysis method. Alternatively, the uplift capacity of a single pile can be determined by uplift load tests in conformance with ASTM D 3689 (ASTM, 1988). If determined by load tests, the allowable uplift design capacity shall not exceed 50% of the failure uplift load. 4.5.6.6.2 4.5.6.4 Pile Group The uplift design capacity for a pile group shall be the lesser of: (1) The single pile uplift design capacity multiplied by the number of piles in the group, or (2) two-thirds of the effective weight of the pile group and the soils contained within a block defined by the perimeter of the group and the embedded length of the piles, or (3) onehalf the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the embedded length of the piles. 4.5.6.7 Vertical Ground Movement The potential for external loading on a pile by vertical ground movements shall be considered as part of the design. Vertical ground movements may result in negative skin friction or downdrag loads due to settlement of compressible soils or may result in uplift loads due to heave of expansive soils. For design purposes, the full magnitude of maximum vertical ground movement shall be assumed. 4.5.6.7.1 Negative Skin Friction The potential for external loading on a pile by negative skin frictionldowndrag due to settlement of compressible soil shall be considered as a part of the design. Evaluation of negative skin friction shall include a load-transfer method of analysis to determine the neutral point (i.e., point of zero relative displacement) and load distribution along shaft (e.g., Fellenius, 1984, Reese and O'Neill, 1988). Due to the possible time dependence associated with vertical ground movement, the analysis shall consider the effect of time on load transfer between the ground and shaft and the analysis shall be performed for the time period relating to the maximum axial load transfer to the pile. If necessary, negative skin friction loads that cause excessive settlement may be reduced by application of bitumen or other viscous coatings to the pile surfaces before installation. 4.5.6.7.2 Expansive Soil Piles driven in swelling soils may be subjected to uplift forces in the zone of seasonal moisture change. Piles shall extend a sufficient distance into moisture-stable soils to provide adequate resistance to swelling uplift forces. In addition, sufficient clearance shall be provided between the ground surface and the underside of pile caps or grade beams to preclude the application of uplift loads at the pile cap. Uplift loads may be reduced by application of bitumen or other viscous coatings to the pile surface in the swelling zone. DMSION I 4.5.6.8 -- 4.5.6.8 DynamicJSeismic Design TABLE 4.5.7.3A Allowable Working Stress for Round Timber Piles Refer to Division I-A for guidance regarding the design of driven piles subjected to dynamic and seismic loads. 4.5.7 Structural Capacity of Pile Section 4.5.7.1 Load Capacity Requirements Piles shall be designed as structural members capable of safely supporting all loads imposed on them by the structure or surrounding soil. 4.5.7.2 Piles Extending Above Ground Surface For portions of piles in air or water, or in soil not capable of providing adequate lateral support throughout the pile length to prevent buckling, the structural design provisions for compression members of Sections 8,9,10, and 13 shall apply except: timber piles shall be designed in accordance with Article 13.5 using the allowable unit stresses given in Article 13.2 for lumber and in Table 4.5.7.3A. 4.5.7.3 Allowable Stresses in Piles The maximum allowable stress on a pile shall not exceed the following limits in severe subsurface conditions. Where pile damage or deterioration is possible, it may be prudent to use a lower stress level than the maximum allowable stress. For steel H-piles, the maximum allowable stress shall not exceed 0.25Fyover the cross-sectionalarea of the pile, not including the area of any tip reinforcement.

The maximum allowable stress may be increased to 0.33Fyin conditions where pile damage is unlikely. Static and/or dynamic load test and evaluation confirming satisfactory results should be performed when using 0.33Fy. For unfilled steel pipe piles, the maximum allowable stress shall not exceed 0.25Fy over the minimum cross-sectional area of the pile. The maximum allowable stress may be increased to 0.33Fyin conditions where pile damage is unlikely. Static and/or dynamic load test and evaluation confirming satisfactory results should be performed when using 0.33Fy. For concrete filled steel pipe piles, the maximum allowable stress shall not exceed 0.25Fy + 0.40f; applied over the cross-sectional area of the steel pipe and on the cross-sectional area of the concrete, respectively. Species Allowable Unit Working Stress Compression Parallel to Grain for Normal Duration of Loading a,(psi) Ash, white Beech Birch Chestnut Cypress, Southern Cypress, Tidewater red Douglas Fir, coast type Douglas Fir, inland Elm, rock Elm, soft Gum, black and red Hemlock, Eastern Hemlock, West Coast Hickory Larch Maple, hard Oak, red and white Pecan P i e , Lodgepole Pine, Norway Pine, Southern Pine, Southern, dense Poplar, yellow Redwood Spruce, Eastern Tupelo For precast concrete piles, the maximum allowable stress shall not exceed 0.33fL on the gross cross-sectional area of the concrete. For prestressed concrete piles fully embedded in soils providing lateral support, the maximum allowable stress shall not exceed 0.33f,' - 0.27f, on the gross cross-sectional area of the concrete. For round timber piles, the maximum allowable stress shall not exceed the values in Table 4.5.7.3A for the pile tip area. For sawn timber piles, the values applicable to "wet condition"for allowable compression parallel to grain shall be used in accordance with Article 13.2. 4.5.7.4 Cross-Section Adjustment for Corrosion For concrete-filled pipe piles where corrosion may be expected, %6 inch shall be deducted from the shell thick- 74 HIGHWAY BRIDGES ness to allow for reduction in section due to corrosion. Area of shell shall be included in determining percentage of reinforcement, p. 4.5.7.5 Scour The probable depth of scour shall be determined by subsurface exploration and hydraulic studies as described in Article 4.3.5. If heavy scour is expected, consideration shall be given to designing the portion of the pile that would be exposed as a column. In all cases, the pile length shall be determined such that the design structural load may be safely supported entirely below the probable scour depth. The pile shall be of adequate cross-section to withstand the driving necessary to penetrate through the anticipated scour depth to the design embedment. 4.5.8 Protection Against Corrosion and Abrasion Where conditions of exposure warrant, concrete encasement or other corrosion protection shall be used on steel piles and steel shells. Exposed steel piles or steel shells shall not be used in salt or brackish water, and only with caution in fresh water. Where the piling is exposed to the abrasive action of the bed load of materials, the section shall be increased in thickness or positive protection shall be provided. 4.5.9 Wave Equation Analysis The constructability of the pile foundation design should be evaluated using a wave equation computer program. The wave equation should be used to confirm that the design pile section can be installed to the desired depth, ultimate capacity, and within the allowable driving stress levels specified in Article 4.5.11 using an appropriately sized driving system. 4.5.7.4 Steel piles 0.90Fy(Compression) 0.90Fy(Tension) 0.85fl (Compression) Concrete piles 0.70Fyof Steel Reinforcement (Tension) Prestressed concrete piles 0.85f,' - f, (Compression) Normal environments 3 + f, (Tension) (f,' and f, must be in psi. The resulting max stress is also in psi.) Severe corrosive environments f, (Tension) Timber piles 3ua(Compression) 3ua(Tension) Driving stresses may be estimated by performing wave equation analyses or by dynamic monitoring of force and acceleration at the pile head during pile driving. 4.5.12 Tolerable Movement Tolerable axial and lateral displacement criteria for driven pile foundations shall be developed by the structural engineer consistent with the function and type of structure, fixity of bearings, anticipated service life, and consequences of unacceptable displacements on the structural performance. Driven pile displacement analyses shall be based on the results of in-situ andlor laboratory testing to characterize the load deformation behavior of the foundation materials. Refer to Article 4.4.7.2.5 for additional guidance regarding tolerable vertical and horizontal movement criteria. 4.5.13 Buoyancy The effect of hydrostatic pressure shall be considered in the design as provided in Article 3.19. 4.5.10 Dynamic Monitoring 4.5.14 Protection Against Deterioration Dynamic monitoring may be specified for piles installed in difficult subsurface conditions such as soils with obstructions and boulders, or a steeply sloping bedrock surface to evaluate compliance with structural pile capacity. Dynamic monitoring may also be considered for geotechnicalcapacity verification where the size of the project or other limitations deter static load testing. 4.5.11 Maximum Allowable Driving Stresses Maximum allowable driving stresses in pile material for top driven piles shall not exceed the following limits: 4.5.14.1 Steel Piles A steel pile foundation design shall consider that steel piles may be subject to corrosion, particularly in fill soils, low ph soils (acidic) and marine environments. A field electric resistivity survey, or resistivity testing and ph testing of soil and ground water samples should be used to evaluate the corrosion potential. Methods of protecting steel piling in corrosive environments include use of protective coatings, cathodic protection, and increased pile steel area. 4.5.14.2 DIVISION I-DESIGN -- - -- 75 - - - 4.5.14.2 Concrete Piles A concrete pile foundation design shall consider that deterioration of concrete piles can occur due to sulfates in soil, ground water, or sea water; chlorides in soils and chemical wastes; acidic ground water and organic acids. Laboratory testing of soil and ground water samples for sulfates and ph is usually sufficient to assess pile deterioration potential. A full chemical analysis of soil and ground water samples is recommended when chemical wastes are suspected. Methods of protecting concrete piling can include dense impermeable concrete, sulfate resisting portland cement, minimum cover requirementsfor reinforcing steel, and use of epoxies, resins, or other protective coatings. 4.5.14.3 Timber Piles A timber pile foundation design shall consider that deterioration of timber piles can occur due to decay from wetting and drying cycles or from insects or marine borers. Methods of protecting timber piling include pressure treating with creosote or other wood preservers. 4.5.15 Spacing, Clearances, and Embedment 4.5.15.1 P i e Footings 4.5.15.1.1 Pile Spacing Pile footings shall be proportioned such that the minimum center-to-center pile spacing shall exceed the greater of 2 feet 6 inches or 2.5 pile diameterslwidths.

The distance from the side of any pile to the nearest edge of the pile footing shall not be less than 9 inches. 4.5.15.1.2 Minimum Projection into Cap The tops of piles shall project not less than 12 inches into concrete after all damaged pile material has been removed, but in special cases, it may be reduced to 6 inches. 4.5.16 Precast Concrete Piles 4.5.16.1 Size and Shape Precast concrete piles shall be of approved size and shape but may be either of uniform section or tapered. In general, tapered piling shall not be used for trestle construction except for the portion of the pile which lies below the ground line; nor shall tapered piles be used in any location where the piles are to act as columns. 4.5.16.2 Minimum Area In general, concrete piles shall have a cross-sectional area, measured above the taper, of not less than 98 square inches. In saltwater a minimum cross-sectional area of 140 square inches shall be used. If a square section is employed, the comers shall be chamfered at least 1 inch. 4.5.16.3 Minimum Diameter of Tapered Piles The diameter of tapered piles measured at the point shall be not less than 8 inches. In all cases the diameter shall be considered as the least dimension through the center. 4.5.16.4 Driving Points Piles preferably shall be cast with a driving point and, for hard driving, preferably shall be shod with a metal shoe of approved pattern. 4.5.16.5 Vertical Reinforcement Vertical reinforcement shall consist of not less than four bars spaced uniformly around the perimeter of the pile, except that if more than four bars are used, the nurnber may be reduced to four in the bottom 4 feet of the pile. The amount of reinforcement shall be at least 1%percent of the total section measured above the taper. 4.5.16.6 Spiral Reinforcement 4.5.15.2 Bent Caps Where a reinforced concrete beam is cast-in-place and used as a bent cap supported by piles, the concrete cover at the sides of the piles shall be a minimum of 6 inches.

The piles shall project at least 6 inches and preferably 9 inches into the cap, although concrete piles may project a lesser distance into the cap if the projection of the pile reinforcement is sufficient to provide adequate bond. The full length of vertical steel shall be enclosed with spiral reinforcement or equivalent hoops. The spiral reinforcement at the ends of the pile shall have a pitch of 3 inches and gage of not less than No. 5 (U.S. Steel Wire Gage). In addition, the top 6 inches of the pile shall have five turns of spiral winding at 1-inch pitch. For the remainder of the pile, the lateral reinforcement shall be a No. 5 gage spiral with not more than 6-inch pitch, or Y4inch round hoops spaced on not more than 6-inch centers.

76 HIGHWAY BRIDGES 4.5.16.7 - 4.5.16.7 Reinforcement Cover The reinforcement shall be placed at a clear distance from the face of the pile of not less than 2 inches and, when piles are used in saltwater or alkali soils, this clear distance shall not be less than 3 inches. 4.5.16.8 Splices Piles may be spliced provided that the splice develops the full strength of the pile. Splices should be detailed on the contract plans. Any alternative method of splicing that provides equal results may be considered for approval. 4.5.16.9 Handling Stresses In computing stresses due to handling, the static loads shall be increased by 50% as an allowance for impact and shock.

4.5.17 Cast-in-PlaceConcrete Piles 4.5.17.1 equate lateral restraint. Where the shell is smooth pipe and more than 0.12 inch in thickness, it may be considered as load carrying in the absence of corrosion. Where the shell is corrngated and is at least 0.075 inch in thickness, it may be considered as providing confinement in the absence of corrosion. 4.5.17.5 Reinforcement into Superstructure Sufficient reinforcement shall be provided at the junction of the pile with the superstructure to make a suitable connection. The embedment of the reinforcement into the cap shall be as specified for precast piles. 4.5.17.6 Shell Requirements The shell shall be of sufficient thickness and strength so that it will hold its original form and show no harmful distortion after it and adjacent shells have been driven and the driving core, if any, has been withdrawn. The plans shall stipulate that alternative designs of the shell must be approved by the Engineer before any driving is done. Materials 4.5.17.7 Splices Cast-in-place concrete piles shall be, in general, cast in metal shells that shall remain permanentlyin place. However, other types of cast-in-place piles, plain or reinforced, cased or uncased, may be used if the soil conditions permit their use and if their design and method of placing are satisfactory. Piles may be spliced provided the splice develops the full strength of the pile. Splices should be detailed on the contract plans. Any alternative method of splicing providing equal results may be considered for approval. 4.5.17.8 Reinforcement Cover 4.5.17.2 Shape Cast-in-place concrete piles may have a uniform crosssection or may be tapered over any portion. 4.5.17.3 Minimum Area The minimum area at the butt of the pile shall be 100 inches and the minimum diameter at the tip of the pile shall be 8 inches. Above the butt or taper, the minimum size shall be as specified for precast piles. The reinforcement shall be placed a clear distance of not less than 2 inches from the cased or uncased sides. When piles are in corrosive or marine environments, or when concrete is placed by the water or sluny displacement methods, the clear distance shall not be less than 3 inches for uncased piles and piles with shells not sufficiently corrosion resistant. 4.5.18 Steel H-Piles 4.5.18.1 Metal Thickness 4.5.17.4 General Reinforcement Requirements Cast-in-place piles, carrying axial loads only where the possibility of lateral forces being applied to the piles is insignificant, need not be reinforced where the soil provides adequate lateral support. Those portions of cast-in-place concrete piles that are not supported laterally shall be designed as reinforced concrete columns in accordance with Articles 8.15.4 and 8.16.4, and the reinforcing steel shall extend 10 feet below the plane where the soil provides ad- Steel piles shall have a minimum thickness of web of 0.400 inch.

Splice plates shall not be less than YSin. thick. 4.5.18.2 Splices Piles shall be spliced to develop the net section of pile. The flanges and web shall be either spliced by butt welding or with plates that are welded, riveted, or bolted. Splices shall be detailed on the contract plans. Prefabri- 4.5.18.2 DIVISION I-DESIGN -- cated spficers may be used d the splice can develop the net section of the pile in compression, tension, shear, and bending. 4.5.18.3 Caps a portion of t w i l e , the pile will be investigated for column action. The provisions of Article 4.5.8 shall apply to unfilled tubular steel piles. 4.5.20 Prestressed Concrete Piles In general, caps are not required for steel piles embedded in concrete. 4.5.18.4 Lugs, Scabs, and Core-Stoppers These devices may be used to increase the bearing capacity of the pile where necessary. They may consist of structural shapes-welded, riveted, or bolted--of plates welded between the flanges, or of timber or concrete blocks securely fastened. 4.5.18.5 Point Attachments If pile penetration through cobbles, boulders, debris fill or obstructions is anticipated, pile tips shall be reinforced with structural shapes or with prefabricated cast steel points. Cast steel points shall meet the requirements of ASTM A 27. 4.5.19 Unfilled lhbular Steel Piles 4.5.19.1 77 Metal Thickness Piles shall have a minimum thickness not less than indicated in the following table: Outside Diameter Less than 14 inches 14 inches and over Wall Thickness 0.25 inch 0.375 inch 4.5.19.2 Splices Piles shall be spliced to develop the full section of the pile. The piles shall be spliced either by butt welding or by the use of welded sleeves. Splices shall be detailed on the contract plans. 4.5.19.3 Driving Tubular steel piles may be driven either closed or open ended. Closure plates should not extend beyond the perimeter of the pile. 4.5.19.4 Column Action Where the piles are to be used as part of a bent structure or where heavy scour is anticipated that would expose 4.5.20.1 Size and Shape Prestressed concrete piles that are generally octagonal, square or circular shall be of approved size and shape. Air entrained concrete shall be used in piles that are subject to freezing and thawing or wetting and drying. Concrete in prestressed piles shall have a minimum compressive strength, fd, of 5,000 psi at 28 days. Prestressed concrete piles may be solid or hollow. For hollow piles, precautionary measures should be taken to prevent breakage due to internal water pressure during driving, ice pressure in trestle piles, and gas pressure due to decomposition of material used to form the void. 4.5.20.2 Main Reinforcement Main reinforcement shall be spaced and stressed so as to provide a compressivestress on the pile after losses, f,, general not less than 700 psi to prevent cracking during handling and installation.

Piles shall be designed to resist stresses developed during handling as well as under service load conditions. Bending stresses shall be investigated for all conditions of handling, taking into account the weight of the pile plus 50-percent allowance for impact, with tensile stresses limited to 5 a . 4.5.20.3 Vertical Reinforcement The full length of vertical reinforcement shall be enclosed within spiral reinforcement. For piles up to 24 inches in diameter, spiral wire shall be No. 5 (U.S. Steel Wire Gage). Spiral reinforcement at the ends of these piles shall have a pitch of 3 inches for approximately 16 turns.

In addition, the top 6 inches of pile shall have five turns of spiral winding at 1-inch pitch. For the remainder of the pile, the vertical steel shall be enclosed with spiral reinforcement with not more than 6-inch pitch. For piles having diameters greater than 24 inches, spiral wire shall be No. 4 (U.S. Steel Wire Gage). Spiral reinforcement at the end of these piles shall have a pitch of 2 inches for approximately 16 turns. In addition, the top 6 inches of pile shall have four turns of spiral winding at 1% inches. For the remainder of the pile, the vertical steel shall be enclosed with spiral reinforcement with not more than 4inch pitch.

The reinforcement shall be placed at a clear distance from the face of the prestressed pile of not less than 2 inches. HIGHWAY BRIDGES 78 4.5.20.4 Hollow Cylinder Piles Large diameter hollow cylinder piles shall be of approved size and shape. The wall thickness for cylinder piles shall not be less than 5 inches. The grouting of posttensioning tendons shall be in accordance with Article 4.33.9, Division 11.

4.5.20.5 Splices When prestressed concrete piles are spliced, the splice shall be capable of developing the full section of the pile. Splices shall be detailed on the contract pl

perpendicul the shaft axis extend aminimum of 2B the d i p p a strata to the potential for shear along natural bedding planes and other slippage surfaces associated with stratification. 4.6.5.3.3.2 Rock Mass Discontinuities The strength and compressibility of rock will be affected by the presence of discontinuities (joints and fractures). The influence of discontinuities on shaft behavior will be dependent on their attitude, frequency and condition, and shall be evaluated on a case-by-case basis as necessary. 4.6.5.3.3.3 Method of Construction The effect of the method of construction on the engineering properties of the rock and the contact between the rock and shaft shall be considered as a part of the design process. 4.6.5.4 Factors of Safety Drilled shafts in soil or socketed in rock shall be designed for a minimum factor of safety of 2.0 against bearing capacity failure (end bearing, side resistance or combined) when the design is based on the results of a load test conducted at the site. Otherwise, shafts shall be designed for a minimum factor of safety 2.5. The minimum recommended factors of safety are based on an assumed normal level of field quality control during shaft construction. If a normal level of field quality control cannot be assured, higher minimum factors of safety shall be used. 4.6.5.3.3.1 4.6.5.5 Deformation of Axially Loaded Shafts The settlement of axially loaded shafts at working or allowable loads shall be estimated using elastic or load transfer analysis methods. For most cases, elastic analysis will be applicable for design provided the stress levels in the shaft are moderate relative to Q,,,.
Where stress levels are high, considerationshould be given to methods of load transfer analysis. 4.6.5.5.1 Shafs in Soil Settlements should be estimated for the design or working load. 4.6.5.5.1.1 Cohesive Soil The short-term settlement of shafts in cohesive soil may be estimated using Figures 4.6.5.5.1.1A and 4.6.5.5.1.18. The curves presented indicate the proporof the ultimate side resistance (Qs) and ultimate tip tions resistance (QT)mobilized at various magnitudes of settlement. The total axial load on the shaft (Q) is equal to the sum of the mobilized side resistance (Qs) and mobilized tip resistance (Q,). The settlement in Figure 4.6.5.5.1.1A incorporates the effects of elastic shortening of the shaft provided the shaft is of typical length (i.e., D < 100 ft).
For longer shafts, the effects of elastic shortening may be estimated using the following: pe = PD/AEc (4.6.5.5.1.1-1) For a shaft with an enlarged base in cohesive soil, the diameter of the shaft at the base (Bb) should be used in Figure 4.6.5.5.1.1B to estimate shaft settlement at the tip. Refer to Article 4.4.7.2.3 for procedures to estimate the consolidation settlement component for shafts extending into cohesive soil deposits. 4.6.5.5.1.2 Cohesionless Soil The short-term settlement of shafts in cohesionless soil may be estimated using Figures 4.6.5.5.1.2A and 4.6.5.5.1.2B. The curves presented indicate the proportions of the ultimate side resistance (Qs) and ultimate tip resistance (QT)mobilized at various magnitudes of settlement. The total axial load on the shaft (Q) is equal to the sum of the mobilized side resistance (Qs) and mobilized tip resistance (Q,). Elastic shortening of the shaft shall be estimated using the following relationship: 4.6.5.5.1.2 DIVISION 1. - - Range of Results Trrnd Line Range of Results .--- Trend Line Settlement , Dlameter of Shaft FIGURE 4.6.5.5.1.1A Load Transfer in Side Resistance Versus Settlement Drilled Shafts in Cohesive Soil After Reese and O'NeiU (1988) 0 1 2 3 4 5 6 7 8 9 1 0 Settlement of Base , % Mametar of Base FIGURE 4.6.5.5.1.1B Load Transfer in Tip Bearing Settlement Drilled Shafts in Cohesive Soil After Reese and O'Neill(1988) 4.6.5.5.1.3 Mixed Soil Projile The short-term settlement of shafts in a mixed soil profile may be estimated by summing the proportional settlement components from layers of cohesive and cohesionless soil comprising the subsurface profile. 4.6.5.5.2 Shafts Socketed into Rock In estimating the displacementof rock-socketed drilled shafts, the resistance to deformation provided by overlying soil deposits may be ignored. Otherwise, the load transfer to soil as a function of displacement may be estimated in accordance with Article 4.6.5.5.1. The butt settlement (p,) of drilled shafts fully socketed into rock may be determined using the following which is modified to include elastic shortening of the shaft. Refer to Figure 4.6.5.5.2B to determine I,,. The rock mass modulus (E d should be determined based on the results of in-situ testing (e.g., pressure-meter) or estimated from the results of laboratory tests in which Emis the modulus of intact rock specimens, and (E,) is estimated in accordance with Article 4.4.8.2.2. For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E,, such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A, may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of a~= 0.15 should be used to estimate Em. 4.6.5.5.3 Tolerable Movement Refer to Figure 4.6.5.5.2A to determine I,,. The uplift displacement (p,) at the butt of drilled shafts fully socketed into rock may be determined using the following which is modified to include elastic shortening of the shaft: Tolerable axial displacement criteria for drilled shaft foundations shall be developed by the structural designer consistent with the function and type of structure, fixity of bearings, anticipated service life, and consequences of unacceptable displacements on the structure performance. Drilled shaft displacement analyses shall be based on the results of in-situ and/or laboratory testing to characterize 88 HIGHWAY BRIDGES 4.6.5.5.3 1.4 - 12 - ----- Range of Re¶ults lor Deflralon-Softdng Rerponu Range of Re~ults Trend Cine Trend Line o 0.0 a0.2 1RI 10.8 10.8 'l.0 1l2 11.4 1l.8 41.8 20 t 1 Settlement , Oiameter of Shaft FIGURE 4.6.5.5.1.2A Load Tkansfer in Side Resistance Versus Settlement Drilled Shafts in Cohesionless Soil After Reese and O'Neill(1988) the load-deformationbehavior of the foundation materials. Refer to Article 4.4.7.2.5 for additional guidance regarding tolerable vertical and hoizontal movement criteria. 4.6.5.6 ---- Range of Rasulta fw OlflecUon-Hardming R a ~ p o n r Lateral Loading The design of laterally loaded drilled shafts shall account for the effects of soil/rock-structureinteraction between the shaft and ground (e.g., Reese, 1984; Borden and Gabr, 1987). Methods of analysis evaluating the ultimate capacity or deflection of laterally loaded shafts (e.g., Broms, 1964a,b;Singh, et al., 1971) may be used for preliminary design only as a means to determine approximate shaft dimensions. 4.6.5.6.1 Factors Affecting Laterally Loaded Shafts 4.6.5.6.1.1 Soil Layering The design of laterally loaded drilled shafts in layered soils shall be based on evaluation of the soil parameters characteristic of the respective layers. 4.6.5.6.1.2 Ground Water The highest anticipated water level shall be used for design. J Settlement of Bare , % Olameter of Bare FIGURE 4.6.5.5.1.2B Load Transfer in Tip Bearing Versus Settlement Drilled Shafts in Cohesionless Soil After Reese and O'Neill(1988) 4.6.5.6.1.3 Scour The potential for loss of lateral capacity due to scour shall be considered in the design. Refer to Article 1.3.2 and FHWA (1988) for general guidance regarding hydraulic studies and design. If heavy scour is expected, consideration shall be given to designing the portion of the shaft that would be exposed as a column. In all cases, the shaft length shall be determined such that the design structural load can be safely supported entirely below the probable scour depth. 4.6.5.6.1.4 Group Action There is no reliable rational method for evaluating the group action for closely spaced, laterally loaded shafts. Therefore, as a general guide, drilled shafts in a group may be considered to act individually when the center-to-center (CTC) spacing is greater than 2.5B in the direction normal to loading, and CTC > 8B in the direction parallel to loading. For shaft layouts not conforming to these criteria, the effects of shaft interaction shall be considered in the design. As a general guide, the effects of group action for in-line CTC < 8B may be considered using the ratios (CGS, 1985) appearing on page 89. 4.6.5.6.1.4 DIVISION I-DESIGN 89 FIGURE 4.6.5.5.2B Influence Coefficient for Elastic Uplift Displacementof Rock-Socketed Drilled Shafts Modified after Pells and 'hrner (1979) 4.6.5.6.1.7 Sloping Ground FIGURE 4.6.5.5.2A Influence Coefficientfor Elastic Settlement of Rock-Socketed Drilled Shafts Modified after Pells and 'hrner (1979) CTC Shaft Spacing for In-line Loading 8B 6B 4B 3B Ratio of Lateral Resistance of Shaft in Group to Single Shaft 1.00 0.70 0.40 0.25 4.6.5.6.1.5 Cyclic Loading The effects of traffic, wind, and other nonseismic cyclic loading on the load-deformation behavior of laterally loaded drilled shafts shall be considered during design. Analysis of drilled shafts subjected to cyclic loading may be considered in the COM624 analysis (Reese, 1984). 4.6.5.6.1.6 Combined Axial and Lateral Loading The effects of lateral loading in combination with axial loading shall be considered in the design. Analysis of drilled shafts subjected to combined loading may be considered in the COM624 analysis (Reese, 1984). For drilled shafts which extend through or below sloping ground, the potential for additional lateral loading shall be considered in the design. The general method of analysis developed by Borden and Gabr (1987) may be used for the analysis of shafts in stable slopes. For shafts in marginally stable slopes, additional consideration should be given for low factors of safety against slope failure or slopes showing ground creep, or when shafts extend through fills overlying soft foundation soils and bear into more competent underlying soil or rock formations. For unstable ground, detailed explorations, testing and analysis are required to evaluate potential additional lateral loads due to slope movements. 4.6.5.6.2 Tolerable Lateral Movements Tolerable lateral displacement criteria for drilled shaft foundations shall be developed by the structural designer consistent with the function and type of structure, fixity of bearings, anticipated service life, and consequences of unacceptable displacements on the structure performance. Drilled shaft lateral displacement analysis shall be based on the results of in-situ andlor laboratory testing to characterize the load-deformation behavior of the foundation materials. 90 HIGHWAY BRIDGES 4.6.5.7 DynamidSeismic Design Refer to Division I-A and Lam and Martin (1986a; 1986b) for guidance regarding the design of drilled shafts subjected to dynamic and seismic loads. 4.6.6 Structural Design and General Shaft Dimensions 4.6.6.1 General Drilled shafts shall be designed to insure that the shaft will not collapse or suffer loss of serviceability due to excessive stress andlor deformation. Shafts shall be designed to resist failure following applicable procedures presented in Section 8. All shafts should be sized in 6-inch increments with a minimum shaft diameter of 18 inches. The diameter of shafts with rock sockets should be sized a minimum of 6 inches larger than the diameter of the socket. The diameter of columns supported by shafts shall be less than or equal to B. 4.6.6.2 Reinforcement Where the potential for lateral loading is insignificant, drilled shafts need to be reinforced for axial loads only. Those portions of drilled shafts that are not supported laterally shall be designed as reinforced concrete columns in accordance with Articles 8.15.4 and 8.16.4, and the reinforcing steel shall extend a minimum of 10 feet below the plane where the soil provides adequate lateral restraint. Where permanent steel casing is used and the shell is smooth pipe and more than 0.12 inch in thickness, it may be considered as load carrying in the absence of corrosion. The design of longitudinal and spiral reinforcement shall be in conformance with the requirements of Articles 8.18.1 and 8.18.2.2, respectively. Development of deformed reinforcement shall be in conformance with the requirements of Articles 8.24,8.26, and 8.27. 4.6.6.2.1 Longitudinal Bar Spacing The minimum clear distance between longitudinal reinforcement shall not be less than 3 times the bar diameter nor 3 times the maximum aggregate size. If bars are bundled in forming the reinforcing cage, the minimum clear distance between longitudinal reinforcement shall 4.6.5.6.7 not be less than 3 times the diameter of the bundled bars. Where heavy reinforcement is required, consideration may be given to an inner and outer reinforcing cage. 4.6.6.2.2 Splices Splices shall develop the full capacity of the bar in tension and compression. The location of splices shall be staggered around the perimeter of the reinforcing cage so as not to occur at the same horizontal plane. Splices may be developed by lapping, welding, and special approved connectors. Splices shall be in conformance with the re8.32quirements 4.6.6.2.3 Transverse Reinforcement Transverse reinforcement shall be designed to resist stresses caused by fresh concrete flowing from inside the cage to the side of the excavated hole. Transversereinorcement may be constructed of hoops or spiral steel. f 4.6.6.2.4 Handling Stresses Reinforcement cages shall be designed to resist handling and placement stresses. 4.6.6.2.5 ~einforcehentCover The reinforcement shall be placed a clear distance of not less than 2 inches from the permanently cased or 3 inches from the uncased sides. When shafts are constructed in corrosive or marine environments, or when concrete is placed by the water or slurry displacement methods, the clear distance shall not be less than 4 inches for uncased shafts and shafts with permanent casings not sufficiently corrosion resistant. The reinforcement cage shall be centered in the hole using centering devices. All steel centering devices shall be epoxy coated. 4.6.6.2.6 Reinforcement into Superstructure Sufficient reinforcement shall be provided at the junction of the shaft with the superstructure to make a suitable connection.
The embedment of the reinforcement into the cap shall be in conformance with Articles 8.24 and 8.25. 4.6.6.3 Enlarged - Bases Enlarged bases shall be designed to insure that plain concrete is not overstressed. The enlarged base shall slope at a side angle not less than 30 degrees from the vertical and have a bottom diameter not greater than 3 times the 4.6.6.3 DIVISION I-DESIGN diameter of the shaft. The thickness of the bottom edge of the enlarged base shall not be less than 6 inches. 4.6.6.4 Center-to-Center Shaft Spacing The center-to-center spacing of drilled shafts should be 3B or greater to avoid interference between adjacent shafts during construction. If closer spacing is required, the sequence of construction shall be specified and the interaction effects between adjacent shafts shall be evaluated by the designer. 4.6.7 Load Testing 4.6.7.1 General Where necessary, a full scale load test (or tests) should be conducted on a drilled shaft foundation(s) to confirm response to load. Load tests shall be conducted using a test shaft(s) constructed in a manner and of dimensions and materials identical to those planned for the production shafts into the materials planned for support. Load testing should be conducted whenever special site conditions or combinations of load are encountered, or when structures of special design or sensitivity (e.g., large bridges) are to be supported on drilled shaft foundations. 4.6.7.2 Load Testing Procedures Load tests shall be conducted following prescribed written procedures which have been developed from accepted standards (e.g., ASTM, 1989; Crowther, 1988) and modified, as appropriate, for the conditions at the site. Standard pile load testing procedures developed by ASTM which may be modified for testing drilled shafts include: r ASTM D 1143, Standard Method of Testing Piles Under Static Axial Compressive Load; ASTM D 3689, Standard Method of Testing Individual Piles Under Static Axial Tensile Load; and ASTM D 3966, Standard Method for Testing Piles Under Lateral Loads. A simplified procedure for testing drilled shafts permitting determination of the relative contribution of side resistance and tip resistance to overall shaft capacity is also available (Osterberg, 1984). As a minimum, the written test procedures should include the following: r Apparatus for applying loads including reaction system and loading system. 91 Apparatus for measuring movements. Apparatus for measuring loads. r Procedures for loading including rates of load application, load cycling and maximum load. Procedures for measuring movements. Safety requirements.
Data presentation requirements and methods of data analysis. Drawings showing the procedures and materials to be used to construct the load test apparatus. As a minimum, the results of the load test(s) shall provide the load-deformation response at the butt of the shaft.
When appropriate, information concerning ultimate load capacity, load transfer, lateral load-displacement with depth, the effects of shaft group interaction, the degree of fixity provided by caps and footings, and other data pertinent to the anticipated loading conditions on the production shafts shall be obtained. 4.6.7.3 Load Test Method Selection Selection of an appropriate load test method shall be based on an evaluation of the anticipated types and duration of loads during service, and shall include consideration of the following: The immediate goals of the load test (i.e., to proof load the foundation and verify design capacity). The loads expected to act on the production foundation (compressive and/or uplift, dead and/or live), and the soil conditions predominant in the region of concern. The local practice or traditional method used in similar soillrock deposits. Time and budget constraints. Part C STRENGTH DESIGN METHOD LOAD FACTOR DESIGN Note to User: Article Number 4.7 has been omitted intentionally. 4.8 SCOPE Provisions of this section shall apply for the design of spread footings, driven piles, and drilled shaft foundations.
92 HIGHWAY BRIDGES 4.9 DEFINITIONS Batter Pile-A pile driven at an angle inclined to the vertical to provide higher resistance to lateral loads.
Combination End-Bearing and Friction Pile-Pile that derives its capacity from the contributions of both end bearing developed at the pile tip and resistance mobilized along the embedded shaft. Deep Foundation-A foundation which derives its support by transferring loads to soil or rock at some depth below the structure by end bearing, by adhesion or friction or both.
Design Load-All applicable loads and forces or their related internal moments and forces used to proportion a foundation. In load factor design, design load refers to nominal loads multiplied by appropriate load factors. Design Strength-The maximum load-carrying capacity of the foundation, as defined by a particular limit state. In load factor design, design strength is computed as the product of the nominal resistance and the appropriate performance factor.
Drilled Shaft-A deep foundation unit, wholly or partly embedded in the ground, constructed by placing fresh concrete in a drilled hole with or without steel reinforcement. Drilled shafts derive their capacities from the surrounding soil andlor from the soil or rock strata below their tips. Drilled shafts are also commonly referred to as caissons, drilled caissons, bored piles or drilled piers. End-Bearing Pile-A pile whose support capacity is derived principally from the resistance of the foundation material on which the pile tip rests. Factored Load-Load, multiplied by appropriate load factors, used to proportion a foundation in load factor design. Friction Pile-A pile whose support capacity is derived principally from soil resistance mobilized along the side of the embedded pile. Limit State-A limiting condition in which the foundation and/or the structure it supports are deemed to be unsafe (i.e., strength limit state), or to be no longer fully useful for their intended function (i.e., serviceability limit state). Load Effect-The force in a foundation system (e.g., axial force, sliding force, bending moment, etc.) due to the applied loads.
Load Factor-A factor used to modify a nominal load effect, which accounts for the uncertainties associated with the determination and variability of the load effect. Load Factor Design-A design method in which safety provisions are incorporated by separately accounting for uncertainties relative to load and resistance. Nominal Load-A typical value or a code-specified value for a load. 4.9 Nominal Resistance-The analytically estimated loadcarrying capacity of a foundation calculated using nominal dimensions and material properties, and established soil mechanics principles. Performance Factor-A factor used to modify a nominal resistance, which accounts for the uncertainties associated with the determination of the nominal resistance and the variability of the actual capacity. Pile-A relatively slender deep foundation unit, wholly or partly embedded in the ground, installed by driving, drilling, augering, jetting, or otherwise, and which derives its capacity from the surrounding soil and/or from the soil or rock strata below its tip. Piping-Progressive erosion of soil by seeping water, producing an open pipe through the soil, through which water flows in an uncontrolled and dangerous manner. Shallow Foundation-A foundation which derives its support by transferring load directly to the soil or rock at shallow depth. If a single slab covers the supporting stratum beneath the entire area of the superstructure, the foundation is known as a combined footing. If various parts of the structure are supported individually, the individual supports are known as spread footings, and the foundation is called a footing foundation. 4.10 LIMIT STATES, LOAD FACTORS, AND RESISTANCE FACTORS 4.10.1 General All relevant limit states shall be considered in the design to ensure an adequate degree of safety and serviceability. 4.10.2 Semceability Limit States Service limit states for foundation design shall include: -settlements, and -lateral displacements. The limit state for settlement shall be based upon rideability and economy. The cost of limiting foundation movements shall be compared to the cost of designing the superstructure so that it can tolerate larger movements, or of correcting the consequences of movements through maintenance, to determine minimum lifetime cost. More stringent criteria may be established by the owner. 4.10.3 Strength Limit States Strength limit states for foundation design shall include: DIVISION I-DESIGN -bearing resistance failure, -excessive loss of contact, -sliding at the base of footing, -loss of overall stability, and -structural capacity. Foundations shall be proportioned such that the factored resistance is not less than the effects of factored loads specified in Section 3. 4.10.4 Strength Requirement Foundations shall be proportioned by the methods specified in Articles 4.11 through 4.13 so that their design strengths are at least equal to the required strengths. The required strength is the combined effect of the factored loads for each applicable load combination stipulated in Article 3.22. The design strength is calculated for each applicable limit state as the nominal resistance, R,, multiplied by an appropriate performance (or resistance) factor, Methods for calculating nominal resistance are provided in Articles 4.11 through 4.13, and values of performance factors are given in Article 4.10.6. +. 4.10.5 Load Combinations and Load Factors Foundations shall be proportioned to withstand safely all load combinations stipulated in Article 3.22 which are applicable to the particular site or foundation type. With the exception of the portions of concrete or steel piles that are above the ground line and are rigidly connected to the superstructure as in rigid frame or continuous structures, impact forces shall not be considered in foundation design. (See Article 3.8.1.) Values of y and p coefficients for load factor design, as given in Table 3.22.1A, shall apply to strength limit state considerations; while those for service load design (also given in Table 3.22.1A) shall apply to serviceability considerations. 4.11 SPREAD FOOTINGS 4.11.1 General Considerations 4.11.1.1 General Provisions of this article shall apply to design of isolated footings, and where applicable, to combined footings. Special attention shall be given to footings on fill. Footings shall be designed to keep the soil pressure as nearly uniform as practicable. The distribution of soil pressure shall be consistent with properties of the soil and the structure, and with established principles of soil mechanics. 4.11.1.2 Depth The depth of footings shall be determined with respect to the character of the foundation materials and the possibility of undermining. Footings at stream crossings shall be founded at depth below the maximum anticipated depth of scour as specified in Article 4.11.1.3. Footings not exposed to the action of stream current shall be founded on a firm foundation and below frost level. Consideration shall be given to the use of either a geotextile or graded granular filter layer to reduce susceptibility to piping in rip rap or abutment backfill. 4.11.1.3 Scour Protection Footings supported on soil or degradable rock strata shall be embedded below the maximum computed scour depth or protected with a scour counter-measure. Footings supported on massive, competent rock formations which are highly resistant to scour shall be placed directly on the cleaned rock surface. Where required, additional lateral resistance shall be provided by drilling and grouting steel dowels into the rock surface rather than blasting to embed the footing below the rock surface. 4.11.1.4 Frost Action 4.10.6 Performance Factors Values of performance factors for different types of foundation systems at strength limit states shall be as specified inTables 4.10.6-1,4.10.6-2, and4.10.6-3, unless regionally specific values are available. If methods other than those given in Tables 4.10.6-1, 4.10.6-2, and 4.10.6-3 are used to estimate the soil capacity, the performance factors chosen shall provide the same reliability as those given in these tables. In regions where freezing of the ground occurs during the winter months, footings shall be founded below the maximum depth of frost penetration in order to prevent damage from frost heave. 4.11.1.5 Anchorage Footings which are founded on inclined smooth solid rock surfaces and which are not restrained by an overburden of resistant material shall be effectively anchored by 94 HIGHWAY BRIDGES 4.11.1.5 TABLE 4.10.6-1 Performance Factors for Strength Limit States for Shallow Foundations Performance Factor (+) 'Qpe of Limit State I. Bearing capacity a. Sand -Semi-empirical procedure using SPT data -Semi-empirical procedure using CPT data -Rational methodusing & estimated from SPT data using +f estimated from CPT data b. Clay -Semi-empirical procedure using CPT data -Rational method using shear strength measured in lab tests using shear strength measured in field vane tests using shear strength estimated from CPT data c. Rock --Semi-empirical procedure (Carter and Kulhawy) 2. Sliding Sliding on clay is controlled by the strength of the clay when the clay shear strength is less than 0.5 times the normal stress, and is controlled by the normal stress when the clay shear strength is greater than 0.5 times the normal stress. a. Precast concrete placed on sand using estimated from SPT data using +f estimated from CPT data b. Concrete cast in place on sand using +f estimated from SPT data using +f estimated from CPT data c.
Clay (where shear strength is less than 0.5 times normal pressure) using shear strength measured in lab tests using shear strength measured in field tests using shear strength estimated from CPT data d. Clay (where the strength is greater than 0.5 times normal pressure) where +f = liictional angle of sand, SPT = Standard Penetration Test, CPT = Cone Penetration Test.
means of rock anchors, rock bolts, dowels, keys or other suitable means. Shallow keying of large footing areas shall be avoided where blasting is required for rock removal. 4.11.1.7 Uplift Where foundations may be subjected to uplift forces, they shall be investigated both for resistance to pullout and for their structural strength. 4.11.1.6 Groundwater 4.11.1.8 Deterioration Footings shall be designed for the highest anticipated position of the groundwater table. The influence of the groundwater table on bearing capacity of soils or rocks, and settlements of the structure shall be considered. In cases where seepage forces are present, they should. also be included in the analyses. Deterioration of the concrete in a foundation by sulfate, chloride, and acid attack should be investigated. Laboratory testing of soil and groundwater samples for sulfates, chloride and pH should be sufficient to assess deterioration potential.
When chemical wastes are suspected, a more thorough chemical anal- TABLE 4.10.6-2 Performance Factors for Geotechnical Strength Limit States in Axially Loaded Piles Performance Factor Method/SoiYCondition Ultimate bearing capacity of single piles Skin friction a-method $-method A-method End bearing Clay (Skempton, 1951) Sand (Kulhawy, 1983) & from CPT & from SIT Rock (Canadian Geotech. Society, 1985) Skin friction and end bearing 0.45 0.35 0.50 Clay 0.45 0.55 0.80 0.70 0.65 Uplift capacity of single piles a-method @-method bmethod SPT-method CPT-method Load Test 0.60 0.40 0.45 0.35 0.45 0.80 Group uplift capacity Sand Clay 0.55 0.55 Block failure ysis of soil and groundwater samples should be considered. 4.11.1.9 Nearby Structures In cases where foundations are placed adjacent to existing structures, the influence of the existing structures on the behavior of the foundation, and the effect of the foundation on the existing structures, shall be investigated. 4.11.2 Notations = footing width (in length units) ,,Cw2 Df Dw & = reduced effective footing width (see Article 4.11.4.1.5) (in length units) = soil cohesion (in units of forceAength2) = correction factors for groundwater effect (dimensionless) = depth to footing base (in length units) = depth to groundwater table (in length units) = elastic modulus of rock masses (in units of force/length2) SPT-method CPT-method Load test Pile driving analyzer 0.70 L' = type of load = reduced effective length (see Article Li N = load type i = average value of standard penetration 1 4.11.4.1.5) (in length units) test blow count (dimensionless) N,, Nc,, N,, = modified bearing capacity factors used in analytic theory (dimensionless) = cone resistance (in units of forceAength2) ¶C = ultimate bearing capacity (in units of 9.a forceAength2) = reduction factor due to the effect of load RI inclination (dimensionless) R, = nominal resistance RQD = rock quality designation = span length (in length units) s = undrained shear strength of soil (in units S, of forceAength2) = load factor coefficient for load type i (see Pi Article 4.10.4) = load factor (see Article 4.10.4) Y = total (moist) unit weight of soil (see ArtiY cle 4.11.4.1.1) TABLE 4.10.6-3 Performance Factors for Geotechnical Strength L i t States in Axially Loaded Drilled Shafts Performance Factor Method/SoiCondition Ultimate bearing capacity of s i d e drilled shafts Side resistance in clay a-method (Reese & O'Neill) 0.65 Base resistance in clay Total Stress (Reese & O'Neill) 0.55 Side resistance in sand 1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill See discussion in article 4.13.3.3.3 Base resistance in sand 1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill See discussion in article 4.13.3.3.3 Side resistance in rock Carter & Kulhawy Horvath and Kemey 0.55 0.65 Canadian Geotechnical Society Pressuremeter Method (Canadian Geotechnical Society) 0.50 Load test 0.80 Clav 0.65 a-method (Reese & O'Neill) Belled Shafts (Reese & O'Neill) 0.55 -- - - Base resistance in rock Side resistance and end bearing Block failure Uplift capacity of single drilledI shafts Group uplift capacity Clay 0.50 0.50 Sand 1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill Rock Carter & Kulhawy Horvath & Kemey 0.45 0.55 Load test 0.80 Sand Clay See discussion in section 4.13.3.3.3 4.11.2 DMSION I-DESIGN 6 = differential settlement between adjacent 4) 4)f footings = performance factor = friction angle of soil 4.11.3 Movement Under Serviceability Limit States 4.11.3.1 General Movement of foundations in both vertical settlement and lateral displacement directions shallbe investigated at service limit states. Lateral displacement of a structure shall be evaluated when: -horizontal or inclined loads are present, -the foundation is placed on an embankment slope, -possibility of loss of foundation support through erosion or scour exists, or -bearing strata are significantly inclined. 4.11.3.2 Loads Immediate settlement shall be determined using the service load combinations given in Table 3.22.1A.
Timedependent settlement shall be determined using only the permanent loads. Settlement and horizontal movements caused by embankment loadings behind bridge abutments should be investigated. In seismically active areas, consideration shall be given to the potential settlement of footings on sand resulting from ground motions induced by earthquake loadings. For guidance in design, refer to Division I-A of these Specifications. 4.11.3.3 Movement Criteria Vertical and horizontal movement criteria for footings shall be developed consistent with the function and type of structure, anticipated service life, and consequences of unacceptable movements on structure performance. The tolerable movement criteria shall be established by empirical procedures or structural analyses. The maximum angular distortion (81s) between adjacent foundations shall be limited to 0.008 for simple span bridges and 0.004 for continuous span bridges.
These 81s limits shall not be applicable to rigid frame structures.
Rigid frames shall be designed for anticipated differential settlements based on the results of special analyses. 97 4.11.3.4 Settlement Analyses Foundation settlements shall be estimated using deformation analyses based on the results of laboratory or in situ testing. The soil parameters used in the analyses shall be chosen to reflect the loading history of the ground, the construction sequence and the effect of soil layering. Both total and differential settlements, including time effects, shall be considered. 4.11.3.4.1 Settlement of Footings on Cohesionless Soils Estimates of settlement of cohesionless soils shall make allowance for the fact that settlements in these soils can be highly erratic. No method should be considered capable of predicting settlements of footings on sand with precision. Settlements of footings on cohesionless soils may be estimated using empirical procedures or elastic theory.
4.11.3.4.2 Settlement of Footings on Cohesive Soils For foundations on cohesive soils, both immediate and consolidation settlements shall be investigated. If the footing width is small relative to the thickness of a compressible soil, the effect of three-dimensional loading shall be considered. In highly plastic and organic clay, secondary settlements are significant and shall be included in the analysis. 4.11.3.4.3 Settlements of Footings on Rock The magnitude of consolidation and secondary settlements in rock masses containing soft seams shall be estimated by applying procedures discussed in Article 4.11.3.4.2. 4.11.4 Safety Against Soil Failure 4.11.4.1 Bearing Capacity of Foundation Soils Several methods may be used to calculate ultimate bearing capacity of foundation soils. The calculated value of ultimate bearing capacity shall be multiplied by an appropriate performance factor, as given in Article 4.10.6, to determine the factored bearing capacity. Footings are considered to be adequate against soil failure if the factored bearing capacity exceeds the effect of design loads. 98 HIGHWAY BRIDGES 4.11.4.1.1 Theoretical Estimation The bearing capacity should be estimated using accepted soil mechanics theories based on measured soil parameters. The soil parameter used in the analysis shall be representative of the soil shear strength under the considered loading and subsurface conditions. 4.11.4.1.2 Semi-empirical Procedures The bearing capacity of foundation soils may be estimated from the results of in situ tests or by observing foundations on similar soils. The use of a particular in situ test and the interpretationof the results shall take local experience into consideration. The following in situ tests may be used: --Standard penetration test (SPT), --Cone penetration test (CPT), and -Pressuremeter test. 4.11.4.1.3 Plate Loading Test Bearing capacity may be determined by load tests providing that adequate subsurface explorations have been made to determine the soil profile below the foundation. The bearing capacity determined from a load test may be extrapolated to adjacent footings where the subsurface profile is similar. Plate load test shall be performed in accordance with the procedures specified in ASTM Standard D 1194-87 or AASHTO Standard T 235. 4.11.4.1.4 Presumptive Values Presumptive values for allowable bearing pressures on soil and rock, given in Table 4.11.4.1.4- 1, shall be used only for guidance, preliminary design or design of temporary structures. The use of presumptive values shall be based on the results of subsurface exploration to identify soil and rock conditions. All values used for design shall be confirmed by field andlor laboratory testing.
The values given in Table 4.11.4.1.4-1 are applicable directly for working stress procedures. When these values are used for preliminary design, all load factors shall be taken as unity. 4.11.4.1.1 sure that: (1) the product of the bearing capacity and an appropriate performance factor exceeds the effect of vertical design loads, and (2) eccentricity of loading, evaluated based on factored loads, is less than % of the footing dimension in any direction for footings on soils. For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact pressure distribution based on factored loads shall be used. 4.11.4.1.6 Effect of Groundwater Table Ultimate bearing capacity shall be determined based on the highest anticipated position of groundwater level at the footing location. In cases where the groundwater table is at a depth less than 1.5 times the footing width below the bottom of the footing, reduction of bearing capacity, as a result of submergence effects, shall be considered. 4.11.4.2 Bearing Capacity of Foundations on Rock The bearing capacity of footings on rock shall consider the presence, orientation and condition of discontinuities, weathering profiles and other similar profiles as they apply at a particular site, and the degree to which they shall be incorporated in the design. For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and RQD may be applicable. Competent rock shall be defined as a rock mass with discontinuities that are tight or open not wider than % inch. For footings on less competent rock, more detailed investigations and analyses shall be performed to account for the effects of weathering, and the presence and condition of discontinuities. Footings on rocks are considered to be adequate against bearing capacity failure if the product of the ultimate bearing capacity determined using procedures described in Articles 4.11.4.2.1 through 4.11.4.2.3 and an appropriate performance factor exceeds the effect of design loads. 4.11.4.2.1 Semi-empirical Procedures 4.11.4.1.5 Effect of Load Eccentricity For loads eccentric to the centroid of the footing, a reduced effective footing area (B' X L') shall be used in design. The reduced effective area is always concentrically loaded, so that the design bearing pressure on the reduced effective area is always uniform. Footings under eccentric loads shall be designed to en- Bearing capacity of foundations on rock may be determined using empirical correlation with RQD, or other systems for evaluating rock mass quality, such as the Geomechanic Rock Mass Rating (RMR) system, or Norwegian Geotechnical Institute (NGI) Rock Mass Classification System. The use of these semi-empirical procedures shall take local experience into consideration. 4.11.4.2.1 99 DIVISION I-DESIGN TABLE 4.11.4.1.4-1 Presumptive Allowable Bearing Pressures for Spread Footing Foundations Modified after U.S. Department of the Navy, 1982 Allowable Bearing Pressure (tsf) m e of Bearing Material Massive crystalline igneous and metamorphic rock: graphite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks) Foliated metamorphic rock: slate, schist (sound condition allows minor cracks) Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities Weathered or broken bedrock of any kind except highly argillacous rock (shale) Compaction shale or other highly argillacous rock in sound condition Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC) Gravel, gravel-sand mixtures, boulder-gravel mixtures (GW, GP, SW, SP) Coarse to medium sand, sand with little gravel (SW, SP) Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC) Find sand, silty or clayey medium to fine sand (SP, SM, SC) Homogeneous inorganic clay, sandy or silty clay (CL, CH) Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML,MH) Ordinary Range Recommended Value for Use Very hard, sound rock 60 to 100 80 Hard sound rock 30 to 40 35 Hard sound rock 15 to 25 20 Medium hard rock 8 to 12 10 Medium hard rock 8 to 12 10 Very dense 8 to 12 10 Very dense Medium dense to dense Loose Very dense Medium dense to dense Loose Very dense Medium dense to dense Loose Very stiff to hard Medium stiff to stiff Soft Very stiff to hard Medium stiff to stiff Soft 6 to 10 4 to 7 2 to 6 4 to 6 2 to 4 1 to 3 3 to 5 2 to 4 1 to 2 3 to 5 2 to 4 1 to 2 3 to 6 1to 3 0.5 to 1 2 to 4 1 to 3 0.5 to 1 7 5 3 4 3 1.5 3 2.5 1.5 3 2.5 1.5 Consistency in Place 4 2 0.5 3 1.5 0.5 100 HIGHWAY BRIDGES 4.1 1.4.2.2 Analytic Method The ultimate bearing capacity of foundations on rock shall be determined using established rock mechanics principles based on the rock mass strength parameters. The influence of discontinuities on the failure mode shall also be considered. 4.11.4.2.3 Load Test Where appropriate, load tests may be performed to determine the bearing capacity of foundations on rock. 4.11.4.2.4 Presumptive Bearing Values For simple structures on good quality rock masses, values of presumptive bearing pressure given in Table 4.11.4.2.4-1 may be used for preliminary design. The use of presumptive values shall be based on the results of subsurface exploration to identify rock conditions. All values used in design shall be confirmed by field andlor laboratory testing. The values given in Table 4.11.4.2.4-1 are directly applicable to working stress procedure, i.e., all the load factors shall be taken as unity. 4.11.4.2.5 Effect of Load Eccentricity If the eccentricity of loading on a footing is less than y.5 of the footing width, a trapezoidal bearing pressure shall be used in evaluating the bearing capacity.
If the eccentricity is between j/6 and f i of the footing width, a triangular bearing pressure shall be used. T i e maximum bearing pressure shall not exceed the product of the ultimate bearing capacity multiplied by a suitable performance factor. The eccentricity of loading evaluated using factored loads shall not exceed ?/s (37.5%) of the footing dimensions in any direction. 4.11.

characteristic length of soil-drilled shaft sys- z Z = depth below ground surface = total embedded length of drilled shaft 41 tem in cohesionless soils Greek a I3 = adhesion factor applied to S, = coefficient relating the vertical effective stress Y 6 = effective unit weight of soil = angle of shearing resistance between soil and rl = drilled shaft group efficiency factor and the unit skin friction of a drilled shaft drilled shaft base Pe Ptol 4 UV = settlement of the base of the drilled shaft = elastic shortening of drilled shaft = tolerable settlement = vertical effective stress = total vertical stress = working load at top of socket Bpi 4) = performance factor or 4)f = angle of internal friction of soil = performance factor for the total ultimate bear4% ing capacity of a drilled shaft = performance factor for the ultimate shaft ca4%~ pacity of a drilled shaft = performance factor for the ultimate tip capac$ 9 ~ ity of a drilled shaft +' 4.13.3 Geotechnical Design Drilled shafts shall be designed to have adequate bearing and structural capacities under tolerable settlements and tolerable lateral movements. The supporting capacity of drilled shafts shall be estimated by static analysis methods (analytical methods based on soil-structure interaction). Capacity may be verified with load test results. The method of construction may affect the drilled shaft capacity and shall be considered as part of the design process. Drilled shafts may be constructed using the dry, casing or wet method of construction,or a combination of methods. In every case, hole excavation, concrete placement, and all other aspects shall be performed in conformance with the provisions of this specification and Division II. 4.13.3.1 DIVISION I-DESIGN 4.13.3.1 Factors Affecting Axial Capacity See Article 4.6.5.2 for drilled shafts in soil and Article 4.6.5.3.3 for drilled shafts in rock. The following sub-articles shall supplement Articles 4.6.5.2 and 4.6.5.3.3. 107 4.13.3.2.3~ Settlement of Single Drilled Shafts The settlement of single drilled shafts shall be estimated considering short-term settlement, consolidation settlement (if constructed in cohesive soils), and axial compression of the drilled shaft. 4.13.3.2.3b Group Settlement 4.13.3.1.1 Downdrag Loads Downdrag loads shall be evaluated, where appropriate, as indicated in Article 4.12.3.1.3. 4.13.3.1.2 Uplift The provisions of Article 4.12.3.1.4 shall apply as applicable. Shafts designed for and constructed in expansive soil shall extend for a sufficient depth into moisture-stable soils to provide adequate anchorage to resist uplift. Sufficient clearance shall be provided between the ground surface and underside of caps or beams connecting shafts to preclude the application of uplift loads at the shaftlcap connection due to swelling ground conditions. Uplift capacity of straight-sided drilled shafts shall rely only on side resistance in conformance with Article 4.13.3.3.2 for drilled shafts in cohesive soils, and Article 4.13.3.3.3 for drilled shafts in cohesionless soils. If the shaft has an enlarged base, Q, shall be determined in conformance with Article 4.13.3.3.6. 4.13.3.2 Movement Under Serviceability Limit State 4.13.3.2.1 General The provisions of Article 4.12.3.2.1 shall apply as applicable. In estimating settlements of drilled shafts in clay, only unfactored permanent loads shall be considered. However unfactored live loads must be added to the permanent loads when estimating settlement of shafts in granular soil. 4.13.3.2.2 Tolerable Movement The provisions of Article 4.12.3.2.2 shall apply as applicable. The settlement of groups of drilled shafts shall be estimated using the same procedures as described for pile groups, Article 4.12.3.2.3. -Cohesive Soil, See Article 4.12.3.2.3a -Cohesionless Soil, See Article 4.12.3.2.3b 4.13.3.2.4 Lateral Displacement The provisions of Article 4.12.3.2.4 shall apply as applicable. 4.13.3.3 Resistance at Strength Limit States The strength limit states that must be considered include: (1) bearing capacity of drilled shafts, (2) uplift capacity of drilled shafts, and (3) punching of drilled shafts bearing in strong soil into a weaker layer below. 4.13.3.3.1 Axial Loading of Drilled Shafts The provisions of Article 4.12.3.3.1 shall apply as applicable.

4.13.3.3.2 Analytic Estimates of Drilled Shaji Capacity in Cohesive Soils Analytic (rational) methods may be used to estimate the ultimate bearing capacity of drilled shafts in cohesive soils. The performance factors for side resistance and tip resistance for three analytic methods shall be as provided in Table 4.10.6-3. If another analytic method is used, application of the performance factors in Table 4.10.6-3 may not be appropriate. 4.13.3.3.3 Estimation of Drilled-Shaft Capacity in Cohesionless Soils The ultimate bearing capacity of drilled shafts in cohesionless soils shall be estimated using applicable methods, and the factored capacity selected using judgment, and any available experience with similar conditions.

4.13.3.2.3 Settlement The settlement of a drilled shaft foundation involving either single drilled shafts or groups of drilled shafts shall not exceed the tolerable settlement as selected according to Article 4.13.3.2.2 4.13.3.3.4 Axial Capacity in Rock In determining the axial capacity of drilled shafts with rock sockets, the side resistance from overlying soil deposits shall be ignored. 108 HIGHWAY'BRIDGES If the rock is degradable, consideration of special construction procedures, larger socket dimensions, or reduced socket capacities shall be considered. The performance factors for drilled shafts socketed in rock shall be as provided in Table 4.10.6-3. 4.13.3.3.5 Load Test Where necessary, a full scale load test or tests shall be conducted on a drilled shaft or shafts to confirm response to load. Load tests shall be conducted using shafts constructed in a manner and of dimensions and materials identical to those planned for the production shafts. Load tests shall be conducted following prescribed written procedures which have been developed from accepted standards and modified, as appropriate, for the conditions at the site.

Standard pile load testing procedures developed by ASTM as specified in Article 4.12.3.3.5 may be modified for testing drilled shafts.

The performance factor for axial compressive capacity, axial uplift capacity, and lateral capacity obtained from load tests shall be as provided in Table 4.10.6-3. 4.13.3.3.6 Uplift Capacity Uplift shall be considered when (i) upward loads act on the drilled shafts and (ii) swelling or expansive soils act on the drilled shafts. Drilled shafts subjected to uplift forces shall be investigated, both for resistance to pullout and for their structural strength. 4.13.3.3.6~ Uplift Capacity of a Single Drilled Shaft The uplift capacity of a single straight-sided drilled shaft shall be estimated in a manner similar to that for estimating the ultimate side resistance for drilled shafts in compression (Articles 4.13.3.3.2, 4.13.3.3.3, and 4.13.3.3.4).

The uplift capacity of a belled shaft shall be estimated neglecting the side resistance above the bell, and assuming that the bell behaves as an anchor. The performance factor for the uplift capacity of drilled shafts shall be as provided in Table 4.10.6-3. 4.13.3.3.6b Group Uplift Capacity See Article 4.12.3.3.7b. The performance factors for uplift capacity of groups of drilled shafts shall be the same as those for pile groups as given in Table 4.10.6-3. 4.13.3.3.7 Lateral Load The design of laterally loaded drilled shafts is usually governed by lateral movement criteria (Article 4.13.3.2) 4.13.3.3.4 or structural failure of the drilled shaft.

The design of laterally loaded drilled shafts shall account for the effects of interaction between the shaft and ground, including the number of piers in the group. 4.13.3.3.8 Group Capacity Possible reduction in capacity from group effects shall be considered. 4.13.3.3.8~ Cohesive Soil The provisions of Article 4.12.3.3.10a shall apply. The performance factor for the group capacity of an equivalent pier or block failure shall be as provided in Table 4.10.62 for both cases of the cap being in contact, and not in contact with the ground. The performance factors for the group capacity calculated using the sum of the individual drilled shaft capacities are the same as those for the single drilled shaft capacities. 4.13.3.3.8b Cohesionless Soil Evaluation of group capacity of shafts in cohesionless soil shall consider the spacing between adjacent shafts. Regardless of cap contact with the ground, the individual capacity of each shaft shall be reduced by a factor q for an isolated shaft, where q = 0.67 for a center-to-center (CTC) spacing of three diameters and q = 1.0 for a center-to-center spacing of eight diameters. For intermediate spacings, the value of q may be determined by linear interpolation.

See Article 4.13.3.3.3 for a discussion on the selection of performance factors for drilled shaft capacities in cohesionless soils. 4.13.3.3.8~ Group in Strong Soil Overlying Weaker Compressible Soil The provisions of Article 4.12.3.3.10~shall apply as applicable.

4.13.3.3.9 Dynamic/Seismic Design Refer to Division I-A for guidance regarding the design of drilled shafts subjected to dynamic and seismic loads. 4.13.4 Structural Design The structural design of drilled shafts shall be in accordance with the provisions of Article 4.6.6, which was developed for allowable stress design proce- 4.13.4 DIVISION I-DESIGN dures. In order to use load factor design procedures for the structural design of drilled shafts, the load factor design procedures in Section 8 for reinforced concrete shall be used in place of the allowable stress design procedures. 4.13.4.1 109 Buckling of Drilled Shafts Stability of drilled shafts shall be considered when the shafts extend through water or air for a portion of their length. Section 5 RETAINING WALLS Part A GENERAL REQUIREMENTS AND MATERIALS 5.1 GENERAL Retaining walls shall be designed to withstand lateral earth and water pressures, including any live and dead load surcharge, the self weight of the wall, temperature and shrinkage effects, and earthquake loads in accordance with the general principles specified in this section. Retaining walls shall be designed for a service life based on consideration of the potential long-term effects of material deterioration, seepage, stray currents and other potentially deleterious environmental factors on each of the material components comprising the wall. For most applications, permanent retaining walls should be designed for a minimum service life of 75 years. Retaining walls for temporary applications are typically designed for a service life of 36 months or less. A greater level of safety andlor longer service life (i.e., 100 years) may be appropriate for walls which support bridge abutments, buildings, critical utilities, or other facilities for which the consequences of poor performance or failure would be severe.

The quality of in-service performance is an important consideration in the design of permanent retaining walls. Permanent walls shall be designed to retain an aesthetically pleasing appearance, and be essentially maintenance free throughout their design service life.

5.2 WALL TYPE AND BEHAVIOR Retaining walls are generally classified as gravity, semigravity, non-gravity cantilever, and anchored. Examples of various types of walls are provided in Figures 5.2A, 5.2B,and 5.2C.Gravity walls derive their capacity to resist lateral loads through a combination of dead weight and lateral resistance. Gravity walls can be further subdivided by type into rigid gravity walls, mechanically stabilized earth (MSE) walls, and prefabricated modular gravity walls. Semi-gravity walls are similar to gravity walls, ex- cept they rely more on structural resistance through cantilevering action, with this cantilevering action providing the means to mobilize dead weight for resistance; Nongravity cantilever walls rely strictly on the structural resistance of the wall and the passive resistance of the soil/rock, in which vertical wall elements are partially embedded in the soil/rock to provide fixity.

Anchored walls derive their capacity through cantilevering action of the vertical wall elements (similar to a non-gravity cantilever wall) and tensile capacity of anchors embedded in stable soil or rock below or behind potential soiUrock failure surfaces.

5.2.1 Selection of Wall Type Selection of wall type is based on an assessment of the magnitude and direction of loading, depth to suitable foundation support, potential for earthquake loading, presence of deleterious environmental factors, proximity of physical constraints, wall site cross-sectionalgeometry, tolerable and differential settlement, facing appearance, and ease and cost of construction. 5.2.1.1 Rigid Gravity and Semi-Gravity Walls Rigid gravity walls use the dead weight of the structure itself and may be constructed of stone masonry, unreinforced concrete, or reinforced concrete. Semi-gravity cantilever, counterfort, and buttress walls are constructed of reinforced concrete. Rigid gravity and semi-gravity retaining walls may be used for bridge substructures or grade separation. Rigid gravity and semi-gravity walls are generally used for permanent wall applications.These types of walls can be effective for both cut and fill wall applications due to their relatively narrow base widths which allows excavation laterally to be kept to a minimum. Gravity and semi-gravity walls may be used without deep foundation support only where the bearing soilhock is not prone to excessive or differential settlement. Due to their rigidity, excessive differential settlement can cause HIGHWAY BRIDGES 112 5.2.1.1 CI P CONCRETE OR SHOTCRETE F4Cl NG 7 F r C l NC PANELS MSE WALL VI TH HOOULAR PRECAST CONCRETE FACING PANELS MSE WALL WITH GEOSYNTHETI C RE1NFORCEMNT AN0 CI P CONCRETE OR SHOTCRETE FACING MSE waLL WITH S E G ~ N T A L CONCRETE BLOCK FACI NC FIGURE 5.2A Typical Mechanically Stabilized Earth Gravity Walls cracking, excessive bending or shear stresses in the wall, or rotation of the overall wall structure. 5.2.1.2 Nongravity Cantilevered Walls Nongravity cantilevered walls derive lateral resistance through embedment of vertical wall elements and support retained soil with facing elements. Vertical wall elements may consist of discrete vertical elements (e.g., soldier or sheet piles, caissons, or drilled shafts) spanned by a structural facing (e.g., wood or reinforced concrete lagging, precast or cast-in-place concrete panels, wire or fiber reinforced shotcrete, or metal elements such as sheet piles). The discrete vertical elements typically extend deeper into the ground than the facing to provide vertical and lateral support. Alternately, the vertical wall elements and facing are continuous and, therefore, also form the structural facing. Typical continuous vertical wall elements include piles, precast or cast-in-place concrete diaphragm wall panels, tangent piles, and tangent caissons. Permanent nongravity cantilevered walls may be constructed of reinforced concrete, timber, andlor metals. Temporary nongravity cantilevered walls may be constructed of reinforced concrete, metal and/or timber. Suitable metals generally include steel for components such as piles, brackets and plates, lagging and concrete reinforcement. Nongravity cantilevered walls may be used for the same applications as rigid gravity and semi-gravity walls, as well as temporary or permanent support of earth slopes, excavations, or unstable soil and rock masses. This type of wall requires little excavation behind the wall and is most effective in cut applications. They are also effective where deep foundation embedment is required for stability.

Nongravity cantilevered walls are generally limited to a maximum height of approximately 5 meters (15 feet), unless they are provided with additional support by means of anchors.

They generally cannot be used effectively where deep soft soils are present, as these walls depend on the passive resistance of the soil in front of the wall. 5.2.1.3 DIVISION I-DESIGN N T A L B I N WALL PRECAST CONCRETE B I N WALL 113 PRECAST CONCRETE CRIB WALL GAB1 ON WALL FIGURE 5.2B Typid Prefabricated Modular Gravity Walls 5.2.1.3 Anchored Walls Anchored walls are typically composed of the same elements as nongravity cantilevered walls (Article 5.2.1 .2), but derive additional lateral resistance from one or more tiers of anchors. Anchors may be prestressed or deadman type elements composed of strand tendons or bars (with corrosion protection as necessary) extending from the wall face to a ground zone or mechanical anchorage located beyond the zone of soil applying load to the wall. Bearing elements on the vertical support elements or facing of the wall transfer wall loads to the anchors. In some cases, a spread footing is used at the base of the anchored wall facing in lieu of vertical element embedment to provide vertical support. Due to their flexibility and method of support, the distribution of lateral pressure on anchored walls is influenced by the method and sequence of wall construction and anchor prestressing. Anchored walls are applicable for temporary and permanent support of stable and unstable soil and rock masses. Anchors are usually required for support of both temporary and permanent nongravity cantilevered walls higher than about 5 meters (15 feet), depending on soil conditions.

Anchored walls are typically constructed in cut situations, in which construction occurs from the top down to the base of the wall. Anchored walls have been successfully used to support fills; however, certain difficulties arising in fill wall applications require special consideration during design and construction. In particular, there is a potential for anchor damage due to settlement of backfill and underlying soils or due to improperly controlled backfilling procedures. Also, there is a potential for undesirable wall deflection if anchors are too highly stressed when the backfill is only partially complete and provides limited passive resistance. The base of the vertical wall elements should be located below any soft soils which are prone to settlement, as settlement of the vertical wall elements can cause destressing of the anchors.

Also, anchors should not be located within soft clays and silts, as it is difficult to obtain 114 5.2.1.3 HIGHWAY BRIDGES Mortar Rubble Ya8OnrY Riaid Cravlty wall Reinlorced Concrete Counterfort Semi-Cravlty Wall Reinlorced Concrete Cantilever Semi-Gravlty Wall Slurry or Cyllnder Pile Non-armvity Cantilever Wall NCHOR ZONE FIGURE 5.2C mica1 Rigid Gravity, Semi-Gravity Cantilever, Nongravity Cantilever, and Anchored Walls adequate long-term capacity in such materials due to creep. 5.2.1.4 Mechanically Stabilized Earth Walls MSE systems, whose elements may be proprietary, employ either metallic (strip or grid type) or geosynthetic (geotextile, strip, or geogrid) tensile reinforcements in the soil mass, and a facing element which is vertical or near vertical.

MSE walls behave as a gravity wall, deriving their lateral resistance through the dead weight of the re- inforced soil mass behind the facing. For relatively thick facings, such as segmental concrete block facings, the dead weight of the facing may also provide a significant contribution to the capacity of the wall system. MSE walls are typically used where conventional gravity, cantilever, or counterforted concrete retaining walls are considered, and are particularly well suited where substantial total and differential settlements are anticipated. The allowable settlement of MSE walls is limited by the longitudinal deformability of the facing and the performance requirements of the structure.

MSE walls have been successfully used in both fill and cut wall applications. However, they are most effective in fill wall applications. MSE walls shall not be used under the following conditions. lutants, other environmental conditions which are defined as aggressive as described in Division 11, Article 7.3.6.3, or where deicing spray is anticipated.

5.2.2 Wall Capacity When utilities other than highway drainage must be constructed within the reinforced zone if future access to the utilities would require that the reinforcement layers be cut, or if there is potential for material which can cause degradation of the soil reinforcement to leak out of the utilities into the wall backfill. With soil reinforcements exposed to surface or ground water contaminated by acid mine drainage, other industrial pollutants, or other environmental conditions which are defined as aggtessive as described in Division II, Article 7.3.6.3, unless environment specific long-term corrosion or degradation studies are conducted. When floodplain erosion may undermine the reinforced fill zone or facing column, or where the depth of scour cannot be reliably determined. Retaining walls shall be designed to provide adequate structural capacity with acceptable movements, adequate foundation bearing capacity with acceptable settlements, and acceptable overall stability of slopes adjacent to walls. The tolerable level of wall lateral and vertical deformations is controlled by the type and location of the wall structure and surrounding facilities. 5.2.2.1 Bearing Capacity The bearing capacity of wall foundation support systems shall be estimated using procedures described in Articles 4.4,4.5, or 4.6, or other generally accepted theories. Such theories are based on soil and rock parameters measured by in situ and/or laboratory tests. 5.2.2.2 Settlement MSE walls may be considered for use under the following special conditions: When two intersecting walls form an enclosed angle of 70" or less, the affected portion of the wall is designed as an internally tied bin structure with at-rest earth pressure coefficients. Where metallic reinforcements are used in areas of anticipated stray currents within 60 meters (200 feet) of the structure, a corrosion expert should evaluate the potential need for corrosion control requirements. 5.2.1.5 Prefabricated Modular Walls Prefabricated modular wall systems, whose elements may be proprietary, generally employ interlocking soilfilled reinforced concrete or steel modules or bins, rock filled gabion baskets, precast concrete units, or dry cast segmental masonry concrete units (without soil reinforcement) which resist earth pressures by acting as gravity retaining walls. Prefabricated modular walls may also use their structural elements to mobilize the dead weight of a portion of the wall backfill through soil arching to provide resistance to lateral loads. Prefabricated modular systems may be used where conventional gravity, cantilever or counterfort concrete retaining walls are considered.

Steel modular systems shall not be used where the steel will be exposed to surface or subsurface water which is contaminated by acid mine drainage, other industrial pol- The settlement of wall foundation support systems shall be estimated using procedures described in Articles 4.4,4.5, or 4.6, or other generally accepted methods.

Such methods are based on soil and rock parameters measured directly or inferred from the results of in situ and/or laboratory test. 5.2.23 Overall Stability The overall stability of slopes in the vicinity of walls shall be considered as part of the design of retaining walls.

The overall stability of the retaining wall, retained slope, and foundation soil or rock shall be evaluated for all walls using limiting equilibrium methods of analysis such as the Modified Bishop, simplified Janbu or Spencer methods of analysis. A minimum factor of safety of 1.3 shall be used for walls designed for static loads, except the factor of safety shall be 1.5 for walls that support abutments, buildings, critical utilities, or for other installations with a low tolerance for failure. A minimum factor of safety of 1.1 shall be used when designing walls for seismic loads. In all cases, the subsurface conditions and soillrock properties of the wall site shall be adequately characterized through in-sit- exploration and testing andlor laboratory testing as described in Article 5.3. Seismic forces applied to the mass of the slope shall be based on a horizontal seismic coefficient k, equal to onehalf the ground acceleration coefficient A, with the vertical seismic coefficient k, equal to zero. It must be noted that, even if overall stability is satisfactory, special exploration, testing and analyses may be required for bridge abutments or retaining walls constructed over soft subsoils where consolidation andlor lateral flow of the soft soil could result in unacceptable longterm settlements or horizontal movements. Stability of temporary construction slopes needed to construct the wall shall also be evaluated.

5.2.2.4 Tolerable Deformations Tolerable vertical and lateral deformation criteria for retaining walls shall be developed based on the function and type of wall, unanticipated service life, and consequencesof unacceptable movements (i.e., both structural and aesthetic).

Allowable total and differential vertical deformations for a particular retaining wall are dependent on the ability of the wall to deflect without causing damage to the wall elements or exhibiting unsightly deformations. The total and differential vertical deformation of a retaining wall should be small for rigid gravity and semi-gravity retaining walls, and for soldier pile walls with a cast-in-place facing.

For walls with anchors, any downward movement can cause significant destressing of the anchors. MSE walls can tolerate larger total and differential vertical deflections than rigid walls. The amount of total and differential vertical deflection that can be tolerated depends on the wall facing material, configuration, and timing of facing construction. A cast-in-place facing has the same vertical deformation limitations as the more rigid retaining wall systems. However, an MSE wall with a castin-place facing can be specified with a waiting period before the cast-in-place facing is constructed so that vertical (as well as horizontal) deformations have time to occur. An MSE wall with welded wire or geosynthetic facing can tolerate the most deformation. An MSE wall with multiple precast concrete panels cannot tolerate as much vertical deformation as flexible welded wire or geosynthetic facings because of potential damage to the precast panels and unsightly panel separation. Horizontal movements resulting from outward rotation of the wall or resulting from the development of internal equilibrium between the loads applied to the wall and the internal structure of the wall must be limited to prevent overstress of the structural wall facing and to prevent the wall face batter from becoming negative. In general, if vertical deformations are properly controlled, horizontal deformations will likely be within acceptable limits. For MSE walls with extensible reinforcements, reinforcement serviceability criteria, the wall face batter, and the facing type selected (i.e., the flexibility of the facing) will influence the horizontal deformation criteria required. Vertical wall movements shall be estimated using conventional settlement computational methods (see Articles 4.4,4.5, and 4.6. For gravity and semi-gravity walls, lateral movement results from a combination of differential vertical settlement between the heel and the toe of the wall and the rotation necessary to develop active earth pressure conditions (see Table 5.5.2A). If the wall is designed for at-rest earth pressure conditions, the deflections in Table 5.5.2A do not need to be considered. For anchored walls, deflections shall be estimated in accordance with Article 5.7.2. For MSE walls, deflections may be estimated in accordance with Article 5.8.10. Where a wall is used to support a structure, tolerable movement criteria shall be established in accordance with Articles 4.4.7.2.5.4.5 and 4.6. Where a wall supports soil on which an adjacent structure is founded, the effects of wall movements and associated backfill settlement on the adjacent structure shall be evaluated. For seismic design, seismic loads may be reduced, as result of lateral wall movement due to sliding, for what is calculated based on Division 1A using the MononobeOkabe method if both of the following conditions are met: the wall system and any structures supported by the wall can tolerate lateral movement resulting from sliding of the structure, the wall base is unrestrained regarding its ability to slide, other than soil friction along its base and minimal soil passive resistance. Procedures for accomplishing this reduction in seismic load are provided in the AASHTO LRFD Bridge Design Specifications,2nd Edition. In general, this only applies to gravity and semi-gravity walls.

Though the specifications in Division 1A regarding this issue are directed at structural gravity and semi-gravity walls, these specifications may also be applicable to other types of gravity walls regarding this issue provided the two conditions listed above are met. 5.23 Soil, Rock, and Other Problem Conditions Geologic and environmental conditions can influence the performance of retaining walls and their foundations, and may require special consideration during design. To the extent possible, the presence and influence of such conditions shall be evaluated as part of the subsurface exploration program. A representative, but not exclusive, listing of problem conditions requiring special consideration is presented in Table 4.2.3A for general guidance. 5.3 SUBSURFACE EXPLORATION AND TESTING PROGRAMS The elements of the subsurface exploration and testing programs shall be the responsibility of the Designer, based DIVISION I-DESIGN on the specific requirements of the project and his or her experience with local geological conditions.

5.3.1 General Requirements As a minimum, the subsurface exploration and testing programs shall define the following, where applicable: Soil strata: -Depth, thickness, and variability -Identification and classification -Relevant engineering properties (i.e., natural moisture content, Atterberg limits, shear strength, compressibility, stiffness, permeability, expansion or collapse potential, and frost susceptibility) -Relevant soil chemistry, including pH, resistivity, and sulfide content Rock strata: -Depth to rock -Identification and classification --Quality (i.e., soundness, hardness, jointing and presence of joint filling, resistance to weathering, if exposed, and solutioning) --Compressive strength (e.g., uniaxial compression, point load index) -Expansion potential Ground water elevation, including seasonal variations, chemical composition, and pH (especially important for anchored, non-gravity cantilevered, modular, and MSE walls) where corrosion potential is an important consideration Ground surface topography ' Local conditions requiring 'pecial consideration (e.g., presence of stray electrical currents). Exploration logs shall include soil and rock strata descriptions, penetration resistance for soils (e.g., SPT or q,), and sample recovery and RQD for rock strata. The drilling equipment and method, use of drilling mud, type of SPT hammer (i.e., safety, donut, hydraulic) or cone penetrometer (i.e., mechanical or electrical), and any unusual subsurface conditions such as artesian pressures, boulders or other obstructions, or voids shall also be noted on the exploration logs. local conditions. Where the wall is supported on deep foundations and for all non-gravity walls, the depth of the subsurface explorations shall extend a minimum of 6 meters (20 feet) below the anticipated pile, shaft, or sluny wall tip elevation. For piles or shafts end bearing on rock. or shafts extending into rock, a minimum of 3 meters (10 feet) of rock core, or a length of rock core equal to at least three times the shaft diameter, which ever is greater, shall be obtained to insure that the exploration has not been terminated on a boulder and to determine the physical characteristics of the rock within the zone of foundation influence for design. 5.3.3 Minimum Coverage A minimum of one soil boring shall be made for each retaining wall. For retaining walls over 30 meters (100 feet) in length, the spacing between borings should be 30 meters (100 feet). The number and spacing of the bore holes may be increased or decreased from 30 meters (100 feet), depending upon the anticipated geological conditions within the project area. In planning the exploration program, considerationshould be given to placing borings inboard and outboard of the wall line to define conditions in the scour zone at the toe of the wall and in the zone behind the wall to estimate lateral loads and anchorage or reinforcement capacities. 5.3.4 Laboratory Testing Laboratory testing shall be performed as necessary to deincluding unit weight, te-e engineering natural moisture content, Atterberg limits, gradation, shear strength, compressive strength and compressibility. In the absence of laboratory testing, engineering characteristics may be estimated based on field tests andlor published prope& correlations. Local experience should be applied when establishing project design values based on laboratory and field tests. 5.3.5 Scour The probable depth of scour shall be determined by subsurface exploration and hydraulic studies. Refer to Article 1.3.2 and FHWA (1991) for general guidance regarding hydraulic studies and design. 5.3.2 Minimum Depth 5.4 NOTATIONS Regardless of the wall foundation type, borings shall extend into a bearing layer adequate to support the anticipated foundation loads, defined as dense or hard soils, or bedrock. In general, for walls which do not utilize deep foundation support, subsurface explorations shall extend below the anticipated bearing level a minimum of twice the total wall height. Greater depths may be required where warranted by The following notations apply for design of retaining walls: A = Acceleration coefficient (dim); (See Article A, = Reinforcement area corrected for corrosion 5.8.9.1) losses (d); (See Article 5.8.6) 118 A, b bf B B' C Cf CR, CR,, C, d Di D* e, e' E, En ER f F* Fp F, F1 F2 HIGHWAY BRIDGES = Maximum wall acceleration coefficient at the centroid (dim); (See Article 5.8.9.1) = Width of discrete wall backfill element (m); (See Article 5.8.6) = Width of vertical or horizontal concentrated dead load (m); (See Article 5.8.12.1) = Total base width of wall, including facing elements (m); (See Article 5.5.5) = Effective base width of retaining wall foundation (m); (See Article 5.8.3) = Overall reinforcement surface area geometry factor (dim); (See Article 5.8.5.2) = Distance from back of wall facing to front edge of footing or other concentrated surcharge load (m); (See Article 5.8.12.1) = A reduction factor to account for reduced connection strength resulting from pullout of the connection (dim); (See Article 5.8.7.2) = A reduction factor to account for reduced ultimate strength resulting from rupture of the connection (dim); (See Article 5.8.7.2) = Soil coefficient of uniformity (dim); (See Article 5.8.5.2) = Distance from back of wall face to center of concentrated dead load (m); (See Article 5.8.12.1); also, the effective depth relative to stem of concrete semi-gravity walls for locating critical section for shear (m); (See Article 5.5.6.1) = Effective width of applied load at depth within or behind wall due to surcharge (m); (See Article 5.8.12.1) = Reinforcement bar diameter corrected for corrosion losses (mm); (See Article 5.8.6) = Eccentricity of forces contributing to bearing pressure (m); (See Articles 5.8.3 and 5.8.12.1) = Thickness of metal reinforcement at end of service life (mm); (See Article 5.8.6) = Nominal thickness of steel reinforcement at construction (mm); (See Article 5.8.6.1.1) = Equivalent sacrificed thickness of metal expected to be lost by uniform corrosion to produce expected loss of tensile strength during service life of structure (mm); (See Article 5.8.6.1.1) = Friction factor (dim); (See Article 5.5.2) = Pullout resistance factor (dim); (See Article5.8.5.2) = Lateral force resulting from &Anv (kN/m); (See Article 5.8.12.1) = Yield strength of the steel (kN/mm2); (See Article 5.8.6.1.1) = Active lateral earth pressure force for level backfill conditions (kN/m); (See Article 5.8.2) = Lateral earth pressure force due to traffic or other continuous surcharge (kN/m); (See Article 5.8.2) 5.4 Horizontal component of active lateral earth pressure force (kNIm); (See Article 5.8.2) FT = Resultant active lateral earth pressure force (kN/m); (See Article 5.8.2) FS = Factor of safety (dim); (See Article 5.5.5) FSoT = Factor of safety against overturning (dim); (See Article 5.8.2) FSpo = Safety factor against pullout (dim); (See Article 5.8.5.2) FSsL = Factor of safety against sliding (dim); (See Article 5.8.2) Fv = Vertical component of active lateral earth pressure force (kN/m); (See Article 5.8.2) G, = Distance to center of gravity of a modular block facing unit, including aggregatefill, measured from the front of the unit (m); (See Article 5.8.7.2) h = Equivalent height of soil representing surcharge pressure or effective total height of soil at back of reinforced soil mass (m); (See Article 5.8.2) = Vertical distance Fp is located from bottom of h, wall (m); (See Article 5.8.12.1) H = Design wall height (m); (See Article 5.8.1) H1 = Equivalent wall height (m); (See Article 5.8.5.1) Hz = Effective wall height (m); (See Article 5.8.9.1) Hh = Hinge height for block facings (m); (See Article 5.8.7.2) H, = Surcharge height (m of soil); (See Article 5.5.2) H, = Facing unit height (m); (See Article 5.8.7.2) H, = Height of water in backfill above base of wall (m) I =Average slope of broken back soil surcharge above wall (deg); (See Article 5.8.2) ib = Inclination of wall base from horizontal (deg); (See Article 5.8.7.2) kh = Horizontalseismic coefficient (dim); (See Article 5.8.9.1) kv = Vertical seismic coefficient (dim); (See Article 5.8.9.1) K = Earth pressure coefficient (dim); (See Article 5.5.2) Kae = Total Mononobe-Okabe seismic lateral earth pressure coefficient (dim); (See Article 5.8.9.1) AK, = Dynamic increment of the Mononobe-Okabe seismic lateral earth pressure coefficient (dim); (See Article 5.8.9.1) Kd = Active earth pressure coefficient for the soil behind the MSE wall reinforcements (dim); (See Article 5.8.2) K, = Lateral earth pressure coefficient for the soil within the MSE wall reinforced soil zone (dim); (See Article 5.8.4.1) Ka = Active earth pressure coefficient (dim); (See Article 5.5.2) K, = At-rest earth pressure coefficient (dim); (See Article 5.5.2) FH = DIVISION I. = Passive earth pressure coefficient for curved fail- ure surface (dim); (See Article 5.5.2) = Passive earth pressure coefficient for planar failure surface (dim); (See Article 5.5.2) = Depth from concentrated horizontal dead load location that force is distributed (m); (See Article 5.8.12.1) = Length of soil reinforcing elements (m); (See Article 5.8.2); length of structural footings (m); (See Article 5.8.12.1) La = Length of reinforcement in the active zone (m); (See Article 5.8.5.2) Le = Length of reinforcement in the resistant zone (m); (See Article 5.8.5.2) Lei = Effective reinforcement length for layer i (m); (See Article 5.8.9.2) m = Relative horizontal distance of point load from back of wall face (dim); (See Article 5.5.2) MA = The moment about point z at base of segmental concrete facing blocks due to force W, (mkNIm); (See Article 5.8.7.2) = The moment about point z at base of segmental concrete facing blocks due to force WE (mkN1m); (See Article 5.8.7.2) = Relative depth below top of wall when calculating lateral pressure due to point load above wall (dim); (See Article 5.5.2) = Number of reinforcement layers vertically within MSE wall (dim); (See Article 5.8.9.2) = Active earth pressure force (kNIm); (See Article 5.5.2) = Inertial force caused by seismic acceleration of the reinforced soil mass (kN/m); (See Article 5.8.9.1) = Inertial force caused by seismic acceleration of the sloping soil surcharge above the reinforced soil mass (kNlm); (See Article 5.8.9.1) = At-rest earth pressure force (kN/m); (See Article 5.5.2) = Earth pressure force resulting from uniform surcharge behind wall (W m) ; (See Article 5.5.2) = Dynamic horizontal thrust due to seismic loading (kNlm); (See Article 5.8.9.1) = Concentrated horizontal dead load force (kN/m); (See Articles 5.5.2 and 5.8.12.1) = Inertial force of mass within active zone due to seismic loading (kN/m); (See Article 5.8.9.2) = Reinforced wall mass inertial force due to seismic loading (kNIm); (See Article 5.8.9.1) = Resultant horizontal load on wall due to point load (kN/m), (See Article 5.5.2) = Concentrated vertical dead load force for strip load (kN/m); (See Article 5.8.12.1) = Concentrated vertical dead load force for isolated footing or point load (kN/m); (See Article 5.8.12.1) P, = Force due to hydrostatic water pressure behind wall (kN/m); (See Article 5.5.3) live load pressure (kN/m2); (See Article q 5.8.2) = Cone end bearing resistance (kN/m2), (See Artiq, cle 5.3.1) = Line load force (kN/m); (See Article 5.5.2) Qp = Point load force (kN); (See Article 5.5.2) = Resultant of foundation bearing pressure (kN or R kN/m); (See Article 5.8.3) R' = Distance above wall base to resultant of lateral pressure due to surcharge (m); (See Article 5.5.2) R, = Soil reinforcement coverage ratio (dim); (SeeArticle 5.8.6) RF = Reduction factor applied to the ultimate tensile strength to account for short and long-term degradation factors such as installation damage, creep, and chemical aging (dim); (See Article 5.8.6.1.2) RF, = Reduction factor applied to the ultimate tensile reinforcement-facingconnection strength to account for long-term degradation factors such as creep and chemical aging(dim);(See Article 5.8.7.2) RF, = Reinforcement strength reduction factor to account for installation damage (dim); (See Article 5.8.6.1.2) RFcR= Reinforcement strength reduction factor to account for creep rupture (dim); (See Article 5.8.6.1.2) = Reinforcement strength reduction factor to account for rupture due to chemical/biological degradation (dim); (See Article 5.8.6.1.2) = Equivalent soil surcharge height above wall (m); S (See Article 5.8.4.1) Sh = Horizontal reinforcement spacing of discrete reinforcements (mm);(See Article 5.8.6) S, = The reinforcement strength needed to resist the static component of load (kN/m); (See Article 5.8.9.2) S, = The reinforcement strength needed to resist the dynamic or transient component of load (kNIm); (See Article 5.8.9.2) St = Transverse grid element spacing (mm); (See Article 5.8.5.2) = Vertical spacing of soil reinforcement (mm); (See S, Article 5.8.4.1) = Transverse grid or bar mat element thickness t (mm);(See Article 5.8.5.2) T = Total load applied to structural frame around obstruction (kN); (See Article 5.8.12.4) T, = The allowable load which can be applied to each soil reinforcement layer per unit width of reinforcement (kN/m); (See Article 5.8.6) T, = The allowable load which can be applied to each soil reinforcement layer per unit width of rein= Traffic 120 HIGHWAY BRIDGES forcement at the connection with the wall face (W m) ; (See Article 5.8.7.2) T, = Maximum load applied to each soil reinforcement layer per unit width of wall (kN/m); (See Article 5.8.4.1) = Allowable long-term reinforcement tension per Td unit reinforcement width for ultimate limit state (kN/m); (See Article 5.8.6.1.2) Tlot = The ultimate wide width tensile strength for the reinforcement material lot used for connection strength testing (W m) ; (See Article 5.8.7.2) Tmd = Incremental dynamic inertia force at level i (kN/m of structure); (See Article 5.8.9.2) To = Applied reinforcement load per unit width of wall at the connection with the facing (kNm); (See Article 5.8.4.2) T,, = The peak load per unit reinforcement width in the connection test at a specified confining pressure where reinforcement pullout is known to be the mode of failure W m) ; (See Article 5.8.7.2) Tmd = The total static plus seismic load applied to each reinforcement layer per unit width of wall (KNJm); (See Article 5.8.9.2) = Ultimate tensile strength of geosynthetic reinTdt forcement per unit rein

Modified after Teng (1962) COHESIVE SO1 L (&.'I NOTE FOR SLOPING BACKFILL USE EFFECTIVE SHEAR STRENGrti PARAMETERS (C - 0) AND FIGURE 5.6.2.C.o FINISHED GRADE a EMBEDMENT IN COHESIVE SOIL RETAINING GRANULAR SOIL b EM0EDMENT I N COHESIVE SOIL RETAINING COHESIVE SOIL NOTE1 REFER TO TABLE 5.6.2A FOR GENERAL NOTES AND LEGEND FIGURE 5.6.2C Simplified Earth Pressure Distributions for Temporary Flexible Cantilevered Walls with Discrete Vertical Wall Elements 5.6.2 DIVISION I-DESIGN 131 -- GRANULAR \ SOIL I [~su, i* COHESIVE SOIL 2 COHESIVE SOIL (7; 1 su21 a. EMBEDMENT IN COHESIVE SOIL RETAINING GRANULAR SOIL b. EMBEDMENT IN COHESIVE SOIL RETAINING COHESIVE SOlL FIGURE 5.6.2D Simplified Earth Pressure Distributions for Temporary Flexible Cantilevered Walls with Continuous Vertical Wall Elements Modified after Teng (1962) TABLE 5.6.2A General Notes and Legend Simplified Earth Pressure Distributions for Permanent and Temporary Flexible Cantilevered Walls with Discrete Vertical Wall Elements LEGEND: 7' b = Effective unit weight of soil 4 = Spacing between vertical wall elements (clc) S, s = Undrained shear strength of cohesive soil = Vertical element width = Shear strength of rock mass P, Pa = Passive resistance per vertical wall element p p' = Ground surface slope behind wall = Active earth pressure per vertical wall element = Ground surface slope in front of wall + for slope up from wall - for slope down from wall Ka = Active earth pressure coefficient; Refer to Figure 5.5.2A K, = Passive earth pressure coefficient; Refer to Figures 5.5.2C and 5.5.2D ' = Effective angle of soil friction Nmm (1) For temporary walls embedded in granular soil or rock, refer to Figure 5.6.2A to determine passive resistance and use diagrams on Figure 5.6.2C to determine active earth pressure of retained soil. (2) Surcharge and water pressures must be added to the indicated earth pressures. (3) Forces shown are per vertical wall element. (4) Pressure distributions below the exposed portion of the wall are based on an effective element width of 3b, which is valid for 2 5b. For < 5b, refer to Figures 5.6.2B and 5.6.2D for continuous wall elements to determine pressure distributions on embedded portions of the wall. 132 HIGHWAY BRIDGES Figures 5.6.2C and 5.6.2D, or other suitable earth pressure distributions, may be used with the following restrictions: The ratio of overburden pressure to undrained shear strength (i.e., stability number N = yWc) must be < 3. The active earth pressure shall not be less than 0.25 times the effective overburden pressure at any depth. 5.6.2 freezing and expansion. In such cases, insulation shall be provided on the walls to prevent freezing of the soil, or consideration should be given during wall design to the pressures which may be exerted on the wall by frozen soil. 5.6.4 Seismic Pressure Refer to Section 6 of Division I-A for guidance regarding the design of flexible cantilevered walls subjected to dynamic and seismic loads. In general, the pseudostatic approach developed by Mononobe-Okabe may be used to estimate the equivalent static forces. Forces resulting from wall inertia effects may be ignored in estimating the seismic lateral earth pressure. Where discrete vertical wall elements are used for support, the width of each vertical element shall be assumed to equal the width of the flange or diameter of the element for driven sections and the diameter of the concrete-filled hole for sections encased in concrete. The magnitude and location of resultant loads and resisting forces for permanent walls with discrete vertical elements embedded in soil and rock for lateral support may be determined using the earth pressure distributions presented in Figures 5.6.2A and 5.6.2C, or other earth pressure distributions developed for use in the design of such walls.

The procedure for determining the resultant passive resistance of a vertical element embedded in soil assumes that net passive resistance is mobilized across a maximum of three times the element width or diameter (reduced, if necessary, to account for soft clay or discontinuitiesin the embedded depth of soil or rock) and that some portion of the embedded depth below finished grade (usually 2 to 3 feet for an element in soil, and 1 foot for an element in rock) is ineffective in providing passive lateral support. In developing the design lateral pressure, the lateral pressure due to traffic, permanent point and line surcharge loads, backfill compaction, or other types of surcharge loads shall be added to the lateral earth pressure in accordance with Articles 3.20.3 and 5.5.2. Flexible cantilevered walls shall be dimensioned to ensure stability against passive failure of embedded vertical elements such that FS 1 1.5. Unfactored dead and live loads shall be used to evaluate the factor of safety against passive failure of embedded vertical elements. Vertical elements shall be designed to support the full design earth, surcharge and water pressures between the elements. In determining the depth of embedment to mobilize passive resistance, consideration shall be given to planes of weakness (e.g., slickensides, bedding planes, and joint sets) that could reduce the strength of the soil or rock determined by field or laboratory tests. Embedment in intact rock, including massive to appreciably jointed rock which should not fail through a joint surface, should be based on an allowable shear strength of O.lOCo to 0. 15C0of the intact rock. 5.6.3 Water Pressure and Drainage 5.6.6 Structure Design Flexible cantilevered walls shall be designed to resist the maximum anticipated water pressure. For a horizontal static ground water table, the total hydrostatic water pressure shall be determined using Equation 5.5.3-1. For differing ground water levels on opposite sides of the wall, the water pressure and seepage forces shall be determined by flow net procedures or other appropriate methods of analysis, where necessary. Seepage shall be controlled by installation of a drainage medium (e.g., preformed drainage panels, sand or gravel drains or wick drains) behind the facing with outlets at or near the base of the wall. Drainage panels shall maintain their drainage characteristics under the design earth pressures and surcharge loadings, and shall extend from the base of the wall to a level 1 foot below the top of the wall.

Where thin drainage panels are used behind walls, saturated or moist soil behind the panels may be subject to Structural design of individual wall elements may be performed by service load or load factor design methods in conformance with Article 3.22. The maximum spacing between vertical supporting elements depends on the relative stiffness of the vertical elements and facing, and the type and condition of soil to be supported. M, in a 1-foot height of wall facing at any level may be determined by the following, or other acceptable design procedures: 5.6.5 Structure Dimensions and External Stability Simple span (no soil arching) Simple span (soil arching) M,, = pat2/12 5.6.6 DIVISION I-DESIGN Continuous (no soil arching) 133 sidered adequate with respect to the decay hazard and expected service life of the structure. 5.6.7 Overall Stability Continuous (soil arching) Refer to Article 5.2.2.3. Equation 5.6.6-1 is applicable for simply supported facing behind which the soil will not arch between vertical supports (e.g., in soft cohesive soils or for rigid concrete facing placed tightly against the in-place soil). Equation 5.6.6-2 is applicable for simply supported facing behind which the soil will arch between vertical supports (e.g., in granular or stiff cohesive soils with flexible facing or rigid facing behind which there is sufficient space to permit the in-place soil to arch). Equations 5.6.6-3 and 5.6.6-4 are applicable for facing which is continuous over several vertical supports (e.g., reinforced shotcrete or concrete). Timber facings should be constructed of stress-grade lumber in conformance with Article 13.2.1. If timber is used where conditions are favorable for the growth of decay-producing organisms, wood should be pressure treated with a wood preservative unless the heartwood of a naturally decay-resistant species is available and is con- 5.6.8 Corrosion Protection Refer to Article 5.7.8. 5.7 ANCHORED WALL DESIGN 5.7.1 Design Terminology Refer to Figure 5.7.1A for terminology used for the design of anchored retaining walls. 5.7.2 Earth Pressure and Surcharge Loadings The development of lateral earth pressures for design shall consider the method and sequence of construction, the rigidity of the walVanchor system, the physical characteristics and stability of the ground mass to be sup- ANCHOR HEAD ANCHOR INCLINATION AS REQUIRED PRIMARY GROUT FIGURE 5.7.1A w i c a l Terms Used in Flexible Anchored WaU Design 134 HIGHWAY BRIDGES SO1L TYPE APPARENT EARTH PRESSURE DISTRIBUTION 5.7.2 NOTATION - H FINAL WALL HEIGHT Ka- ACTIVE EARTH PRESSURE COEFFlClENT 7'- EFFECTIVE SOlL UNIT WEIGHT SAND (4) (OR PERMANENT WALLS IN CLAY) - y TOTAL SOlL UNIT WEIGHT m REDUCTION FACTOR q, UNCONFINED COMPRESSIVE STRENGTH - -IE- 0.25 H SOFT TO (4) MEDIUM CLAY (q, KaYH m 8 0 . 4 FOR NORMALLY CONSOLIDATED CLAY (21 4 (3) VALUE OF 0.4 SHOUU) BE USED A e 4 0.50 H a 4 -fl CLAYS m LESS :I FORTHAN OVERCONSOLIDATED 0.25 0.25 H STIFF TO (4) HARD CLAY K ~ TAN^ = (45 12' K a = I - m L2qUIyH) BOTNOT 0.75H 0.25 fO 1.0 TSF) "' NOTES v 0.25 H FOR LONG-TERM EXCAVATIONS ; VALUES BETWEEN 0.4 AND 0.2 MAY BE USED FOR SHORT-TERM CONDITIONS.

(4) SURCHARGE AND WATER PRESSURES MUST BE ADDED TO THESE EARTH PRESSURE DIAGRAMS. THE TWO LOWER DIAGRAMS ARE NOT VALID FOR PERMANENT WALLS OR WALLS WHERE WATER LEVEL LIES ABOVE BOTTOM OF EXCAVATION.

FIGURE 5.7.2A Guidelines for Estimating Earth Pressure on Walls with %o or More Levels of Anchors Constructed from the Top Down Modified after Tenaghi and Peck (1967) ported, allowable wall deflections, the space between anchors, anchor prestress, and the potential for anchor yield. For stable ground masses, the final distribution and magnitude of lateral earth pressure on a completed anchored wall with two or more levels of anchors constructed from the top down may be computed using the apparent earth pressure distributions shown in Figure 5.7.2A or any other applicable earth pressure distribution developed for this purpose. For unstable or marginally stable ground masses, the design earth pressure may exceed those shown in Figure 5.7.2A and loads should be estimated using methods of slope stability analysis which incorporate the effects of anchors or which consider interslice equilibrium and provide information on interslice forces. In developing the design earth pressure for a particular wall section, consideration shall be given to wall displacements that may affect adjacent structures or underground utilities. Very approximate estimates of settlements adjacent to braced or anchored flexible walls can be made using Figure 5.7.2B. If wall deflections estimated using Figure 5.7.2B are excessive for a particular application, a more detailed analysis using beam on elastic foundation, finite element or other methods of analysis which consider the soil-structure interaction effects of anchored walls may be warranted. 5.7.2 DIVISION I-DESIGN 135 DISTANCE FROM EXCAVATION DEPM OF EXCAVATION CURVE I = Sand If = CURVE IIt = CURVE CURVE Z !! Stiff to very hard clay Soft to medium clay, factor of Safety against basal heave Equal to 2.0 = Soft to medium clay, factor of Safety against basal heave Equal to 1.2 (, yu) H + q FIGURE 5.7.2B Settlement Profiles Behind Braced or Anchored Walls Modified after Clough and O'Rourke (1990) Anchored walls with one level of anchors may be designed using a triangular earth pressure distribution in accordance with Article 5.6.2 or using another suitable earth pressure distribution consistent with the expected wall deflection. For the case where excavation has advanced down to the first anchor level but the first row of anchors has not yet been installed, the wall shall be treated as a nongravity cantilevered wall and the earth pressure distribution loading on the wall shall be assumed as triangular in accordance with Article 5.6.2. Overstressing of the anchors should be avoided as excessive anchor loads relative to the capacity of the retained ground mass to support the anchor loads can result in undesirable deflections, or passive failure of the wall into the retained soil. In developing the design lateral pressure for walls constructed from the top down, the lateral pressure due to traffic or other surcharge loading, shall be added to the lateral earth pressure in accordance with Articles 3.23.3 and 5.5.2, using an earth pressure coefficient consistent with the estimated magnitude of wall deflection. For the conditions where there is no or one anchor level, the magnitude and distribution of lateral resisting forces for embedded vertical elements in soil or rock shall be determined following procedures described in Article 5.6.2. When two or more levels of anchors have been installed, the magnitude of lateral resistance provided by embedded vertical elements will depend on the element stiffness and deflection under load. The earth pressures on anchored walls constructed in fill situations from the bottom up are affected by the method and sequence of construction.

Therefore, the method and sequence of construction must be considered when selecting appropriate lateral earth pressures for anchored walls in fill situations. As a general guide, the following may be considered: For walls with a single anchor level-A triangular distribution defined by Kay per unit length of wall height plus surcharge loads. 136 HIGHWAY BRIDGES For walls with multiple anchor levels-A rectangular pressure distribution derived by increasing the total force from the triangular pressure distribution described above by one-third and applying the force as a uniform pressure distribution. 5.7.3 Water Pressure and Drainage 5.7.2 Refer to Article 5.7.2 for general guidance regarding wall deflections. 5.7.6 Structure Design Depending on the characteristics of the wall, the wall components shall be designed by service load or load factor methods in conformance with Article 3.22. Refer to Article 5.6.3. 5.7.6.1 General 5.7.4 Seismic Pressure Refer to Section 6 of Division I-A for guidance regarding the design of anchored retaining walls subjected to dynamic and seismic loads. In general, the pseudo-static approach developed by Mononobe-Okabe may be used to estimate the equivalent static forces provided the maximum lateral earth pressure be computed using a seismic coefficient kh= 1.5A. Forces resulting from wall inertia effects may be ignored in estimating the seismic lateral earth pressure. 5.7.5 Structure Dimensions and External Stability The design of anchored walls includes determination of the following: Size, spacing, and depth of embedment of vertical wall elements and facing; v p e , capacity, spacing, depth, inclination and corrosion protection of anchors; and Structural capacity and stability of the wall, wall foundation, and surrounding soil mass for all intermediate and final stages of construction. The bearing capacity and settlement of vertical wall elements under the vertical component of the anchor forces and other vertical loads shall be determined in accordance with Articles 4.4,4.5, or 4.6. For walls supported in or through soft clays with S, < 0.3yrH, continuous vertical elements extending well below the exposed base of the wall may be required to prevent heave in front of the wall. Otherwise, the vertical elements are embedded several feet as required for stability or end bearing. (Where significant embedment of the wall is required to prevent bottom heave, the lowest section of wall below the lowest row of anchors must be designed to resist the moment induced by the pressure acting between the lowest row of anchors and the base of the exposed wall, and the force Pb = 0.7(yHBe - 1.4cH - PCB,) acting at the midheight of the embedded depth of the wall.) The required embedment depth @ or Do) may be determined in accordance with Article 5.6.2. The procedure for anchored wall design depends on the number of anchor rows and the construction sequence. For a typical wall with two or more rows of anchors constructed from the top down, the procedure requires design for the final structure with multiple rows of anchors and checking the design for the various stages of wall construction. The required horizontal component of each anchor force shall be computed using the apparent earth pressure distributions in Figure 5.7.2A, or other applicable earth pressure distributions, and any other horizontal water pressure, surcharge or seismic forces acting on the wall. The total anchor force shall be determined based on the anchor inclination. The horizontal anchor spacing and anchor capacity shall be selected to provide the required total anchor force.

The vertical wall elements shall be designed to resist all horizontal earth pressure, surcharge, water pressure, anchor and seismic loadings as well as the vertical component of the anchor loads and any other vertical loads. Supports may be assumed at each anchor location and at the bottom of the wall if the vertical element is extended below the bottom of the wall. The stresses in and the design of the wall facing shall be computed in accordance with the requirements of Article 5.6.6. All components of the anchored wall system shall be checked for the various earth pressure distributions and other loading conditions which will exist during the course of construction. 5.7.6.2 Anchor Design Anchor design shall include an evaluation of the feasibility of using anchors, selection of an anchor system, estimates of anchor capacity, determination of unbonded length, and determination of corrosion protection requirements. In determining the feasibility of employing anchors at a particular location, consideration shall be given to the availability or ability to obtain underground easements, proximity of buried facilities to anchor locations, and the suitability of subsurface soil and rock conditions within the anchor stressing zone.

5.7.6.2 DIVISION I-DESIGN TABLE 5.7.6.2A 137 Presumptive Ultimate Values of Load Transfer for Preliminary Design of Anchors in Soil Modified after Cheney (1982) Relative Density1 Consistency(') Soil Type Estimated Ultimate ~ransferLoad (kipstlineal foot) Loose Medium dense Dense Loose Medium dense Dense Loose Medium dense Dense Stiff Hard Sand and Gravel Sand and Silt Silt-clay M,ixturewith Minimum LL, PI, and LI Restrictions, or ~ i n~ei c a c e o u s (~) ~or a nSilt d Uixtures (''Values corrected for overburden pressure. ("The presence of mica tends to increase the volume and compressibility of sand and soft deposits due to bridging action and subsequent flexibility under increased pressures. TABLE 5.7.6.2B Presumptive Ultimate Values of Load Transfer for Preliminary Design of Anchors in Rock Modified after Cheney (1982) Rock Type Granite or Basalt Dolomitic Limestone Soft Limestone Sandstone Slates and Hard Shales Soft Shales Estimated Ultimate Transfer Load (kipsllineal foot) 50 40 30 30 25 10 The required anchor forces shall be determined in accordance with Article 5.7.6.1. The ultimate anchor capacity per unit length may be preliminarily estimated using the guidelines presented in Tables 5.7.6.2A and 5.7.6.2B for soil and rock, respectively. These guidelines are for preliminary design of straight shaft anchors installed in small diameter holes using a low grout pressure. Other anchor types and installation procedures could provide other estimated ultimate anchor capacities. Final determination of the anchor capacity and required bond length shall be the responsibility of the anchored wall specialty contractor. The allowable anchor capacity for small diameter anchors may be estimated by multiplying the ultimate anchor capacity per unit length times the bonded (or stressing) length and dividing by a FS of 2.5 for anchors in soil and 3.0 for anchors in rock. Bearing elements for anchors shall be designed to maintain shear stresses in the vertical wall elements and facing within allowable values.

The capacity of each anchor shall be verified as part of a stressing and testing program. (See Division 11.) Determination of the unbonded anchor length shall consider the location of the critical failure surface farthest from the wall, the minimum length required to insure minimal loss of anchor prestress due to long-term ground movements, and the depth to adequate anchoring strata. As shown in Figure 5.7.1A, the unbonded (or free) anchor length should not be less than 15 feet and should extend beyond the critical failure surface in the soil mass being retained by the wall. For granular soils or drained cohesive soils, the critical failure surface is typically assumed to be the active failure wedge which is defined by a plane extending upward from the base of the wall at an angle of 45 +'I2 from the horizontal.Longer free lengths may be required for anchors in plastic soils or where critical failure surfaces are defined by planes or discontinuities with other orientations. Selection of an anchor inclination shall consider the location of suitable soil or rock strata, the presence of buried utilities or other geometric constraints, and constructability of the anchor drill holes. The component of vertical load resultingfrom anchor inclination shall be included in evaluating the end bearing and settlement of vertical wall elements. The minimum horizontal spacing of anchors should be either three times the diameter of the bonded zone or 4 feet, whichever is larger. If smaller spacings are required, + 138 HIGHWAY BRIDGES consideration can be given to differing anchor inclinations between alternating anchors. 5.7.7 Overall Stability Refer to Article 5.2.2.3. 5.7.8 Corrosion Protection Prestressed anchors and anchor heads shall be protected against corrosion consistent with the ground and ground water conditions at the site. The level and extent of corrosion protection shall be a function of the ground environment and the potential consequences of an anchor failure. Corrosion protection shall be applied in accordance with Section 6 of Division 11--Ground Anchors. 5.7.9 Anchor Load Testing and Stressing All anchors shall be tested in accordance with Section 6 of Division 11--Ground Anchors, Article 6.5.5, Testing and Stressing. 5.8 MECHANICALLY STABILIZED EARTH (MSE) WALL DESIGN MSE walls shall be designed for external stability of the wall system as well as internal stability of the reinforced soil mass behind the facing. Internal design of MSE wall systems requires knowledge of short and longterm properties of the materials used as soil reinforcements as well as the soil mechanics which govern MSE wall behavior. Structural design of the wall facing may also be required. The specifications provided herein for MSE walls do not apply to geometrically complex MSE wall systems such as tiered walls (walls stacked on top of one another), back-to-back walls, or walls which have trapezoidal sections.

Design guidelines for these cases are provided in FMWA publication No. FHWA SA-96-071"Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines." Compound stability should also be evaluated for these complex MSE wall systems (see Article 5.8.2). 5.8.1 Structure Dimensions An illustration of the MSE wall element dimensions needed for design is provided in Figure 5.8.1A. MSE walls shall be dimensioned to ensure that the minimum factors of safety required by Article 5.5.5 for sliding and overturning stability are satisfied. In addition, the minimum factors of safety provided in Article 5.8 for foundation bearing capacity (5.8.3) and pullout resistance 5.7.6.2 (5.8.5.2) shall also be satisfied, as well as overall stability requirements as provided in Article 5.2.2.3. The soil reinforcement length shall be calculated based on external and internal stability considerations in accordance with Articles 5.2.2.3 and 5.5.5, and all relevant portions of Article 5.8. Soil reinforcement length shall be as a minimum approximately 70% of the wall height (as measured from the leveling pad) and not less than 2.4 meters (8 feet). The wall height is defined as the difference in elevation between the top of the wall at the wall face (i.e., where the finished grade intersects the back of the wall face) and the top of the leveling pad. The reinforcement length shall be uniform throughout the entire height of the wall, unless substantiating evidence is presented to indicate that variation in length is satisfactory. External loads such as surcharges will increase the minimum reinforcement length.

Greater reinforcementlengths may also be required for very soft soil sites and to satisfy global stability requirements. The minimum embedment depth of the bottom of the reinforced soil mass, which is the same as the top of the leveling pad, shall be based on bearing capacity, settlement, and stability requirements determined in accordance with Articles 5.2.2.1, 5.2.2.2 and 5.2.2.3, and pertinent portions of Article 5.8, including the effects of frost heave, scour, proximity to slopes, erosion, and the potential future excavation in front of the wall. The lowest backfill reinforcement layer shall not be located above the long-term ground surface in front of the wall. As an alternative to being below the depth of frost penetration, the soil below the wall but above the depth of frost penetration can be removed and replaced with nonfrost susceptible clean granular soil. In addition to general bearing capacity, settlement, and stability considerations, the minimum embedment required shall consider the potential for local bearing capacity failure under the leveling pad or footing due to higher vertical stresses transmitted by the facing. A minimum horizontal bench 1.2 meters (4 feet) wide shall be provided in front of walls founded on slopes. For walls constructed along rivers and streams, embedment depths shall be established at a minimum of 0.6 meters (2 feet) below potential scour depth as determined in accordance with Article 5.3.5. 5.8.2 External Stability Stability computations shall be made by assuming the reinforced soil mass and facing to be a rigid body. The coefficient of active earth pressure, Kd used to compute the horizontal force resulting from the retained backfill behind the reinforced zone and other loads shall be computed on the basis of the friction angle of the retained backfill. In the absence of specific data, a maximum friction angle of 30" should be used. The limitation also applies when determining the coefficient of sliding friction 5.8.2 DIVISION I-DESIGN 139 For externol end rnternsl steblltty colculot~ons, the welght end dlmenstons of the facrng elements ore qjprcolly rgnored. However. r t 1s acceptable t o tnclude the facing d~mons~ons and werght rn slrding, overturning, and beorrng capacity c a l c u l o t ~ m s .For I n t e r n a l stabill colculot~ons. the wall dimensrons ore considered t o begrn s t the back of the facing e ements. ?' FIGURE 5.8.1A MSE Wall Element Dimensions Needed for Design at the wall base. Passive pressures shall be neglected in stability computations. The active earth pressure coefficients for retained backfill (i.e., fill behind the reinforced soil mass) for external stability calculations only is computed as shown in Figure 5.5.2A, with 8 = P.

Figures 5.8.2A, 5.8.2B, and 5.8.2C illustrate external stability equations for MSE walls with horizontal backslope, inclined backslope, and a broken backslope, respectively. Dead load surcharges, if present, shall be taken into account in accordance with Figures 5.8.12.1A, 5.8.12.1.B, and 5.8.12.1C. If a break in the slope behind the wall facing is located horizontally within two times the height of the wall (2H), a broken backslope design (A.R.E.A. method) shall be used, as illustrated in Figure 5.8.2.C. Alternatively, a broken back slope design can be performed for the actual slope geometry by using a graphical Coulomb procedure such as the Culmann method. For sliding stability, the coefficient of sliding used to calculate frictional resistance at the base of the wall shall be the minimum of the following determinations: Tan 4 at the base of the wall, where 4 is the friction angle of the backlil or the foundation soil, whichever is lowest. Tan p if continuous or near continuous reinforcement layers are used, where p is the soillreinforcement interface angle for the bottom of the lowest reinforcement layer. HIGHWAY BRIDGES 140 5.8.2 HORIZONTAL BACKSLOPE WITH TRAFFIC SURCHARGE ! k+lliiiliiti 1 i I I i 4 1l I r Assumed f o r beerlng cepecltg end o v e r e l l (globel) s t o b l l l t y comps. Assumed f o r overturning end slldzng reslstanee comps. q FACTOR OF SAFETY AGAINST OVERTURNING (MOMENTS ABOUT POINT 011 FSor - - Z Moments Reslrtlnq (Mr) Z Moments Overturning (Me) - F V, (L/2) (HI31 + 5 (HIZ~ 2*0 FACTOR OF SAFETY AGAINST SLIOINGI - += F r i c t ~ o nAngle where q = o f Reinforced B s c k f l l l o r Foundet~on. whichever 1s lower+. T r e f f l c Ltve Load 'Tenp 1s f o r contrnuour r o l l reinforcements (0.9..

g r ~ d sand sheets). P 1s the r o l l / For dlscontlnuour sol1 reinforcements (e.g.. strips) use Ton relnforcement ~ n t o r f e c ef r l c t l o n ongle. Use the lower of Ten + s t the boss of the well or Ten p s t the lowest relnforcement layer f o r continuous r e r n f orcements.

For r e l a t r v e l y thlck fecrng elements kg.. segmental concrete fecrng blocks) may be desirable t o ~ n c l u d ethe fecing dimenalona end weight In slldlng end overturning celculetrona (1.a.. u w '0' in lieu of 'Lm). lt FIGURE 5.8.2A External Stability for Wall with Horizontal Backslope and aaf'fic Surcharge DIVISION I-DESIGN 5.8.2 141 SLOPING BACKSLOPE CASE h I- -1 B FnCTOR OF SAFETY AGAINST OVERTURNING (MOMENTS ABOUT POINT 0) : V, (L/2) + V1(2L/31 + F, (L) X Moments Resrstlnq (Mr) FSo, = r 2.0 Z Moments Overturning (Mo) FH (h/3) . FACTOR OF SAFETY - X F s AGAINST SLIDING: Horizonto1 Resistinq Force(.) = ~ ~ H o r i r o n t a ~Driving Force(s) += F r i c t i o n = (VI +Vz +F.)(Ton P o r Tan FH .:= 1.5 Angle o f Reinforced B a c k f i l l o r Foundation. whichever i s lowest. 'Tan p i s f o r continuous s o i l reinforcements (e.g.. grids and sheets). PIS the s o i l / For d~scontinuouss o i l r e i n f o r c e m e n t r (e.9.. strips) use Ten reinforcement i n t e r f a c e f r i c t i o n angle.

Use the lower o f Ton + a t the base of the wall o r Tanp a t the lowest r e i n f o r c e m e n t layer f o r continuous reinforcements. Note: For r e l a t i v e l y t h i c k f a c i n g elements (e.9.. segmental c o n c r e t e facing blocks) i t may be desirabla t o include the f a c i n g dimensions and weight i n s l i d i n g and overturning calculations (1.8.. use *Be in l i e u of 'Lm). +. - FIGURE 5.8.2B External Stability for Wall with Sloping Backslope HIGHWAY BRIDGES 142 F,=F, 5.8.2 cor(1) FV= FT stn (1 =B FOR INFINITE SLOPE I K, For Retalned F111Using 8 = B = I (See Flgure 5.5.2A): I Srn2 OSrn(O - 8) ~1n(+'+8) Sln (9'- I S1d8 8)Sln(O +I - AGAINST OVERTURNING (MOMENTS ABOUT POINT 018 v, (L/2) + v2(2L/3) + Fv (L) X Moments R e r l r t r n ~(Mr) r 2.0 Z Moments Overturnrng (Mo) F, (h/3) FACTOR OF SAFETY F S ~ ,= - FACTOR OF SAFETY AGAZNST SLIDING8 F s=~ X Horrzontal Rekrstrnq Force(.) T~tlorlzontal Orivrng Force(.) * = F r r c t l o n Angle o f Reinforced = (V, +V2 +F,)(Ten P o r Ten FH zld B e c k f i l l or Foundetlon.

whichever 1s lowest, ' f a n P 1s for contrnuous sol1 reinforcements (6.9.. grrdr end sheets). strips) use 1an P IS the r o l l / r o l l rernforcements (e. For d~scont~nuous rernforcement r n ~ r f a o ef r r c t l o n engl* t i e the lower of Ten. e t the base of the w a l l o r Tenp e t the lowest rernforcement layer f o r continuous -ernforcements. Note: For reletlvely thrck fecrng elements (e.9.. segmental concreta fecrng blocks) ~tmay be derrrable t o rnclude the facing drmenrlons and welght ln slldlng end overturning celculetlons (1.0.. use '8' ln lieu o f 'L'l.

+. FIGURE 5.8.2C External Stability for Wall with Broken Backslope 5.8.2 DIVISION I-DESIGN See Appendix A of FHWA Publication No. FHWA SA96-071 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines" for how to determine Tan p from pullout or direct shear tests. If site specific data for tan p is not available, use 0.67Tan for the coefficient of sliding for continuous or near continuous reinforcement layers. For calculations of external stability, the continuous traffic surchargeloads shall be considered to act beyond the end of the reinforced zone as shown in Figure 5.8.2.A. Overall stability analyses shall be performed in accordance with Article 5.2.2.3. Additionally for MSE walls with complex geometries, or where walls support steep, infinite, sloping surcharges (i.e., a slope greater than 2H in length as shown in Figure 5.8.2C and a slope of 2H: 1V or steeper), 143 compound failure surfaces which pass through a portion of the reinforced soil mass as illustrated in Figure 5.8.2D shall be analyzed, especially where the wall is located on sloping or soft ground where overall stability is marginal. Factors of safety and methods of analysis provided in Article 5.2.2.3 are still applicable. The long-term strength of each backfill surface should be considered as restoring forces in the limit equilibrium slope stability analysis. + 5.8.3 Bearing Capacity and Foundation Stability Allowable bearing capacities for MSE walls shall be computed using a minimum factor of safety of 2.5 for Group 1 loading applied to the calculated ultimate bearing capacity. A lesser FS, of 2.0, could be used if justified Compound Stsb111q Failure rurfacer I I I I I I I I I I I I I I I I I I I I / I ' / / / I / 1 / I / / / / 0 0 0 / 0 # 0 0 / / / 0 / / / / / # / 0 # 0 / / / 0 .

. 0 0 / / / / 0 / / / Overall S t e b ~ l i : ~ Failure surface / / / 0 / / / / / I 1 / / I I / / / I I I 0 0 0 0 0 0 0 0 0 0 # / 0 @ 0 & & 0 0 / 0 *@ ---- ---- # - - --ee FIGURE 5.8.2D Overall and Compound Stability of Complex MSE Wall Systems HIGHWAY BRIDGES 144 crete wall facings are used due to their weight. Furthermore, differential settlements between the facing elements and the reinforced soil zone of the wall due to concentrated bearing stresses from the facing weight on soft soil could create concentrated stresses at the connection between the facing elements and the wall backfill reinforcement. In both cases, the leveling pad shall be embedded adequately to meet bearing capacity and settlement requirements or dimensioned and designed to keep bearing stresses beneath the leveling pad and the remainder of the wall as uniform as possible. by a geotechnical analysis. The width of the footing for ultimate bearing capacity calculations shall be considered to be the length of the reinforcement calculated at the foundation level. The location of the resultant center of pressure shall be as stated in Article 5.5.5. Provided the resultant location meets this criteria, an overturning stability analysis is not necessary. Bearing pressures shall be computed using the Meyerhof distribution, which considers a uniform base pressure distribution over an effective base width of B' = L - 2e, as shown in Figures 5.8.3A and 5.8.3B.

It is acceptable to use "By' in lieu of "L," especially for walls with relatively thick facing units. Where soft soils are present or if on sloping ground, the difference in bearing stress calculated for the wall reinforced soil zone relative to the local bearing stress beneath the facing elements shall be considered when evaluating bearing capacity. This is especially important where con- Reinforced Sorl Mass *r yr lC 5.8.3 5.8.4 Calculation of Loads for Internal Stability Design Reinforcement loads calculated for internal stability design are dependent on the soil reinforcement extensi- Retained Fill 3 Yr ICr T4 C- G 6- H 4 v V, = YpHL a * F2 =qHYr C- 1 II q = Traffic Live Load R = Resultant of Vertical Forces SUMMING MOMENTS ABOUT POINT CI If have concentrated dead loads, such as those illustrated i n figures 5.8.12.1A and 5.8.12.18. the externel forces resulting from those dead loads should be added t o the earth pressure force3 shown above by superposition (see Figure 5.8.12.1C). Note: For celatively thick facing elements (8.g.. segmental concrete facing blocks) it may be desirable to include the facing dimensions and weight I n bearlng capacity calculations (1.e..

use '8' i n lieu o f 'La). FIGURE 5.8.3A Calculation of Vertical Stress for Bearing Capacity Calculations (for Horizontal Backslope Condition) 5.8.4 R DIVISION I-DESIGN = 145 Resultant of verticel forces - Note: For r e l a t i v e l y thick fecrng elements (0.9.. segments1 concrete facing blocks) :may be dosirable to ~ncludethe facing dimrnsions and weight i n bearing capaclty ~ l c u l e t i o n s(1.0.. use '0' i n lieu of 'La). FIGURE 5.8.3B Calculation of Vertical Stress for Bearing Capacity Calculations (for Sloping Backslope Condition) 146 HIGHWAY BRIDGES bility and material type. In general, inextensible reinforcements consist of metallic strips, bar mats, or welded wire mats, whereas extensible reinforcements consist of geotextiles or geogrids. Inextensible reinforcements reach their peak strength at strains lower than the strain required for the soil to reach its peak strength. Extensible reinforcements reach their peak strength at strains greater than the strain required for soil to reach its peak strength. Internal stability failure modes include soil reinforcement rupture (ultimate limit state), soil reinforcement pullout (ultimate limit state), and excessive reinforcement elongation under the design load (serviceability limit state).

The serviceability limit state is not evaluated in current practice for internal stability design. Internal stability is determined by equating the tensile load applied to the reinforcement to the allowable tension for the reinforcement, the allowable tension being governed by reinforcement rupture and pullout. The load in the reinforcement is determined at two critical locations, i.e., at the zone of maximum stress and at the connection with the wall face, to assess the internal stability of the wall system. Potential for reinforcement rupture and pullout are evaluated at the zone of maximum stress. The zone of maximum stress is assumed to be located at the boundary between the active zone and the resistant zone. Potential for reinforcement rupture and pullout are also evaluated at the connection of the reinforcement to the wall facing. The maximum friction angle used for the computation of horizontal force within the reinforced soil mass shall be assumed to be 34", unless the specific project select backfill is tested for frictional strength by triaxial or direct shear testing methods, AASHTO T 234 and T 236, respectively. 5.8.4 account through an equivalent uniform surcharge and assuming a level backslope condition. For these calculations, the depth "Z" is referenced from the top of the wall at the wall face, excluding any copings and appurtenances. The lateral earth pressure coefficient ''K'is determined by applying a multiplier to the active earth pressure coefficient. The active earth pressure coefficient shall be determined using the Coulomb method as shown in Figure 5.5.2A, but assuming no wall friction (i.e., set 6 = fl). Note that since it is assumed that 6 = p, and fl is assumed to always be zero for internal stability, for a vertical wall, the Coulomb equation simplifies mathematically to the simplest form of the Rankine equation: If the wall face is battered, the following simplified form of the Coulomb equation can be used: with variables as defined in Figure 5.5.2A. The multiplier to K, shall be determined as shown in Figure 5.8.4.1C. Based on this figure, the multiplier to K, is a function of the reinforcement type and the depth of the reinforcement below the wall top. These multipliers are sufficiently accurate for the reinforcement types covered in Figure 5.8.4.1C. Multipliers for other reinforcement types can be developed as needed through analysis of measurements of reinforcementload and strain in full scale structures. The applied load to the reinforcements, T-, shall be calculated on a load per unit of wall width basis. Therefore, the reinforcemeit load, accounting for the tributary area of the lateral stress, is determined as follows: 5.8.4.1 Calculation of Maximum Reinforcement Loads Maximum reinforcement loads shall be calculated using a Simplified Coherent Gravity approach.

For this approach, the load in the reinforcements is obtained by multiplying a lateral earth pressure coefficient by the vertical pressure at the reinforcement, and applying the resulting lateral pressure to the tributary area for the reinforcement. Other widely accepted and published design methods for calculation of reinforcement loads may be used at the discretion of the wall owner or the approving agency. The vertical stress, a,, is the result of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present. Vertical stress for maximum reinforcement load calculations shall be determined as shown in Figures 5.8.4.1Aand 5.8.4.1B. Note that sloping soil surcharges are taken into where, ahis the horizontal soil stress at the reinforcement, S, is the vertical spacing of the reinforcement, K is the lateral earth pressure coefficient for a given reinforcement type and location, a, is the vertical eaah pressure at the reinforcement, and Amh is the horizontal stress at the reinforcement location resulting from a concentrated horizontal surcharge load. (See Article 5.8.12.1.) The design specifications provided herein assume that the wall facing combined with the reinforced backfill acts as a coherent unit to form a gravity retaining structure. The effect of relatively large vertical spacing of reinforcement on this assumption is not well known, and a vertical spacing greater than 0.8 meters (31 inches) shall DMSION I-DESIGN 5.8.4.1 J ,, Rernfor cud 147 Assumed only f

determined as follows: (See Figure 5.8.6B.) where, FS is a global safety factor which accounts for uncertainties in structure geometry, fill properties, externally applied loads, the potential for local overstress due to load nonu~formities,and uncertainties in long-term reinforcement strength. For ultimate limit state conditions for per- manent walls, a FS of 1.5 shall be used. Note that the uncertainty of determining long-term reinforcementstrength is taken into account through an additional factor of safety, which is typically about 1.2, depending on the amount of creep data available, through the creep extrapolation protocol provided in Appendix B of the FHWA-SA-96-071, "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines." 5.8.7 Soil Reinforcernenflacing Connection Strength Design 5.8.7.1 Connection Strength for Steel Soil Reinforcements Connections shall be designed to resist stresses resulting from active forces (To, as described in Article 5.8.4.2) as well as from differential movements between the reinforced backfill and the wall facing elements. Elements of the connection which are embedded in the facing element shall be designed with adequate bond length and bearing area in the concrete to resist the connection forces. The capacity of the embedded connector shall be checked by tests as required in Article 8.31. Connections between steel reinforcement and the wall facing units (e.g., welds, bolts, pins, etc.) shall be designed in accordance with Article 10.32.
Connection materials shall be designed to accommodate losses due to corrosion in accordance with Article 5.8.6.1 .l.
Potential differences between the environment at the face relative to the environment within the rein- forced soil mass shall be considered when assessing potential corrosion losses. 5.8.7.2 Connection Strength for Geosynthetic Reinforcements To evaluate the long-term geosynthetic strength at the connection with the wall facing, reduce T,, using the connectionlseamstrength determined in accordance with ASTM D 4884 for structural (i.e., not partial or full friction) connections. ASTM D 4884 will produce a shortterm connection strength equal to T, X C%. (See Equation 5.8.7.2-1.) Note that ASTM D 4884 will need to be modified to accommodategeogrid joints such as a Bodkin joint. The portion of the connection embedded in the concrete facing shall be designed in accordance with Article 8.31. For reinforcements connected to the facing through embedment between facing elements using a partial or full friction connection (e.g., segmental concrete block faced walls), the capacity of the connection shall be reduced from T,,, for the backfill reinforcement using the connection strength determined from laboratory tests. (See Equation 5.8.7.2-1.) This connection strength is based on the lessor of the pullout capacity of the connection, the long-term rupture strength of the connection and Tdas determined in Article 5.8.6.1.2. An appropriate laboratory testing and interpretation procedure, which is a modification of NCMATest Method SRWU-1 (Simac, et. al., 1993), is discussed in Appendix A of FHWA Publication No. FHWA SA-96-071 "Mechanically Stabilized TABLE 5.8.7.2A Default and Minimum Values for the Total Geosynthetic Ultimate Limit State Strength Reduction Factor at the Facing Connection, RF. Application Total Reduction Factor, RE'. -- All applications, but with product specific data obtained and analyzed in accordance with FHWA Publication No. FHWA SA-96-071 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines"-Appendix B, and FHWA Publication No. FHWA SA-96-072 "CorrosionlDegradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes." Permanent applications not having severe consequences should poor performanceor failure occur, nonaggressivesoils, and polymers meeting the requirements listed in Table 5.8.6.1.2A, provided product specific data is not available. If using polyester reinforcement, the pH regime at the connection must be investigated and determined to be within the pH requirements for a nonaggressive environment. (See Division 11, Article 7.3.6.3.) Temporary applications not having severe consequences should poor performance or failure occur, nonaggressivesoils, and polymers meeting the requirements listed in Table 5.8.6.1.2A, provided product specific data is not available. All reduction factors shall be based on product specific data. RFIDand RFDshall not be less than 1.1. 2.5 5.8.7.2 DIVISION I-DESIGN 159 tho k.01 of baa.
unit ww *wA -WB=\-,Include a11 u n r u tkot or0 ateckod over th. h l (Point 2) of the bwo aogmont.1 wit rh.ro % HA Werght ovor tho bow unrt Hinge Herght rhm lb= 0 Htnam Hmiaht., H,,.
Tho full rorght of a l l amgmont.1 facmg block unru rrthrn Hk m i l l bo conarderod act o t tho bow of tho lowormoat sogmont.1 facing block. Hh= 2 C(W, where: - - Gu 0.5Hu ten corn 1b l/tan(W + ib % %= &, = Segmental fecrng block u n i t hrrght (m) frgmmn+ol f ecing block unrt wrdth. f r o n t t o beck trn) drstancr to- the centar o f grevrtg o f a horizontel segmentml facing block unrt. rncluding aggregate frll. meessured from the f r o n t o f the u n i t (m) W = well b e t u r due to setback per courrno (deg) H = total height of wall (m) Hh= hrngm height (rn) FIGURE 5.8.7.2A Determinationof Hinge Height for Segmental Concrete Block Faced MSE Walls Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines." From this test, a peak connection strength load as a function of vertical confining stress, Tul, or T,, are obtained, which can be used to determine CR,, and CR, as follows: Tultc CR, = Tlot (5.8.7.2-1) CR, = (5.8.7.2-2) Tlot where, Td, is the peak load per unit reinforcement width in the connection test at a specified confining pressure where rupture of the reinforcement is known to be the mode of failure, T,, is the peak load per unit of reinforcement width in the connection test at a specified confining pressure where pullout is known to be the mode of failure, TlOtis the ultimate wide width tensile strength (ASTM D 4595) for the reinforcement material lot used for the connection strength testing, CR, is a reduction factor to account for reduced ultimate strength resulting from the connection where rupture is the mode of failure, and CR, is a reduction factor to account for reduced strength due to connection pullout. Therefore, determine the long-term geosynthetic connection strength T,, on a load per unit reinforcement width basis as follows: If the failure mode for the connection is rupture, 160 HIGHWAY BRIDGES 5.8.7.2 mine the minimum overlap length required, but in no case shall the overlap length be less than 1.0 meter (3.3 feet). If Tan p is determined experimentally based on soil to reinforcement contact, Tan p shall be reduced by 30 % where reinforcement to reinforcement contact is anticipated. 5.8.8 Design of Facing Elements If the failure mode for the connection is pullout, Tac = Tu~t x CRs (5.8.7.2-5) FS where, FS is as defined previously and is equal to 1.5 for permanent structures, RF, is a reduction factor to account for potential long-term degradation of the reinforcement at the wall face connection due to the environmentalfactors mentioned previously, and other variables are as defined previously. Note that the environment at the wall face connection may be different than the environment away from the wall face in the wall backfill.
This shall be considered when determining RFcRand RFD.
Values for RFcRand RFD shall in general be determined from product specific test results. Guidelines for how to determine & and RFDfrom product specific data are provided in FHWA Publication No. FHWA SA-96-071 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelinesw-Appendix B, and in FHWA Publication No. FHWA SA-96-072 "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes."For wall applications which are defined as not having severe consequences should poor performance or failure occur, having nonaggressive soil conditions, and if the geosynthetic product meets the minimum requirements listed in Table 5.8.6.1.2A, the long-term connection strength may be determined using a default reduction factor for RF,as provided in Table 5.8.7.2A for the ultimate limit state in lieu of product specific test results. Note that it is possible use of default reduction factors may be acceptable where the reinforcement load is maximum (i.e., in the middle of the wall backfill) and still not be acceptable at the facing connection if the facing environment is defined as aggressive. CR, and CR, shall be determined at the anticipated vertical confining pressure at the wall face between the facing blocks. The vertical confining pressure shall be calculated using the Hinge Height Method as shown in Figure 5.8.7.2A. Note that T, should not be greater than T,. Geosynthetic walls are sometimes designed using a flexible reinforcement sheet as the facing using only an overlap with the main backfill reinforcement.The overlaps shall be designed using a pullout methodology. Equation 5.8.5.2-1, but replacing T, with To,can be used to deter- Facing elements shall be designed to resist the horizontal forces calculated according to Articles 5.8.4.2 and 5.8.9.3. In addition to these horizontal forces, the facing elements shall also be designed to resist potential compaction stresses occurring near the wall face during erection of the wall. The facing elements shall be stabilized such that they do not deflect laterally or bulge beyond the established tolerances. 5.8.8.1 Design of Stiff or Rigid Concrete, Steel, and Timber Facings Facing elements shall be structurally designed in accordance with Sections 8, 10, and 13 for concrete, steel, and timber facings, respectively.
Reinforcement for concrete panels shall be provided to resist the average loading conditions for each panel.
As a minimum, temperature and shrinkage steel shall be provided. Epoxy coating for corrosion protection of panel reinforcement where salt spray is anticipated is recommended. 5.8.8.2 Design of Flexible Wall Facings If welded wire, expanded metal, or similar facing panels are used, they shall be designed in a manner which prevents the occurrence of excessive bulging as backfill behind the facing elements compresses due to compaction stresses or self weight of the backfill. This may be accomplished by limiting the size of individual panels vertically and the vertical spacing of the soil reinforcement layers, and by requiring the facing panels to have an adequate amount of vertical slip between adjacent panels. Furthermore, the top of the flexible facing panel at the top of the wall shall be attached to a soil reinforcement layer to provide stability to the top facing panel. For segmental concrete facing blocks, facing stability calculations shall include an evaluation of the maximum vertical spacing between reinforcement layers, the maximum allowable facing height above the uppermost reinforcement layer, inter-unit shear capacity, and resistance of the facing to bulging. The maximum vertical spacing between reinforcement layers shall be limited to twice the width, W,, (see Figure 5.8.7.2A), of the proposed segmental concrete facing unit or 0.8 meter (31 inches), 5.8.8.2 DIVISION I-DESIGN whichever is less, and the maximum facing height above the uppermost reinforcement layer and the maximum depth of facing below the bottom reinforcement layer should be limited to the width, W, (see Figure 5.8.7.2A), of the proposed segmental concrete facing unit. Geosynthetic facing elements shall not, in general, be left exposed to sunlight (specifically ultraviolet radiation) for permanent walls. If geosynthetic facing elements must be left exposed permanently to sunlight, the geosynthetic shall be stabilized to be resistant to ultraviolet radiation. Furthermore, product specific test data shall be provided which can be extrapolated to the intended design life and which proves that the product will be capable of performing as intended in an exposed environment. 5.8.8.3 Corrosion Issues for MSE Facing Design Steel to steel contact between the soil reinforcement connections and the concrete facing steel reinforcement shall be prevented so that contact between dissimilar metals (e.g., bare facing reinforcement steel and galvanized soil reinforcement steel) does not occur.
Steel to steel contact in this case can be prevented through the placement of a nonconductive material between the soil reinforcement face connection and the facing concrete reinforcing steel. 161 5.8.9.1A. Values of Ps and P, for structures with horizontal backfill shall be determined using the following equations: "A" is defined as the ground acceleration coefficient as determined in Division I-A, Article 3.2, in particular Figure 3. A, is defined as the maximum wall acceleration coefficient at the centroid of the wall mass. For ground accelerations greater than 0.45 g, A, would be calculated to be less than A. Therefore, if A > 0.45 g, set A, = A. The equation for P, was developed assuming a friction angle of 30". Pm may be adjusted for other soil friction angles using the Mononobe-Okabe method, with the horizontal acceleration kh equal to A, and k, equal to zero. For structures with sloping backfills, the inertial force (P,) and the dynamic horizontal thrust (PAE)are based on a height Hz near the back of the wall determined as follows: 5.8.9 Seismic Design The seismic design procedures provided herein do not directly account for the lateral deformation which may occur during large earthquake seismic loading. It is therefore recommended that if the anticipated ground acceleration is greater than 0.29 g, a detailed lateral deformation analysis of the structure during seismic loading should be performed. P, shall be adjusted for sloping backfills using the Mononobe-Okabe method, with the horizontal acceleration kh equal to A, and k, equal to zero. A height of HZ shall be used to calculate PAEin this case. P, for sloping backfills shall be calculated as follows: 5.8.9.1 External Stability Stability computations (i.e., sliding, overturning, and bearing capacity) shall be made by including, in addition to static forces, the horizontal inertial force (P,) acting simultaneously with 50% of the dynamic horizontal thrust (Pm) to determine the total force applied to the wall. The dynamic horizontal thrust Pm is evaluated using the pseudo-static Mononobe-Okabe method and is applied to the back surface of the reinforced fill at a height of 0.6H from the base for level backfill conditions. The horizontal inertial force P, is determined by multiplying the weight of the reinforced wall mass, with dimensions of H (wall height) and 0.5H, assuming horizontal backfill conditions, by the acceleration A,. P, is located at the centroid of the structure mass. These forces are illustrated in Figure where, Pi,is the inertial force caused by acceleration of the reinforced backfill and Pisis the inertial force caused by acceleration of the sloping soil surcharge above the reinforced backfill, with the width of mass contributing to P, equal to 0.5H2.PIRacts at the combined centroid of Pi,and Pis.This is illustrated in Figure 5.8.9.1A. Factors of safety against sliding, overturning, and bearing capacity failure under seismic loading may be reduced to 75% of the factors of safety defined in Articles 5.8.2 and 5.8.3. The factor of safety for overall stability may be reduced to 1.1. (See Article 5.2.2.3.) 162 HIGHWAY BRIDGES (a) Level backf rll condrtlon / Y k m m for ree~mur*) fa+.. (b)Slopinq backf ~ 1 condrtron 1 FIGURE 5.8.9.1A Seismic External Stability of a MSE Wall 5.8.9.2 5.8.9.2 DIVISION I-DESIGN 163 Internal Stability Lei Tmd= Pi 7 Reinforcements shall be designed to withstand horizontal forces generated by the internal inertial force (PI) in addition to the static forces. The total inertial force PI per unit width of structure shall be considered equal to the weight of the active zone times the maximum wall acceleration coefficient A,. This inertial force is distributed to the reinforcements proportionally to their resistant areas on a load per unit of wall width basis as follows: Ac trve (5.8.9.2-1) xed As shown in Figure 5.8.9.2A, the total load applied to the reinforcement on a load per unit of wall width basis is as follows: is determined using Equation 5.8.4.1-3. where, T, / I--- u Inex tensible Rernf orcements Extensrble Relnf orcements S - L .I = The length of reinforcementin the resistant zone of the iYh layer. T- = The load per unit wall width applied to each reinforcement due to static forces. Td = The load per unit wall width applied to each reinforcement due to dynamic forces. Internalinertial force due to the weight of the backfill within the active zone. = TThe total load per unit wall width applied to each layer, For seismic loading, the dimensions of the active zone are the same as for the static loading. (See Figure 5.8.5.1A.) + d FIGURE 5.8.9.2A Seismic Internal Stability of a MSE Wall 164 HIGHWAY BRIDGES For seismic loading conditions, the value of F*, the pullout resistance factor, shall be reduced to 80% of the values used for static design. Factors of safety under combined static and seismic loads for pullout and breakage of reinforcement may be reduced to 75% of the factors of safety used for static loading. For geosynthetic reinforcement rupture, the reinforcement must be designed to resist the static and dynamic components of the load as follows: For the static component, 5.8.9.2 If the seismic performance category is "C" or higher (see Section 3, Division I-A), facing connections in segmental block faced walls shall not be fully dependent on frictional resistance between the backfill reinforcement and facing blocks. Shear resisting devices between the facing blocks and backfill reinforcement such as shear keys, pins, etc. shall be used. For steel reinforcement connections, safety factors for combined static and seismic loads may be reduced to 75% of the safety factors used for static loading. Based on these safety factors, the available connection strength must be greater than Tmd.
For the static component, For the dynamic component, Tmd S, x RC FSxmxRF,, (5.8.9.2-4) Therefore, the ultimate strength of the geosynthetic reinforcement required is, Tult = S, + S* (5.8.9.2-5) For reinforcement pullout, FSPO x Tm, L > e-0.8F* x x x xo,x C x Rc (5.8.9.2-6) For the dynamic component, T md Lateral wall displacements are a function of overall structure stiffness, compaction intensity, soil type, reinforcement length, slack in reinforcement-to-facing connections, and deformability of the facing system. A first order estimate of lateral wall displacements occurring during wall construction for simple MSE walls on firm foundations can be determined from Figure 5.8.10A.
If significant vertical settlement is anticipated or heavy surcharges are present, lateral displacements could be considerably greater. Appropriate uses of this figure are as a guide to establish an appropriate wall face batter to obtain a near vertical wall or to determine minimum clearances between the wall face and adjacent objects or structures.
5.8.11 Drainage MSE walls in cut areas and side-hill fills with established ground water levels should be constructed with drainage blankets in back of and beneath the reinforced zone. Internal drainage measures should be considered for all structures to prevent saturation of the reinforced backfill or to intercept any surface flows containing aggressive elements such as deicing chemicals. DIVISION I-DESIGN 5.8.11 165 3 . -% a H/250 (INEXTENSIBLEI &mex w . H/75 &me, gmox= MAXIMUM DISPLACEMENT WHERE: IN UNITS OF H I H \ (EXTENSIBLE) I = HEIGHT OF WALL IN m. = EMPIRICALLY DERIVE0 RELATIVE DISPLACEMENT " I Based on 6 m (20 foot) high wells. r e l a t i v e displacement increases approximetely 25% f o r every 19 kPa (400 psf) of surcharge. Experience indicates t h a t f o r higher wells, the surcherge e f f e c t mey be greater. Note: This figure i s only e guide. Actus1 displecement w ~ l l depend, i n addition to the poremeters eddressed i n the figure. on s o i l cherecteristics. compaction efforf, end c o n t r e c t o r workmenship. FIGURE 5.8.10A Empirical Curve for Estimating Anticipated Lateral Displacement During Construction for MSE Walls For MSE walls utilizing metallic backfill reinforcements supporting roadways which are chemically deiced in the winter, an impervious membrane should be placed below the pavement and just above the first row of reinforcements to intercept any flows containing deicing chemicals.
The membrane should be sloped to drain away from the facing to an intercepting longitudinal drain outletted beyond the reinforced zone. Typically, a minimum membrane thickness of 0.8 mrn (30 mils) should be used. All seams in the membrane shall be welded to prevent leakage. 5.8.12 Special Loading Conditions 5.8.12.1 Concentrated Dead Loads Concentrated dead loads shall be incorporated into the internal and external stability design by using a simplified uniform vertical distribution of 2 vertical to 1 horizontal to determine the vertical component of stress with depth within the reinforced soil mass as shown in Figure 5.8.12.1A. Figure 5.8.12.1B shows how concentrated horizontal dead loads are distributed within and 166 HIGHWAY BRIDGES 2 U F o r r t r r p loed: 5.8.12.1 Au, - 5 0, F o r r r o l a t m d f o o t r n g load: A v -,&&z For p o i n t loeda A e, Where: 0, = E f f e c t r v e wrdth of shown above P"' w i t h bf T + 8 a p p l i e d l o a d a t any depth. calculated as b f = Wrdth o f applred loed. F O ~ f o o t r n g r whrch a r e e e c c e n t r i c a l l y loaded (0.9.r brrdge ebutmont footrngs). s e t b, equal t o the equrvalmnt f o o t r n g wrdth B' by reducrng i t by 2e'. where e' rs t h e e c c e n t r r c i t y o f t h e f o o t r n g l o a d (r.m.. b y -20'). L = Length of footrng Pv = Load pmr lrnmar meter (f o o t l of s t r r p P;= Load on i s o l a t e d r e c t a n g u i e r f o o t r n g z a = dmpth wherm e f f e c t i v e footrng o r p o l n t load wrdth i n t e r s e c t s back of wall f a c e = 2d - bf Assume the rncreased v e r t i c a l s t r e s s due t o the surcharge l o a d her n o rnfluence o n s v e s s e r used t o evaluate r n t e r n a l s t a b i l i t y if the r u r c h a r c - l o a d r r l o c a t e d behrnd the r a r n f o r c e d r o r l mars. For e x t e r n e l r t a b r l r f ~ assume t h e surcharge has no rnfluence r f ~t1s l o c a t e d outsrde th a c t i v e zone behind the well. FIGURE 5.8.12.1A Distribution of Stress from Concentrated Vertical Load P, for Internal and External Stability Calculations 5.8.12.1 DIVISION I-DESIGN Ao,mox a 2 ZF/1 167 I ZFgPn +Fl +Fz lotorel forco due to eorth prasswe l o t a r o l forca dua to t r o f f i c suichorga lotorol force d w to s u p n s t r u c t u e or other concentrotad lotarol loods Stress Oistributron a's accmntrrcity of lood on f w t l n ~ (sae Frgure 5.8.12.1C for erompla of how to colculot. thasl o. Oistribuuon of Stress f o r 1nt.rnol Stability Colculotros. PMZ=l o t ~ r oforcm l d m to superstructure or other concentrotad lotaro1 loods f - I If footrng is locotmd completaly outsrde octrve zom behrnd roll. the footing load does n o t namd to be considerad m-the axtarno1 stsbtlrty colculotrons. b. Otstributron of Strmss fe Ertdrnol Smbtltty Colculstrons. FIGURE 5.8.12.1B Distribution of Stress from Concentrated Horizontal Loads behind the reinforced soil mass. Concentrated horizontal loads at the top of the wall shall also be distributed within the reinforced soil mass as shown in this figure. Figure 5.8.12.1C shows how these loads can be combined using superposition principles to evaluate external and internal wall stability. Depending on the size and location of the concentrated dead load, the location of the boundary between the active and resistant zones may need to be adjusted. Figure 5.8.12.1D illustrates how this adjustment should be made. When dead load surcharges above or within the reinforced soil zone are present, the reinforcement connections to the wall face shall be designed for 100% of T, (or T,,,, for seismic loads) throughout the height of the wall. If concentrated dead loads are located behind the reinforced soil mass, they shall be distributed in the same way as would be done within the reinforced soil mass. The vertical stress distributed behind the reinforced zone in this way shall be multiplied by Kd to determine the effect this surcharge load has on external stability. The concentrated 168 HIGHWAY BRIDGES Concentrated Deed Load .l m 5.8.12.1 , Concantrated Dead Load .2 m Notu: These equetlons assume t h a t concentrated dead load *2 1s located wrthrn the sctrve zone behlnd the e I r e l n f orced sol1 mass. For External Stabrllty. Summing Moments about Point 0: y (LIZ) + Pn (XI + qL (L/2) FSoT= F1(H/3) + 5 (HI21 + P(,H) + Pm(H-12/3) + fo h, (Vl+qL+Pvl)Tan FSu = F, + f 2 + P,++,P F, B For Bearing Caluletlons. Summrng Moments about Polnt C: e fl (HI31+ = Fz (HI21+ Pm (H-12 /3) + F, ho+ PHI H f Pv (L/2 - XI r;HL+qL+ P , +qL + &", L -2e Note: For relatively thlck facrng elements (e.g., segmental concrete fsclng blocks) r t may be d e ~ l r a b l eto lnclude the facing dlmenslons end welght I n slldlng. overturn and beerang capacltg calculat~ons(l.e.,-use '8' l n l l e u of 'L'). v, = v1 are as determrned from Note: Pvl, PWI.P!. &*Aq,,&,.h~n.and l2 f lgures S.8.12.1A and 5.8.12.18 and f, results from P! (1.e. K H 1s the t o t a l wall height s t the face. from f lgure 5.8.12-1A)-FIGURE 5.8.12.1C Superposition of Concentrated Dead Loads for External Stability Evaluation 5.8.12.1 DIVISION I-DESIGN 169 P v A A ,,- , / v / .:.,.',,/ ,,,' I I I H 2 H A H 2 u RESIST ANT ZONE - FIGURE 5.8.12.1D Location of Maximum Tensile Force L i e in Case of Large Surcharge Slabs (Inextensible Reinforcements) horizontal stress distributed behind the wall can be taken into account directly. 5.8.12.2 Traffic Loads and Barriers Traffic loads shall be treated as uniform surcharge loads in accordance with the criteria outlined in Article 3.20.3. The live load surcharge pressure shall be equal to not less than 0.6 meter (2 feet) of earth.
Parapets and traffic barriers, constructed over or in line with the front face of the wall shall be designed to resist overturning moments by their own mass. Base slabs shall not have any transverse joints except construction joints, and adjacent slabs shall be joined by shear dowels. The upper row(s) of soil reinforcement shall have sufficient tensile capacity to resist a concentrated horizontal load of 45 kN (10 kips) distributed over a barrier length of 1.5 meters (5 feet).
This force distribution accommodates the local peaking of force in the soil reinforcements in the vicinity of the concentrated load. This distributed force would be equal to PHIin Figure 5.8.12.1B, and would be distributed to the reinforcements assuming bf equal to the width of the base slab. Adequate room shall be provided laterally between the back of the facing panels and the traffic barrierlslab to allow the traffic barrier and slab to resist the impact load in sliding and overturning without directly transmitting load to the top facing units. For checking pullout safety of the reinforcements, the lateral traffic impact load shall be distributed to the upper soil reinforcement and facing units using Figure 5.8.12.1B, assuming bf equal to the width of the base slab. The full length of reinforcements shall be considered effective in resisting pullout due to impact load. The upper row(s) of soil reinforcement shall have sufficient pullout capacity to resist a horizontal load of 45 kN (10 kips) distributed over the full 6 meters (20 feet) base slab length. The force distribution for pullout computations is different than what is used for tensile capacity computations because the entire base slab must move laterally to initiate a pullout failure due to the relatively large deformation required, This distributed force would be equal to PHI in Figure 5.8.12.1B. Due to the transient nature of tr c barrier impact loads, when designing for reinforcement rupture, the geosynthetic reinforcement must be designed to resist the static and transient (impact) components of the load as follows: For the static component, see equation 5.8.9.2-3. 170 HIGHWAY BRIDGES For the transient component, 5.8.12.2 described in Article 5.8.12.4. The upper two rows of reinforcement shall be designed for an additional horizontal load of 4,400 N per linear meter of wall (300 pounds per linear foot of wall). where Aohis the traffic barrier impact stress applied over the reinforcement tributary area as determined in Article 5.8.12.1. The reinforcement strength required for the static component, S,,, must be added to the reinforcement strength required for the transient component, S,, to determine the total ultimate strength required for the reinforcement, T,,,.
Parapet reinforcement shall be in accordance with Article 2.7. The anchoring slab shall be strong enough to resist the ultimate strength of the standard parapet. Flexible post and beam barriers, when used, shall be placed at a minimum distance of 1.0 meter (3.3 feet) from the wall face, driven 1.5 meters (5 feet) below grade, and spaced to miss the reinforcements where possible. If the reinforcements cannot be missed, the wall shall be designed accounting for the presence of an obstruction as 5.8.12.3 Hydrostatic Pressures For structures along rivers and canals, a minimum differential hydrostatic pressure equal to 1.0 meter (3.3 feet) of water shall be considered for design. This load shall be applied at the high-water level. Effective unit weights shall be used in the calculations for internal and external stability beginning at levels just below the equivalentsurface of the pressure head line. Situations where the wall is influenced by tide or river fluctuations may require that the wall be designed for rapid drawdown conditions, which could result in differential hydrostatic pressure considerably greater than 1.0 meter (3.3 feet), or alternatively rapidly draining backfill material such as shot rock or open graded coarse gravel be used as backfill. Backfill material meeting the gradation Soil Rornforcornant T or ffffflt! Structural F r a m e Obrwuct~on T " / / // T = T o t a l Load which S t r u c t u r o l F r a m e must c a r r y . Plen V i e w FIGURE 5.8.12.4A Structural Connection of Soil Reinforcement Around Backfill Obstructions 5.8.12.3 DMSION I-DESIGN requirements in Article 7.3.6.3 of Division 11 is not considered to be rapid draining. 5.8.12.4 Design for Presence of Obstructions in the Reinforced Soil Zone If the placement of an obstruction in the wall soil reinforcement zone such as a catch basin, grate inlet, signal or sign foundation, guardrail post, or culvert cannot be avoided, the design of the wall near the obstruction shall be modified using one of the following alternatives: (1) Assuming reinforcement layers must be partially or fully severed in the location of the obstruction, design the surrounding reinforcement layers-to carry the additionalload which would have been carried by the severed reinforcements. (2) Place a structural frame around the obstruction which is capable of carrying the load from the reinforcements in front of the obstruction to reinforcements connected to the structural frame behind the obstruction. This is conceptually illustrated in Figure 5.8.12.4A. (3) If the soil reinforcements consist of discrete strips or bar mats rather than continuous sheets, depending on the size and location of the obstruction, it may be possible to splay the reinforcementsaround the obstruction. - -- -- For the first alternative, the portion of the wall facing in front of the obstruction shall be made stable against a toppling (overturning) or sliding failure. If this cannot be accomplished, the soil reinforcements between the obstruction and the wall face can be structurally connected to the obstruction such that the wall face does not topple, or the facing elements can be structurally connected to adjacent facing elements to prevent this type of failure. For the second alternative, the frame and connections shall be designed in accordance with Article 10.32 for steel frames. Note that it may be feasible to connect the soil reinforcement directly to the obstruction depending on the reinforcement type and the nature of the obstruction. For the third alternative, the splay angle, measured from a line perpendicular to the wall face, shall be small enough that the splaying does not generate moment in the reinforcement or the connection of the reinforcement to the wall face. The tensile capacity of the splayed reinforcement shall be reduced by the cosine of the splay angle. If the obstruction must penetrate through the face of the wall, the wall facing elements shall be designed to fit around the obstruction such that the facing elements are stable (i.e., point loads should be avoided) and such that wall backfill soil cannot spill through the wall 171 face where it joins the obstruction. To this end, a collar next to the wall face around the obstruction may be needed. If driven piles must be placed through the reinforced zone, the recommendations provided in Section 7 of Division I shall be followed. 5.9 PREFABRICATED MODULAR WALL DESIGN 5.9.1 Structure Dimensions Prefabricated modular walls shall be dimensioned to-ensure that the applicable factors of safety outlined in Article 5.5.5 are satisfied.
Minimum embedment and scour protection shall satisfy the requirements of Article 5.8.1. 5.9.2 External Stability Stability computations shall be made by assuming that the system acts as a rigid body. Lateral pressures shall be computed by wedge theory using a plane surface of sliding (Coulomb theory). Where the rear of the prefabricated modular systems forms an irregular surface (stepped modules), pressures shall be computed on an average plane surface drawn from the lower back heel of the lowest module to the upper rear heel of the top module, as shown in Figures 5.9.2A and 5.9.2B.
The following wall friction angles, 6, shall be used unless more exact coefficients are demonstrated: (a) (b) (c) Case Significant vibrations of backfill or modules settling more than backfill Continuous pressure surface of precast concrete (uniform width modules) Averaged pressure surface (stepped modules) Wall Friction Angle (6) 0 1/24) 3/44) Computations for stability shall be made at every module level. At each level, the required factors of safety with respect to overturning shall be provided. The value of K, used to compute the lateral thrust resulting from the random backfill and other loads shall be computed on HIGHWAY BFUDGES 172 5.9.2 FIGURE 5.9.2A Lateral Earth Pressures for Prefabricated Modular Walls Case I-Continuous Pressure Surfaces the basis of the friction angle of the backfill behind the modules. If sufficient amounts of structural backfill are used behind the prefabricated modules, a value of 34" may be used for In the absence of specific data, a maximum friction angle of 30" shall be used. The coefficient of sliding friction at the wall base shall be the lesser of the coefficients of the backfill or the foundation soil. Passive pressures shall be neglected in stability computations. Computations for overturning stability shall consider that only 80% of the soil-fill unit weight inside the mod- +. ules is effective in resisting overturning moments. In the absence of specific data, a total unit weight of 110 pounds per cubic foot shall be assumed. Computations for sliding stability may consider that 100% of the soil-fill weight inside the modules is effective in resisting sliding motion. The value of of the foundation soils shall be used in these computations. For structures loaded with sloping surcharges, refer to Article 5.2.2.3 regarding overall stability analysis of slopes. + 5.9.3 DIVISION I-DESIGN 173 FIGURE 5.9.2B Lateral Earth Pressures for Prefabricated Modular Walls Case II-Irregular Pressure Surfaces 5.9.3 Bearing Capacity and Foundation Stability Allowable bearing capacities for concrete modular systems shall be computed using a minimum factor of safety of 3 for Group I loading applied to the ultimate bearing capacity or to a bearing capacity obtained in accordance with Articles 4.4.7 and 4.4.8. Footing loads shall be computed by assuming that dead loads and earth pressure loads are resisted by point sup- ports per unit length, at the rear and front of the modules or at the location of the bottom legs. For modules supported on integrally cast legs, the reactions shall be similarly calculated. For this computation, a minimum of 80% of the soil weight inside the modules shall be considered effective. If foundation conditions require a footing under the total area of the module, 100% of the soil weight inside the modules shall be considered. 174 HIGHWAY BRIDGES The overall slope stability condition, of which the retaining wall may only be part, shall be evaluated in accordance with Article 5.2.2.3. 5.9.4 Allowable Stresses Prefabricated modular units shall be designed for developed earth pressures behind the wall and from pressures developed inside the modules. Rear face surfaces shall be designed for the difference of these pressures. Allowable stresses and reinforcement requirements for concrete modules shall be in accordance with Section 8. Inside pressures (bin) shall be the same for each module and shall not be less than as follows: Concrete modules shall be designed for bending in both vertical and horizontal directions between their supports. Steel reinforcing shall be symmetrical on both faces unless positive identification of each face can be ensured to preclude reversal of units. Comers shall be adequately reinforced. Allowable stresses for steel module members shall be in accordance with Article 10.32. The net section used for design shall be reduced in accordance with Article 5.8.6.1. 5.9.5 Drainage Prefabricated modular units in cut and side-hill fill areas shall be designed with a continuous subsurface drain placed at, or near, the footing grade and out-letted as required.
In cut and side-hill fill areas with established or potential ground water levels above the footing grade, a continuous drainage blanket shall be provided and connected to the longitudinal drain system. For systems with open front faces, a surface drainage system shall be provided as needed above the top of the wall to collect and divert surface runoff and prevent erosion of the front face. Part C STRENGTH DESIGN METHOD LOAD FACTOR DESIGN 5.10 SCOPE The provisions of this Part shall apply for the design of rigid gravity and semi-rigid gravity walls, and nongravity cantilevered walls. The probabilistic Lm) basis of these specifications which produces an inter-related combination of load, load 5.9.3 factor, and statistical reliability shall be considered when selecting procedures for calculating resistance. The procedures used in developing values of performance factors contained in this Part are summarized in Appendix A of the Final Report for NCHRP Project 24-4 (Barker, et al., 1991). Other methods may be used if the statistical nature of the factors given above are considered, and are approved by the owner. 5.11 DEFINITIONS Only terms relating to retaining walls are provided in this Section. Definitions for terms relating to foundation types and LFD design are given in Article 4.8. Cantilever Walls-Walls that resist the forces exerted on them by flexural strength. These walls consist of a concrete wall stem, a concrete slab, and possibly a shear key. Gravity Walls-Massive stone or concrete masonry walls which depend primarily on their weights to maintain stability. Only a nominal amount of steel is placed near the exposed faces of these walls to prevent surface cracking due to temperature changes. Retaining Walls--Structures that provide lateral support for a mass of soil and that owe their stability primarily to their own weights and to the weights of any soils located directly above their base. Semi-gravity M1alls-These walls are somewhat more slender than gravity walls and require reinforcement consisting of vertical bars along the inner.face and dowels continuing into the footing. 5.12 NOTATIONS F, = sliding resistance H = height of retaining wall Hp = factored horizontal load K = coefficient of earth pressure K,, = coefficient of earth pressure at rest N = factored bearing pressure resultant P = lateral earth pressure Pa = active earth load Ph = lateral earth load P, = vertical earth load qf = factored bearing capacity q,, = maximum bearing pressure calculated using factored loads q, = surcharge loading qd, = ultimate bearing capacity R1 = reduction factor due to load inclination effect R, = nominal resistance Vf = factored vertical load y = distance to the point of action for lateral earth pressure 5.12 A DMSION I-DESIGN , weeK p Pe = load factor coefficient (see Article 5.13.4) y yeq 6 A = load factor (See Article 5.13.4) = equivalent fluid pressure + = load factor coefficient for earth pressure 175 -groundddormabiKty, -groundwater, and -swelling pressure in clay backfills. 5.13.3 Strength Requirement = angle of shearing resistance between wall and soil = wall displacement = performance factor 5.13 LIMIT STATES, LOAD FACTORS AND RESISTANCE FACTORS All relevant limit states shall be considered in the design to ensure an adequate degree of safety and serviceability, 5.13.1 Serviceability Limit States Design of rigid gravity and semi-gravity walls, and nongravity cantilever walls shall consider the following serviceability limit states: e x c e s s i v e movements of retaining walls and their foundations, -excessive vibrations caused by dynamic loadings, and deterioration of element(s) of retaining structures. The limit state for settlement shall be based upon rideability and economy. The cost of limiting foundation movements shall be compared to the cost of designing the superstructure so that it can tolerate larger movements, or of correcting the consequences of movements through maintenance, to determine minimum lifetime cost. More stringent criteria may be established by the owner. 5.13.2 Strength Limit States Design of rigid gravity and semi-gravity walls, and nongravity cantilever walls shall be checked against the strength limit states of: Retaining walls and their foundations shall be proportioned by the methods specified in Article 5.14 so that their design strength exceeds the required strength. The required strength is the combined effect of factored loads for each applicable load combination stipulated in Article 3.22. The design strength is calculated for each applicable limit state as the nominal resistance, R,, multiplied by an appropriate performance (or resistance) factor, 4. Procedures for calculating nominal resistance are provided in Article 5.1, and values of performance factors are given in Article 5.13.5. 5.13.4 Load Combinations and Load Factors Retaining structures and their foundations shall be proportioned to withstand safely all load combinations stipulated in Article 3.22 which are applicable to the particular site or wall/foundation type. Impact forces shall not be included in retaining wall design.
(Refer to Article 3.8.) Values of y and p coefficients for load factor design, as given in Table 3.22.1A, shall apply to strength limit state considerations; while those for service load design (also given in Table 3.22.1A) shall apply to serviceability considerations.
5.13.5 Performance Factors Values of performance factors for geotechnical design of foundations are given in Tables 4.10.6-1 through 4.10.6-3, while those for structural design are provided in Article 8.16.1.2.2. If methods other

zontal distance equal to 50% of the height of the abutment wall. The horizontal forces transmitted to the piles shall be resisted by the lateral capacity of the pile itself, or the soil reinforcements in the upper part of the wall designed to carry the additional loads transmitted from the piles to the reinforced soil backfill. Where interference between the piles and the soil reinforcement occurs, the reinforcements must be designed around the piles, and the piles treated as backfill obstructions (see Article 5.8.12.4). A clear distance of no less than 0.5 meters (1.5 feet) from the back of the wall facing to the edge of the nearest pile or pile casing shall be provided. Piles should be driven prior to wall construction and cased through fill if necessary. Lateral loads transmitted from the piles to the reinforced backfill may be determined using a P-Y lateral load analysis technique. 7.5.5 Abutments on Modular Systems Abutments seats constructed on modular units shall be designed by considering, in addition to earth pressures, the supplemental horizontal pressures from the abutment seat beam and earth pressures on the back wall. The top module shall be proportioned to be stable, with the required factor FIGURE 7.5.4A Limiting Values of Dierential Settlement Based on Field Suweys of Simple and Continuous Span Structures of Various Span Lengths, Moulton, et al. (1985) 7.5.4 DIVISION I-DESIGN of satety, under the comblned achons or normal and supplementary earth pressures. Minimum top module width shall be 6 feet. The center line of bearing shall be located a minimum of 2 feet from the outside face of the top precast module. The abutment beam seat shall be supported and cast integrally to the top module. The front face thickness of the top module shall be designed for bending forces developed by supplemental earth pressures. Abutment beamseat loadings shall be carried to foundation level and shall be considered in the design of footings. Differential settlement restrictions in Article 7.5.4. shall apply. 7.5.6 Wingwalls 7.5.6.1 Length Wingwalls shall be of sufficient length to retain the roadway embankment to the required extent and to furnish protection against erosion.
The wingwall lengths shall be computed using the required roadway slopes. 7.5.6.2 Reinforcement Reinforcing bars or suitable rolled sections shall be spaced across the junction between wingwalls and abut- 187 ments to tie &em together. Such b ~ ~ 3 i € masonry on each side of the joint far enough to develop the strength of the bar as specified for bar reinforcement, and shall vary in length so as to avoid planes of weakness in the concrete at their ends. If bars are not used, an expansion joint shall be provided and the wingwall shall be keyed into the body of the abutment. Part C STRENGTH DESIGN METHOD LOAD FACTOR DESIGN 7.6 GENERAL The provisions of Articles 7.1 through 7.5 shall apply to the load factor design of abutments with the exception that: (1) Article 7.5.2 on loading shall be replaced by the articles for loads, earth pressures and water pressures in Articles 5.13 and 5.14 for retaining walls, and (2) Article 7.5.2.1 shall be replaced by the articles for stability in Articles 5.13 and 5.14.
Abutments shall be designed to withstand earth pressures, water pressures and other loads similar to the design of retaining walls. F Section 8 REINFORCED CONCRETE* Part A GENERAL REQUIREMENTS AND MATERIALS = area of reinforcement in bracket or corbel re- 8.1 APPLICATION 8.1.1 General The specifications of this section are intended for design of reinforced (nonprestressed) concrete bridge members and structures. Bridge members designed as prestressed concrete shall conform to Section 9. 8.1.2 Notations a = depth of equivalent rectangular stress block ab = depth of equivalent rectangular stress block (Article 8.16.2.7) a, A Ab & &V for balanced strain conditions, in. (Article 8.16.4.2.3) = shear span, distance between concentrated load and face of support (Articles 8.15.5.8 and 8.16.6.8) = effective tension area, in square inches, of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires.
When the flexural reinforcement consists of several bar or wire sizes, the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used. For calculation purposes, the thickness of clear concrete cover used to compute A shall not be taken greater than 2 inches. = area of an individual bar, sq. in. (Article 8.25.1) = area of core of spirally reinforced compression member measured to the outside diameter of the spiral, sq. in. (Article 8.18.2.2.2) = area of concrete section resisting shear transfer, sq. in. (Article 8.16.6.4.5) Ask sisting moment, sq. in. (Articles 8.15.5.8 and 8.16.6.8) = gross area of section, sq. in. = area of shear reinforcement parallel to flexural tension reinforcement, sq. in. (Articles 8.15.5.8 and 8.16.6.8) = area of reinforcement in bracket or corbel resisting tensile force N, (Nu!)sq.
, in. (Articles 8.15.5.8 and 8.16.6.8) = area of tension reinforcement, sq.
in. = area of compression reinforcement, sq. in. = area of reinforcement to develop compressive strength of overhanging flanges of I- and T-sections (Article 8.16.3.3.2) = area of skin reinforcement per unit height in one side face, sq. in.
per ft. (Article 8.17.2.1.3). = total area of longitudinal reinforcement (Articles 8.16.4.1.2 and 8.16.4.2.1) = area of shear reinforcement within a distance s = area of shear-friction reinforcement, sq. in. (Article 8.15.5.4.3) = area of an individual wire to be developed or spliced, sq. in. (Articles 8.30.1.2 and 8.30.2) = loaded area (Articles 8.15.2.1.3 and 8.16.7.2) = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area (Articles 8.15.2.1.3 and 8.16.7.2) = width of compression face of member = perimeter of critical section for slabs and footings (Articles 8.15.5.6.2 and 8.16.6.6.2) = width of cross section at contact surface being investigated for horizontal shear (Article 8.15.5.5.3) *The specifications of Section 8 are patterned after and are in general conformity with the provisions of ACI Standard 318 for reinforced concrete design and its commentary, ACI 318 R, published by the American Concrete Institute.
HIGHWAY BRIDGES = web width, or diameter of circular section (Article 8.15.5.1.1) = distance from extreme compression fiber to neutral axis (Article 8.16.2.7) = factor relating the actual moment diagram to an equivalent uniform moment diagram (Article 8.16.5.2.7) = distance from extreme compression fiber to centroid of tension reinforcement, in. For computing shear strength of circular sections, d need not be less than the distance from extreme compression fiber to centroid of tension reinforcement in opposite half of member. For computing horizontal shear strength of composite members, d shall be the distance from extreme compression fiber to centroid of tension reinforcementfor entire composite section. = distance from extreme compression fiber to centroid of compression reinforcement, in. = distance from centroid of gross section, neglecting the reinforcement, to centroid of tension reinforcement, in.
= nominal diameter of bar or wire, in. = distance measured from extreme tension fiber to center of the closest bar or wire in inches. For calculation purposes, the thickness of clear concrete cover used to compute d,shall not be taken greater than 2 inches. = modulus of elasticity of concrete, psi (Article 8.7.1) = flexural stiffness of compression member (Article 8.16.5.2.7) = modulus of elasticity of reinforcement, psi (Article 8.7.2) = average bearing stress in concrete on loaded area(Artic1es 8.15.2.1.3 and8.16.7.1) = extreme fiber compressive stress in concrete at service loads (Article 8.15.2.1.1) = specified compressive strength of concrete, psi = square root of specified compressive strength of concrete, psi = average splitting tensile strength of lightweight aggregate concrete, psi = fatigue stress range in reinforcement, ksi (Article 8.16.8.3) = algebraic minimum stress level in reinforcement (Article 8.16.8.3) = modulus of rupture of concrete, psi (Article 8.15.2.1.1) 8.1.2 = tensile stress in reinforcement at service loads, psi (Article 8.15.2.2) = stress in compression reinforcement at bal- anced conditions (Articles 8.16.3.4.3 and 8.16.4.2.3) = extreme fiber tensile stress in concrete at service loads (Article 8.15.2.1.1) = specified yield strength of reinforcement, psi = overall thickness of member, in.
= compression flange thickness of I- and Tsections = moment of inertia of cracked section transformed to concrete (Article 8.13.3) = effective moment of inertia for computation of deflection (Article 8.13.3) = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement = moment of inertia of reinforcement about centroidal axis of member cross section = effective length factor for compression members (Article 8.16.5.2.3) = additional embedment length at support or at point of inflection, in. (Article 8.24.2.3) = development length, in.
(Articles 8.24 through 8.32) = development length of standard hook in tension, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook (point of tangency) plus radius of bend and one bar diameter), in. (Article 8.29) = ebb x applicable modification factor = basic development length of standard hook in tension, in. = unsupported length of compression member (Article 8.16.5.2.1) = computed moment capacity (Article 8.24.2.3) = maximum moment in member at stage for which deflection is being computed (Article 8.13.3) = nominal moment strength of a section at balanced strain conditions (Article 8.16.4.2.3) = moment to be used for design of compression member (Article 8.16.5.2.7) = cracking moment (Article 8.13.3) = nominal moment strength of a section = nominal moment strength of a section in the direction of the x axis (Article 8.16.4.3) = nominal moment strength of a section in the direction of they axis (Article 8.16.4.3) = factored moment at section DIVISION I- = factored moment component in the direction of the x axis (Article 8.16.4.3) = factored moment component in the direction of they axis (Article 8.16.4.3) = value of smaller end moment on compression member due to gravity loads that result in no appreciable sidesway calculated by conventional elastic frame analysis, positive ifmember is bent in single curvature, negative if bent in double curvature (Article 8.16.5.2.4) = value of larger end moment on compression member due to gravity loads that result in no appreciable sidesway calculated by conventional elastic frame analysis, always positive (Article 8.16.5.2.4) = value of larger end moment on compression member due to lateral loads or gravity loads that result in appreciable sidesway, defined by a deflection A, greater than eu/1500,calculated by conventional elastic frame analysis, always positive. (Article 8.16.5.2) = modular ratio of elasticity = EJE, (Article 8.15.3.4) = design axial load normal to cross section occurring simultaneously with V to be taken as positive for compression, negative for tension and to include the effects of tension due to shrinkage and creep (Articles 8.15.5.2.2 and 8.15.5.2.3) = design tensile force applied at top of bracket of corbel acting simultaneously with V, to be taken as positive for tension (Article 8.15.5.8) = factored axial load normal to the cross section occurring simultaneously with V, to be taken as positive for compression, negative for tension, and to include the effects of tension due to shrinkage and creep (Article 8.16.6.2.2) = factored tensile force applied at top of bracket or corbel acting simultaneously with V,, to be taken as positive for tension (Article 8.16.6.8) = nominal axial load strength of a section at balanced strain conditions (Article 8.16.4.2.3) = critical load (Article 8.16.5.2.7) = nominal axial load strength of a section at zero eccentricity (Article 8.16.4.2.1) = nominal axial load strength at given eccentricity = nominal axial load strength corresponding to M,, with bending considered in the direction of the x axis only (Article 8.16.4.3) pv = nominal axial load strength corresponding to M,, with bending considered in the direction of the y axis only (Article 8.16.4.3) = nominal axial load strength with biaxial loadP ~ Y ing (Article 8.16.4.3) = factored axial load at given eccentricity pu = radius of gyration of cross section of a comr pression member (Article 8.16.5.2.2) = spacing of shear reinforcement in direction s parallel to the longitudinal reinforcement, in. = spacing of wires to be developed or spliced, SW in. S = span length, ft = design shear force at section (Article V 8.15.5.1.1) v = design shear stress at section (Article 8.15.5.1.1) = nominal shear strength provided by concrete vc (Article 8.16.6.1) = permissible shear stress carried by concrete VC (Article 8.15.5.2) = design horizontal shear stress at any cross Vdh section (Article 8.15.5.5.3) = permissible horizontal shear stress (Article Vh 8.15.5.5.3) = nominal shear strength (Article 8.16.6.1) vn = nominal horizontal shear strength (Article Vi+, 8.16.6.5.3) = nominal shear strength provided by shear revs inforcement (Article 8.16.6.1) vu = factored shear force at section (Article 8.16.6.1) = weight of concrete, lb per cu ft Wc = distance from centroidal axis of gross secYt tion, neglecting reinforcement, to extreme fiber in tension (Article 8.13.3) = quantity limiting distribution of flexural reinz forcement (Article 8.16.8.4) a (alpha) = angle between inclined shear reinforcement and longitudinal axis of member (YE = angle between shear-friction reinforcement and shear plane (Articles 8.15.5.4 and 8.16.6.4) pb(beta) = ratio of area of reinforcement cut off to total area of reinforcement at the section (Article 8.24.1.4.2) = ratio of long side to short side of concentrated PC load or reaction area; for a circular concentrated load or reaction area, PC= 1.0 (Articles 8.15.5.6.3 and 8.16.6.6.2) 192 HIGHWAY BRIDGES 8.1.2 - = absolute value of ratio of maximum dead load moment to maximum total load moment, always positive = ratio of depth of equivalent compression zone to depth from fiber of maximum compressive strain to the neutral axis (Article 8.16.2.7) = correction factor related to unit weight for concrete (Articles 8.15.5.4 and 8.16.6.4) = coefficient of friction (Article 8.15.5.4.3) = tension reinforcement ratio = A,/b,d, AJbd = compression reinforcement ratio = Al/bd = reinforcement ratio producing balanced strain conditions (Article 8.16.3.1.1) = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member (Article 8.18.2.2.2) = reinforcement ratio used in Equation (8-4) and Equation (8-48) = moment magnification factor for members braced against sidesway to reflect effects of member curvature between ends of compression member = moment magnification factor for members not braced agaiiist sidesway to reflect lateral drift resulting from lateral and gravity loads = strength reduction factor (Article 8.16.1.2) 8.1.3 Definitions The following terms are defined for general use in Section 8. Specialized definitions appear in individual Articles. Bracket or corbel-Short (haunched) cantilever that projects from the face of a column or wall to support a concentrated load or beam reaction. See Articles 8.15.5.8 and 8.16.6.8. Compressive strength of concrete (f,')-Specified compressive strength of concrete in pounds per square inch (psi). Concrete, structural lightweight-A concrete containing lightweight aggregate having an air-dry unit weight as determined by "Method of Test for Unit Weight of Structural Lightweight Concrete" (ASTM C 567), not exceeding 115 pcf. In this specification, a lightweight concrete without natural sand is termed "all-lightweight concrete" and one in which all fine aggregate consists of normal weight sand is termed "sand-lightweight concrete." Deformed reinforcement-Deformed reinforcing bars, deformed wire, welded smooth wire fabric, and welded deformed wire fabric. Design load-All applicable loads and forces or their related internal moments and forces used to proportion members. For design by SERVICE LOAD DESIGN, design load refers to loads without load factors. For design by STRENGTH DESIGN METHOD, design load refers to loads multiplied by appropriate load factors. Design strength-Nominal strength multiplied by a strength reduction factor, 4. Development length-Length of embedded reinforcement required to develop the design strength of the reinforcement at a critical section. Embedment length-Length of embedded reinforcement provided beyond a critical section. Factored load-Load, multiplied by appropriate load factors, used to proportion members by the STRENGTH DESIGN METHOD. Nominal strength-Strength of a member or cross section calculated in accordance with provisions and assumptions of the STRENGTH DESIGN METHOD before application of any strength reduction factors. Plain reinforcement-Reinforcement that does not conform to the definition of deformed reinforcement. Requiredstrength--Strength of a member or cross section required to resist factored loads or related internal moments and forces in such combinations as are stipulated in Article 3.22. Service load-Loads without load factors. Spiral reinforcement-Continuously wound reinforcement in the form of a cylindrical helix. Splitting tensile strength (f,J-Tensile strength of concrete determined in accordance with "Specifications for Lightweight Aggregates for Structural Concrete," AASHTO M 195 (ASTM C 330). Stirrups or ties-Lateral reinforcement formed of individual units, open or closed, or of continuously wound reinforcement. The term "stirrups" is usually applied to lateral reinforcement in horizontal members and the term "ties" to those in vertical members. Tension tie member-Member having an axial tensile force sufficient to create tension over the entire cross section and having limited concrete cover on all sides. Examples include: arch ties, hangers carrying load to an overhead supporting structure, and main tension elements in a truss.
Yield strength or yieldpoint (f,)-Specified minimum yield strength or yield point of reinforcement in pounds per square inch. 8.2 CONCRETE The specified compressive strength, f,', of the concrete for each part of the structure shall be shown on 8.2 DIVISION I-DESIGN for fd shall be based on tests of the plans. The cylinders made and tested in accordance with Section 4-Division II. 193 8.3.3 Designs shall not use a yield strength, f,, in excess of 60,000 psi. 8.3.4 Deformed reinforcement shall be used except that plain bars or smooth wire may be used for spirals and ties. 8.3 REINFORCEMENT 8.3.1 The yield strength or grade of reinforcement shall be shown on the plans. 8.3.2 Reinforcement to be welded shall be indicated on the plans and the welding procedure to be used shall be specified. 8.3.5 Reinforcement shall conform to the specifications listed in Division 11, Section 5, except that, for reinforcing bars, the yield strength and tensile strength shall correspond to that determined by tests on full-sized bars. Part B ANALYSIS 8.4 GENERAL 8.6 STIFFNESS AU members of continuous and rigid frame structures shall be designed for the maximum effects of the loads specified in Articles 3.2 through 3.22 as determined by the theory of elastic analysis.
8.6.1 Any reasonable assumptions may be adopted for computing the relative flexural and torsional stiffnesses of continuous and rigid frame members. The assumptions made shall be consistent throughout the analysis. 8.5 EXPANSION AND CONTRACTION 8.5.1 In general, provisions for temperature changes shall be made in simple spans when the span length exceeds 40 feet. 8.5.2 In continuous bridges, the design shall provide for thermal stresses or for the accommodation of thermal movement with rockers, sliding plates, elastomeric pads, or other means. 8.5.3 The coefficient of thermal expansion and contraction for normal weight concrete may be taken as 0.000006 per deg F. 8.5.4 The coefficient of shrinkage for normal weight concrete may be taken as 0.0002. 8.5.5 Thermal and shrinkage coefficients for lightweight concrete shall be determined for the type of lightweight aggregate used. 8.6.2 The effect of haunches shall be considered both in determining moments and in design of members. 8.7 MODULUS OF ELASTICITY AND POISSON'S RATIO 8.7.1 The modulus of elasticity, E,, for concrete may be taken as wa5 33 f i in psi for values of w, between 90 and 155 pounds per cubic foot. For normal weight concrete (w, = 145 pcf), E, may be considered as 57,000fi. 8.7.2 The modulus of elasticity, E,, for nonprestressed steel reinforcement may be taken as 29,000,000 psi. 8.7.3 Poisson's ratio may be assumed as 0.2. 8.8 SPAN LENGTH 8.8.1 The span length of members that are not built integrally with their supports shall be considered the clear span plus the depth of the member but need not exceed the distance between centers of supports. 194 HIGHWAY BRIDGES 8.8.2 In analysis of continuous and rigid frame members, distances to the geometric centers of members shall be used in the determination of moments. Moments at faces of support may be used for member design. When fillets making- an angle - of 45" or more with the axis of a continuous or restrained member are built monolithic with the member and support, the face of support shall be considered at a section where the combined depth of the member and fillet is at least one and one-half times the thickness of the member. No portion of a fillet shall be considered as adding to the effective depth. 8.8.3 The effective span length of slabs shall be as specified in Article 3.24.1. TABLE 8.9.2 8.8.2 Recommended Minimum Depths for Constant Depth Members Superstructure Bridge slabs with main reinforcementparallel to traffic Minimum Depth in Feet" Simple Spans 1.2(s + 10)130 Continuous Spans (S + 10)/30 2 0.542 T-Girders 0.070s 0.065s Box-Girders 0.060s 0.055s Pedestrian Structure Girders 0.033s 0.033s When variable depth members are used, values may be adjusted to account for change in relative stiffness of positive and negative moment sections. S =span length as defined in Article 8.8 in feet. 8.9 CONTROL OF DEFLECTIONS 8.9.1 General thickness of the slab or one-half the clear distance to the next web. Flexural members of bridge structures shall be designed to have adequate stiffness to limit deflections or any deformations that may adversely affect the strength or serviceability of the structure at service load plus impact. 8.10.1.2 For girders having a slab on one side only, the flange width shall not exceed %2 of effective th, span length of the girder, six times the thickness of the slab, or one-half the clear distance to the next web. 8.9.2 Superstructure Depth Limitations 8.10.1.3 Isolated T-girders in which the T-shape is used to provide a flange for additional compression area shall have a flange thickness not less than one-half the width of the girder web and an effective flange width not more than four times the width of the girder web. The minimum depths stipulated in Table 8.9.2 are recornmended unless computation of deflection indicates that lesser depths may be used without adverse effects. 8.9.3 Superstructure Deflection Limitations When making deflection computations, the following criteria are recommended.
8.9.3.1 Members having simple or continuous spans preferably should be designed so that the deflection due to service live load plus impact shall not exceed %m of the span, except on bridges in urban areas used in part by pedestrians whereon the ratio preferably shall not exceed %m. 8.9.3.2 The deflection of cantilever arms due to service live load plus impact preferably should be limited to X m of the cantilever arm except for the case including pedestrian use, where the ratio preferably should be X75. 8.10 COMPRESSION FLANGE WIDTH 8.10.1 T-Girder 8.10.1.1 The total width of slab effective as a Tgirder flange shall not exceed one-fourth of the span length of the girder. The effective flange width overhanging on each side of the web shall not exceed six times the 8.10.1.4 For integral bent caps, the effective flange width overhanging each side of the bent cap web shall not exceed six times the least slab thickness, or YIOthe span length of the bent cap. For cantilevered bent caps, the span length shall be taken as two times the length of the cantilever span. 8.10.2 Box Girders 8.10.2.1 The entire slab width shall be assumed effective for compression. 8.10.2.2 For integral bent caps, see Article 8.10.1.4. 8.11 SLAB AND WEB THICKNESS 8.11.1 The thickness of deck slabs shall be designed in accordance with Article 3.24.3 but shall not be less than specified i n h i c l e 8.9. 8.11.2 The thickness of the bottom slab of a box girder shall be not less than YMof the clear span between girder 8.11.2 DIVISION I-DESIGN 195 webs or 5 Y2 inches, except that the thickness need not be greater than the top slab unless required by design. live loading shall be considered uniformly distributed to all longitudinal flexural members. 8.11.3 When required by design, changes in girder web thickness shall be tapered for a minimum distance of 12 times the difference in web thickness. 8.13.3 Deflections that occur immediately on application of load shall be computed by the usual methods or formulas for elastic deflections. Unless stiffness values are obtained by a more comprehensive analysis, immediate deflections shall be computed taking the modulus of elasticity for concrete as specified in Article 8.7 for normal weight or lightweigh; concrete and taking the moment of inertia as either the gross moment of inertia, Z, or the moment of inertia, I, as follows: 8.12 DIAPHRAGMS 8-12.1 Diaphragms shall be used at th' ends of '-girder and box girder spans unless other means are provided to resist lateral forces and to maintain section geometry.
Diaphragms may be omitted where tests or structural analysis show adequate strength. 8.12.2 In T-girder construction, one intermediate diaphragm is recommended at the point of maximum positive moment for spans in excess of 40 feet. 8.12.3 Straight box girder bridges and curved box girder bridges with an inside radius of 800 feet or greater do not require intermediate diaphragms. For curved box girder bridges having an inside radius less than 800 feet, intermediate diaphragms are required unless shown otherwise by tests or structural analysis.
For such curved box girders, a maximum diaphragm spacing of 40 feet is recommended to assist in resisting torsion. where: wr = Y ~ Y , (8-2) and f, = modulus of rupture of concrete specified in Article 8.15.2.1.1. For continuous members, effective moment of inertia may be taken as the average of the values obtained from Equation (8-1) for the critical positive and negative moment sections.
For prismatic members, effective moment of inertia may be taken as the value obtained from Equation (8-1) at midspan for simple or continuous spans, and as the value at the support for cantilevers. 8.13 COMPUTATION OF DEFLECTIONS 8.13.1 Computed deflections shall be based on the cross-sectional properties of the entire superstructure section excluding railings, curbs, sidewalks, or any element not placed monolithically with the superstructure section before falsework removal. 8.13.4 Unless values are obtained by a more comprehensive analysis, the long-time deflection for both normal weight and lightweight concrete flexural members shall be the immediate deflection caused by the sustained load considered, computed in accordance with Article 8.13.3, multiplied by one of the following factors: 8.13.2 Live load deflection may be based on the assumption that the superstructure flexural members act together and have equal deflection. The live loading shall consist of all traffic lanes fully loaded, with reduction in load intensity allowed as specified in Article 3.12.
The (a) Where the immediate deflection has been based on I,, the multiplication factor for the long-time deflection shall be taken as 4. (b) Where the immediate deflection has been based on L,the multiplication factor for the long-time deflection shall be taken as 3 - 1.2(A:/AS) r 1.6. Part C DESIGN 8.14 GENERAL 8.14.1 Design Methods allowable stresses as provided in SERVICE LOAD DESIGN or, alternatively, with reference to load factors and strengths as provided in STRENGTH DESIGN. 8.14.1.1 The design of reinforced concrete members shall be made either with reference to service loads and 8.14.1.2 All applicable provisions of this specification shall apply to both methods of design, except Articles 196 HIGHWAY BRIDGES 3.5 and 3.17 shall not apply for design by STRENGTH DESIGN.
8.14.1.3 The strength and serviceability requirements of STRENGTH DESIGN may be assumed to be satisfied for design by SERVICE LOAD DESIGN if the service load stresses are limited to the values given in Article 8.15.2. 8.14.2 Composite Flexural Members 8.14.2.1 Composite flexural members consist of precast and/or cast-in-place concrete elements constructed in separate placements but so interconnected that all elements respond to superimposed loads as a unit. When considered in design, shoring shall not be removed until the supported elements have developed the design properties required to support all loads and limit deflections and cracking. 8.14.2.2 The entire composite member or portions thereof may be used in resisting the shear and moment.
The individual elements shall be investigated for all critical stages of loading a d shall be designed to support all loads introduced prior to the full development of the design strength of the composite member. Reinforcement shall be provided as necessary to prevent separation of the individual elements. 8.14.2.3 If the specified strength, unit weight, or other properties of the various elements are different, the properties of the individual elements, or the most critical values, shall be used in design. 8.14.2.4 In calculating the flexural strength of a composite member by strength design, no distinction shall be made between shored and unshored members. 8.14.2.5 When an entire member is assumed to resist the vertical shear, the design shall be in accordance with the requirements of Article 8.15.5 or Article 8.16.6 as for a monolithically cast member of the same cross-sectional shape. 8.14.2.6 Shear reinforcement shall be fully anchored into the interconnected elements in accordance with Article 8.27. Extended and anchored shear reinforcementmay be included as ties for horizontal shear. 8.14.2.7 The design shall provide for full transfer of horizontal shear forces at contact surfaces of intercon- 8.14.1.2 nected elements. Design for horizontal shear shall be in accordance with the requirements of Article 8.15.5.5 or Article 8.16.6.5. 8.14.3 Concrete Arches 8.14.3.1 The combined flexure and axial load strength of an arch ring shall be in accordance with the provisions of Articles 8.16.4 and 8.16.5. Slenderness effects in the vertical plane of an arch ring, other than tied arches with suspended roadway, may be evaluated by the approximate procedure of Article 8.16.5.2 with the unsupported length, t',, taken as one-half the length of the arch ring, and the radius of gyration, r, taken about an axis perpendicular to the plane of the arch at the quarter point of the arch span. Values of the effective length factor, k, given in Table 8.14.3 may be used. In Equation (8-41), C, shall be taken as 1.0 and shall be taken as 0.85. + 8.143.2 Slenderness effects between points of lateral support and between suspenders in the vertical plane of a tied arch with suspended roadway, shall be evaluated by a rational analysis taking into account the requirements of Article 8.16.5.1.1. 8.14.3.3 The shape of arch rings shall conform, as nearly as is practicable, to the equilibrium polygon for full dead load. 8.14.3.4 In arch ribs and barrels, the longitudinal reinforcement shall provide a ratio of reinforcement area to gross concrete area at least equal to 0.01, divided equally between the intrados and the extrados. The longitudinal reinforcement shall be enclosed by lateral ties in accordance with Article 8.18.2. In arch barrels, upper and lower levels of transverse reinforcement shall be provided that are designed for transverse bending due to loads from columns and spandrel walls and for shrinkage and temperature stresses. 8.14.3.5 If transverse expansion joints are not provided in the deck slab, the effects of the combined action of the arch rib, columns and deck slab shall be considered.
Expansion joints shall be provided in spandrel walls. TABLE 8.14.3 Effective Length Factors, k - ~ Rise-to-Span Ratio 3-Hinged 2-Hinged Arch Arch Fixed Arch 0.1 - 0.2 0.2 - 0.3 0.3 - 0.4 1.16 1.13 1.16 1.04 1.10 1.16 0.70 0.70 0.72 8.14.3.6 X , &3AFWdsl DIVISION I-DESIGN exceeding 8 feet in helght on fined spandrel arches shall be laterally supported by transverse diaphragms or counterforts with a slope greater than 45 degrees with the vertical to reduce transverse stresses in the arch barrel. The top of the arch barrel and interior faces of the spandrel walls shall be waterproofed and a drainage system provided for the fill. 197 within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal. When the loaded area is subjected to high-edge stresses due to deflection or eccentric loading, the allowable bearing stress on the loaded area, including any increase due to the supporting surface being larger than the loaded area, shall be multiplied by a factor of 0.75. 8.15.2.2 Reinforcement 8.15 SERVICE LOAD DESIGN METHOD (ALLOWABLE STRESS DESIGN) The tensile stress in the reinforcement, f,, shall not exceed the following: 8.15.1 General Requirements 8.15.1.1 Service load stresses shall not exceed the values given in Article 8.15.2. 8.15.1.2 Development and splices of reinforcement shall be as required in Articles 8.24 through 8.32. 8.15.2 Allowable Stresses Grade 40 reinforcement ..... 20,000 psi Grade 60 reinforcement .....24,000 psi In straight reinforcement, the range between the maximum tensile stress and the minimum stress caused by live load plus impact shall not exceed the value given in Article 8.16.8.3. Bends in primary reinforcement shall be avoided in regions of high-stress range. 8.15.2.1 Concrete 8.15.3 Flexure Stresses in concrete shall not exceed the following: 8.15.3.1 For the investigation of stresses at service loads, the straight-line theory of stress and strain in flexure shall be used with the following assumptions. 8.15.2.1.1 Flexure Extreme fiber stress in compression, fc .....0.40ff Extreme fiber stress in tension for plain concrete, ft .....0.21fr Modulus of rupture, f,, from tests, or, if data are not available: Normal weight concrete . . . . . .
. . . . . .7.5 "Sand-lightweight" concrete .....6.3 "All-lightweight" concrete . . . . .
. . . . .
. . . .5.5 e 8.15.2.1.2 Shear For detailed summary of allowable shear stress, v,, see Article 8.15.5.2. 8.15.2.1.3 Bearing Stress The bearing stress, fb,on loaded area shall not exceed 0.30 f,'. When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the but not by loaded area may be multiplied by more than 2. When the supporting surface is sloped or stepped, A2 may be taken as the area of the lower base of the largest frustrum of the right pyramid or cone contained wholly m, 8.15.3.2 The strain in reinforcement and concrete is directly proportional to the distance from the neutral axis, except that for deep flexural members with overall depth to span ratios greater than 2/5 for continuous spans and % for simple spans, a nonlinear distribution of strain shall be considered. 8.15.3.3 In reinforced concrete members, concrete resists no tension. 8.15.3.4 The modular ratio, n = E,/E,, may be taken as the nearest whole number (but not less than 6). Except in calculations for deflections, the value of n for lightweight concrete shall be assumed to be the same as for normal weight concrete of the same strength. 8.15.3.5 In doubly reinforced flexural members, an effective modular ratio of 2E,/Ec shall be used to transform the compression reinforcement for stress computations. The compressive stress in such reinforcement shall not be greater than the allowable tensile stress. 8.15.4 Compression Members The combined flexural and axial load capacity of compression members shall be taken as 35% of that computed 198 HIGHWAY BRIDGES in accordance with the provisions of Article 8.16.4. Slenderness effects shall be included according to the requirements of Article 8.16.5. The term P, in Equation (8-41) shall be replaced by 2.5 times the design axial load. In using the provisions of Articles 8.16.4 and 8.16.5, shall be taken as 1.O. 8.15.4 taken as 0.95 fi.
A more detailed calculation of the allowable shear stress can be made using: + 8.15.5 Shear 8.15.5.1 Shear Stress 8.15.5.1.1 Design shear stress, v, shall be computed by: Note: (a) M is the design moment occurring simultaneously with V at the section being considered. (b) The quantity Vd/M shall not be taken greater than 1.0. 8.15.5.2.2 Shear in Compression Members For members subject to axial compression, the allowable shear stress carried by the concrete, v,, may be taken A more detailed calculation can be made as 0.95 using: e. where V is design shear force at section considered, b, is the width of web, and d is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement.
Whenever applicable, effects of torsion* shall be included. 8.15.5.1.2 For a circular section, b, shall be the diameter and d need not be less than the distance from the extreme compression fiber to the centroid of the longitudinal reinforcement in the opposite half of the member. 8.15.5.1.3 For tapered webs, b, shall be the average width or 1.2 times the minimum width, whichever is smaller. 8.15.5.1.4 When the reaction, in the direction of the applied shear, introduces compression into the end regions of a member, sections located less than a distance d from the face of support may be designed for the same shear, V, as that computed at a distance d. An exception occurs when major concentrated loads are imposed between that point and the face of support. In that case sections closer than d to the support shall be designed for V at distance d plus the major concentrated loads. 8.15.5.2 Shear Stress Carried by Concrete , .8.15.5.2.1 Shear in Beams and One-Way Slabs and Footings For members subject to shear and flexure only, the allowable shear stress carried by the concrete, v,, may be *The design criteria for combined torsion and shear givenin "Building Code Requirements for Reinforced Concrete"-American Concrete Institute 318 Bulletin may be used. The quantity NIA, shall be expressed in pounds per square inch. 8.15.5.2.3 Shear in Tension Members For members subject to axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using Note: (a) N is negative for tension. (b) The quantity NIA, shall be expressed in pounds per square inch. 8.15.5.2.4 Shear in Lightweight Concrete The provisions for shear stress, v,, carried by the concrete apply to normal weight concrete. When lightweight aggregate concretes are used, one of the following modifications shall apply: (a) When f,, is specified, the shear stress, v,, shall be modified by substituting fJ6.7 for @,but the value of fJ6.7 used shall not exceed (b) When f, is not specified, the shear stress, v,, shall be multiplied by 0.75 for "all-lightweight" concrete, and e. 8.15.5.2.4 DIVISION I-DESIGN 199 0 . 8 5 - f o ~ ~ h ~ f l i r Linear ~ C CinterpoiaI . tion may be used when partial sand replacement is used.
8.15.5.3.9 The value of (v - v,) shall not exceed 4 fl. Shear Stress Carried by Shear Reinforcement 8.15.5.3.10 When flexural reinforcement located within the width of a member used to compute the shear strength is terminated in a tension zone, shear reinforcement shall be provided in accordance with Article 8.24.1.4. 8.15.5.3.1 Where design shear stress v exceeds shear stress carried by concrete, v,, shear reinforcement shall be provided in accordance with this article. Shear reinforcement shall also conform to the general requirements of Article 8.19.
8.15.5.3.2 When shear reinforcement perpendicular to the axis of the member is used: 8.15.5.3.3 When inclined stirrups are used: A, = (V- vc)b,s fs(sin a + cos a) - (8 8) 8.15.5.3.4 When shear reinforcement consists of a single bar or a single group of parallel bars all bent up at the same distance from the support: A, = (v-vc)bwd fssin a where (v-v,) shall not exceed 1.5 8.15.5.4 Shear Friction 8.15.5.4.1 Provisions for shear-friction are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times. 8.15.5.4.2 A crack shall be assumed to occur along the shear plane considered.
Required area of shear-friction reinforcement A~ across the shear plane may be designed using either Article 8.15.5.4.3 or any other shear transfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests. Provisions of Articles 8.15.5.4.4 through 8.15.5.4.8 shall apply for all calculations of shear transfer strength. 8.15.5.4.3 Shear-friction Design Method (a) When shear-friction reinforcement is perpendicular to the shear plane, area of shear-friction reinforcement A~ shall be computed by: e. 8.15.5.3.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bentup bars at different distances from the support, the required area shall be computed by Equation (8-8). 8.15.5.3.6 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement. 8.15.5.3.7 Where more than one type of shear reinforcement is used to reinforce the same portion of the member, the required area shall be computed as the sum of the values computed for the various types separately. In such computations, v, shall be included only once. 8.15.5.3.8 When (v - v,) exceeds 2 flthe maximum spacings given in Article 8.19 shall be reduced by one-half. A,=- v (8 - 10) fSP where pis the coefficient of friction in accordance with Article 8.15.5.4.3(~). (b) When shear-friction reinforcement is inclined to the shear plane such that the shear force produces tension in shear-friction reinforcement, the area of shearfriction reinforcement Ad shall be computed by: A, = v fs(ysin af+ cosaf) (8-11) where afis the angle between the shear-friction reinforcement and the shear plane. (c) Coefficient of friction p in Equations (8-10) and (8-11) shall be: concrete placed monolithically . . . . . .
. .
. .1.4A concrete placed against hardened concrete with surface intentionally roughened as specified in Article 8.15.5.4.7 . . . . . .
. . . . . .
.l.OA 200 HIGHWAY BRIDGES concrete placed against hardened concrete not intentionally roughened . . . . .
. .
. . . . . .0.6X concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see Article 8.15.5.4.8 .....0.7X where X = 1.0

0.03h) inches, the computed end moments may be used to evaluate MldMZbin Equation (8-45). (b) If computations show that there is essentially no moment at either end of the member, the ratio Ml&IZb shall be equal to one. 1 1 6, = I-- xpu 21.0 (8 - 41A) and n2~1 PC= - (8 - 42) For members braced against sidesway, 6, shall be taken as 1.O. For members not braced against sidesway, 6, shall be evaluated as for a braced member and 6, for an unbraced member. In lieu of a more precise calculation, EI may be taken either as ECI, EI = 5 -+ ESIs (8 - 43) 8.16.5.2.9 In structures that are not braced against sidesway, the flexural members framing into the compression member shall be designed for the total magnified end moments of the compression member at the joint. 8.16.5.2.10 When compression members are subject to bending about both principal axes, the moment about each axis shall be magnified by 6, computed from the corresponding conditions of restraint about that axis. 8.16.5.2.11 When a group of compression members on one level comprise a bent, or when they are connected integrally to the same superstructure, and collectively resist the sidesway of the structure, the value of 6s shall be computed for the member group with ZP, and ZP, equal to the summations for all columns in the group. 8.16.6 Shear 8.16.6.1 Shear Strength l+pd 8.16.6.1.1 Design of cross sections subject to shear shall be based on or conservatively as ECI, 2.5 EI = 1+Pd v, 5 4)vn where V, is the factored shear force at the section considered and V, is the nominal shear strength computed by, where Pdis the ratio of maximum dead load moment to maximum total load moment and is always positive. For members braced against sidesway and without transverse loads between supports, C, may be taken as C, = 0.6 + 0.4 (Ml&b) (8-46) (8 - 44) (8-45) but not less than 0.4. For all other cases, C, shall be taken as 1.O. 8.16.5.2.8 If computations show that there is no moment at either end of a compression member braced or unbraced against sidesway or that computed end eccentricities are less than (0.6 + 0.03h) inches, MZband Ma in Equation (8-40) shall be based on a minimum eccentricity of (0.6 + 0.03h) inches about each principal axis separately. The ratio Ml&IZbin Equation (8-45) shall be determined by either of the following: V, = V, + V, (8-47) where V, is the nominal shear strength provided by the concrete in accordance with Article 8.16.6.2, and Vsis the nominal shear strength provided by the shear reinforcement in accordance with Article 8.16.6.3. Whenever applicable, effects of torsion* shall be included. 8.16.6.1.2 When the reaction, in the direction of applied shear, introduces compression into the end regions of a member, sections located less than a distance d from the face of support may be designed for the same shear, V,, as that computed at a distance d. An exception occurs when major concentrated loads are imposed between that point and the face of support. In that case, sections closer *The design criteria for combined torsion and shear given in "Building Code Requirementsfor Reinforced Concrete" ACI 318 may be used. 208 HIGHWAY BRIDGES 8.16.6.1.2 than d to the support shall be designed for V at a distance d plus the major concentrated loads. 8.16.6.2 Shear Strength Provided by Concrete 8.16.6.2.1 Shear in Beams and One-Way Slabs and Footings For members subject to shear and flexure only, V, shall be computed by, Note: (a) Nuis negative for tension. (b) The quantity N,/A, shall be expressed in pounds per square inch. 8.16.6.2.4 Shear in Lightweight Concrete The provisions for shear stress, v,, carried by the concrete apply to normal weight concrete. When lightweight aggregate concretes are used, one of the following modifications shall apply: (a) When f,, is specified, the shear strength, V,, shall be modified by substituting fJ6.7 for but the value of fJ6.7 used shall not exceed *.

(b) When f,, is not specified, V, shall be multiplied by 0.75 for "all lightweight" concrete, and 0.85 for "sandlightweight" concrete. Linear interpolation may be used when partial sand replacement is used. fl, where b, is the width of web and d is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement. Whenever applicable,effects of torsion shall be included. For a circular section, b, shall be the diameter and d need not be less than the distance from the extreme compression fiber to the centroid of the longitudinal reinforcement in the opposite half of the member.

For tapered webs, b, shall be the average width or 1.2 times the minimum width, whichever is smaller. Note: (a) V, shall not exceed 3.5 bwdwhen using more detailed calculations. (b) The quantity Vud/M,shall not be greater than 1.0 where Muis the factored moment occurring simultaneously with Vuat the section being considered. fl 8.16.6.3 Shear Strength Provided by Shear Reinforcement 8.16.6.3.1 Where factored shear force Vu exceeds shear strength +Vc, shear reinforcement shall be provided to satisfy Equations (8-46) and (8-47), but not less than that required by Article 8.19. Shear strength V, shall be computed in accordance with Articles 8.16.6.3.2 through 8.16.6.3.10. 8.16.6.3.2 When shear reinforcement perpendicular to the axis of the member is used: 8.16.6.2.2 Shear in Compression Members For members subject to axial compression, V, may be computed by: where A, is the area of shear reinforcement within a distance s. or, V, = 2 *b,d (8-51) Note: The quantity Nu/A, shall be expressed in pounds per square inch. 8.16.6.2.3 Shear in Tension Members For members subject to axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using: 8.16.6.3.3 When inclined stirrups are used: V, = A, fy (sin a + cos a)d (8 - 54) S 8.16.6.3.4 When a single bar or a single group of parallel bars all bent up at the same distance from the support is used: V, = A,f, sin a 3~ b , d (8-55) 8.16.6.3.5 DIVISION I-DESIGN 8.16.6.3.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bentup bars at different distances from the support, shear strength V, shall be computed by Equation (8-54). 8.16.6.3.6 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement. 8.16.6.3.7 Where more than one type of shear reinforcement is used to reinforce the same portion of the member, shear strength V, shall be computed as the sum of the V, values computed for the various types. 8.16.6.3.8 When shear strength V, exceeds 4 fl b,d, spacing of shear reinforcement shall not exceed onehalf the maximum spacing given in Article 8.19.3. 8.16.6.3.9 Shear strength V, shall not be taken greater than 8 flb,d. 8.16.6.3.10 When flexural reinforcement, located within the width of a member used to compute the shear strength, is terminated in a tension zone, shear reinforcement shall be provided in accordance with Article 8.24.1.4. 8.16.6.4 Shear Friction 8.16.6.4.1 Provisions for shear-friction are to be applied where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times. 8.16.6.4.2 Design of cross sections subject to shear transfer as described in Article 8.16.6.4.1 shall be based on Equation (8-46), where shear strength V, is calculated in accordance with provisions of Article 8.16.6.4.3 or 8.16.6.4.4. 8.16.6.4.3 A crack shall be assumed to occur along the shear plane considered. Required area of shear-friction reinforcement A,+across the shear plane may be designed using either Article 8.16.6.4.4 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests. Provisions of Articles 8.16.6.4.5 through 8.16.6.4.9 shall apply for all calculations of shear transfer strength. 8.16.6.4.4 Shear-Friction Design Method (a) When the shear-friction reinforcement is perpendicular to the shear plane, shear strength, V,, shall be computed by: 209 where y is the coefficient of friction in accordance with Article (c). (b) When the shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in shear-friction reinforcement, shear strength V, shall be computed by: V, = A,+fy(p sin af+ cos af) (8-5614) where afis the angle between the shear-friction reinforcement and the shear plane. (c) Coefficient of friction y in Equations (8-56) and (8-56A) shall be: Concrete placed monolithically ..... .1.41 Concrete placed against hardened concrete with surface intentionally roughened as specified in Article 8.16.6.4.8 . .

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. . . .l.OX Concrete placed against hardened concrete not intentionally roughened .....0.61 Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see Article 8.16.6.4.9) ..... .0.71 where X = 1.0 for normal weight concrete; 0.85 for "sand lightweight" concrete; and 0.75 for "all lightweight" concrete. Linear interpolation may be applied when partial sand replacement is used. 8.16.6.4.5 Shear strength V, shall not be taken greater than 0.2f,' 4, nor 800 &, in pounds, where &, is the area of the concrete section resisting shear transfer. 8.16.6.4.6 Net tension across the shear plane shall be resisted by additional reinforcement. Permanent net compression across the shear plane may be taken as additive to the force in the shear-friction reinforcement, Adfy, when calculating required A,+. 8.16.6.4.7 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop the specified yield strength on both sides by embedment, hooks, or welding to special devices. 8.16.6.4.8 For the purpose of Article 8.16.6.4, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If y is assumed equal to 1.01, the interface shall be roughened to a full amplitude of approximately % inch. 210 HIGHWAY BRIDGES 8.16.6.4.9 When shear is transferred between asrolled steel and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint. 8.16.6.5 Horizontal Shear Strength for Composite Concrete Flexural Members 8.16.6.5.1 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

8.16.6.5.2 Design of cross sections subject to horizontal shear may be in accordance with provisions of Article 8.16.6.5.3 or 8.16.6.5.4, or any other sheartransfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests. 8.16.6.5.3 Design of cross sections subject to horizontal shear may be based on: where V, is the factored shear force at the section considered, V* is the nominal horizontal shear strength in accordance with the following, and where d is for the entire composite section.

(a) When contact surface is clean, free of laitance, and intentionally roughened, shear strength V, shall not be taken greater than 80bvd,in pounds. (b) When minimum ties are provided in accordance with Article 8.16.6.5.5, and contact surface is clean and free of laitance, but not intentionally roughened, shear strength V, shall not be taken greater than 80 bvd,in - (c) When minimum ties are provided in accordance with Article 8.16.6.5.5, and contract surface is clean, free of laitance, and intentionally roughened to a full amplitude of approximately Xi inch, shear strength Vnh shall not be taken greater than 350b,d, in pounds. (d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by Article 8.16.6.5.5, shear strength V* may be increased by (160f,/40,000)bvd,in pounds. 8.16.6.5.4 Horizontal shear may be investigated by computing, in any segment not exceeding one-tenth of the span, the change in compressive or tensile force to be transferred, and provisions made to transfer that force as horizontal shear between interconnected elements. The factored horizontal shear force shall not exceed horizon- 8.16.6.4.9 tal shear strength +V, in accordance with Article 8.16.6.5.3, except that the length of the segment considered shall be substituted for d. 8.16.6.5.5 Ties for Horizontal Shear (a) When required, a minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than 50b,s/fy, and tie spacing, s, shall not exceed four times the least web width of the support element, nor 24 inches. (b) Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric. All ties shall be adequately anchored into interconnected elements by embedment or hooks. 8.16.6.6 Special Provisions for Slabs and Footings 8.16.6.6.1 Shear strength of slabs and footings in the vicinity of concentrated loads or reactions shall be governed by the more severe of two conditions: (a) Beam action for the slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from the face of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 8.16.6.1 through 8.16.6.3 except at footings supported on piles, the shear on the critical section shall be determined in accordance with Article 4.4.11.3. (b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the member and located so that its perimeter b, is a minimum, but need not approach closer than d/2 to the perimeter of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Articles 8.16.6.6.2 and 8.16.6.6.3. 8.16.6.6.2 Design of slab or footing for two-way action shall be based on Equation (8-46), where shear strength V, shall not be taken greater than shear strength V, given by Equation (8-58), unless shear reinforcement is provided in accordance with Article 8.16.6.6.3. p, is the ratio of long side to short side of concentrated load or reaction area, and b, is the perimeter of the critical section defined in Article 8.16.6.6.1(b). 8.16.6.6.3 DIVISION I-DESIGN 8.16.6.6.3 Shear reinforcement consisting of bars or wires may be used in slabs and footings in accordance with the following provisions: (a) Shear strength V, shall be computed by Equation (8-47), where shear strength V, shall be in accordance with paragraph (d) and shear strength V, shall be in accordance with paragraph (e). (b) Shear strength shall be investigated at the critical section defined in Article 8.16.6.6.1(b), and at successive sections more distant from the support. (c) Shear strength V, shall not be taken greater than 6 *bod, where b, is the perimeter of the critical section defined in paragraph (b). (d) Shear strength V, at any section shall not be taken where b, is the perimeter of the greater than 2 *bod, critical section defined in paragraph (b). (e) Where the factored shear force V, exceeds the shear strength +Vcas given in paragraph (d), the required area A, and shear strength V, of shear reinforcementshall be calculated in accordance with Article 8.16.6.3. 8.16.6.7 Special Provisions for Slabs of Box Culverts 8.16.6.7.1 For slabs of box culverts under 2 feet or more fill, shear strength Vc may be computed by: (V,= 2 . 1 4 f i + 4 , 6 0 0 pM y)ub d 211 8.16.6.8.2 Depth at the outside edge of bearing area shall not be less than 0.5d. 8.16.6.8.3 The section at the face of the support shall be designed to resist simultaneously a shear V,, a moment (Vuav+ Nu, (h - d)), and a horizontal tensile force Nu,. Distance h shall be measured at the face of support. (a) In all design calculations in accordance with Article 8.16.6.8, the strength reduction factor shall be taken equal to 0.85. (b) Design of shear-friction reinforcement Ad to resist shear Vu shall be in accordance with Article 8.16.6.4. For normal weight concrete, shear strength V, shall not be taken greater than 0.2f,'bwd nor 800bwdin pounds. For "all lightweight" or "sand-lightweight" concrete, shear strength V, shall not be taken greater than (0.2 0.07av/d)f,'b,d nor (800 - 280av/d)b,d in pounds. (c) Reinforcement Af to resist moment (V,a, + Nu, (h - d)) shall be computed in accordance with Articles 8.16.2 and 8.16.3. (d) Reinforcement A, to resist tensile force Nu, shall be determined from Nu, 5 +A,f,. Tensile force Nu, shall not be taken less than 0.2Vuunless special provisions are made to avoid tensileforces. Tensileforce Nu, shall be regarded as a live load even when tension results from creep, shrinkage, or temperature change. (e) Area of primary tension reinforcement A, shall be made equal to the greater of (Af+ A,J or: + (8-59) *+an. a 3 but V, shall not exceed 4 bd. For single cell box culverts only, V, for slabs monolithic with walls need not be bd, and Vc for slabs simply supported on walls need not be taken less than 2.5 fibd. The quantity V,dM, shall not be taken greater than 1.0 where Mu is the factored moment occurring simultaneously with V, at the section considered. For slabs of box culverts under less than 2 feet of fill, applicable provisions of Articles 3.24 and 6.4 should be used. * 8.16.6.8 Special Provisions for Brackets and Corbels* 8.16.6.8.1 Provisions of Article 8.16.6.8 shall apply to brackets and corbels with a shear span-to-depth ratio aJd not greater than unity, and subject to a horizontal tensile force Nucnot larger than V,. Distance d shall be measured at the face of support. Framing bar to anchor J stirrups or tles *These provisions do not apply to beam ledges. The PCA publication, "Notes on ACI 318-83" contains an example design of beam ledgesPart 16, example 16-3. FIGURE 8.16.6.8 212 HIGHWAY BRIDGES 8.16.6.8.4 Closed stinups or ties parallel to A,, with a total area Ah not less than 0.5(AS- A,), shall be uniformly distributed within two-thirds of the effective depth adjacent to A,. 8.16.6.8.5 O.04(ff/fy).

Ratio p = AJbd shall not be less than 8.16.6.8.6 At front face of bracket or corbel, primary tension reinforcement A, shall be anchored by one of the following: (a) a structural weld to a transverse bar of at least equal size; weld to be designed to develop specified yield strength f, of A, bars, (b) bending primary tension bars A, back to form a horizontal loop, or (c) some other means of positive anchorage. 8.16.6.8.7 Bearing area of load on bracket or corbel shall not project beyond straight portion of primary tension bars A,, nor project beyond interior face of transverse anchor bar (if one is provided).

8.16.7 Bearing Strength 8.16.6.8.4 at service load shall be limited to satisfy the requirements for fatigue in Article 8.16.8.3, and for distribution of reinforcement in Article 8.16.8.4. The requirements for control of deflections in Article 8.9 shall also be satisfied. 8.16.8.2 Service Load Stresses For investigation of stresses at service loads to satisfy the requirements of Articles 8.16.8.3 and 8.16.8.4, the straight-line theory of stress and strain in flexure shall be used and the assumptions given in Article 8.15.3 shall apply. 8.16.8.3 Fatigue Stress Limits The range between a maximum tensile stress and minimum stress in straight reinforcement caused by live load plus impact at service load shall not exceed: where: ff = stress range in kips per square inch; f- = algebraic minimum stress level, tension positive, compression negative in kips per square inch; 8.16.7.1 The,bearing stress, fb,on concrete shall not exceed 0.85+ f,' except as provided in Articles 8.16.7.2, 8.16.7.3, and 8.16.7.4. 8.16.7.2 When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be multiplied by but not by more than 2. m, 8.16.7.3 When the supporting surface is sloped or stepped, Az may be taken as the area of the lower base of the largest frustum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal. 8.16.7.4 When the loaded area is subjected to high edge stresses due to deflection or eccentric loading, the allowable bearing stress on the loaded area, including any increase due to the supporting surface being larger than the loaded area, shall be multiplied by a factor of 0.75. 8.16.8 Serviceability Requirements 8.16.8.1 r/h = ratio of base radius to height of rolled-on transverse deformations; when the actual value is not known, use 0.3. Bends in primary reinforcement shall be avoided in regions of high stress range. Fatigue stress limits need not be considered for concrete deck slabs with primary reinforcement perpendicular to traffic and designed in accordance with the approximate methods given under Article 3.24.3, Case A. Fatigue stress limits for welded splices and mechanical connections that are subjected to repetitive loads shall conform with the requirements of Article 8.32.2.5. 8.16.8.4 Distribution of Flexural Reinforcement To control flexural cracking of the concrete, tension reinforcement shall be well distributed within maximum flexural zones. When the design yield strength, f,,, for tension reinforcement exceeds 40,000 psi, the bar sizes and spacing at maximum positive and negative moment sections shall be chosen so that the calculated stress in the reinforcement at service load f,, in ksi does not exceed the value computed by: Application For flexural members designed with reference to load factors and strengths by Strength Design Method, stresses where: 8.16.8.4 DIVISION I-DESIGN A = effective tension area, in square inches, of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires. When the flexural reinforcement consists of several bar or wire sizes, the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used. For calculation purposes, the thickness of clear concrete cover used to compute A shall not be taken greater than 2 in. d, = distance measured from extreme tension fiber to center of the closest bar or wire in inches. For 213 calculation purposes, the thickness of: clear concrete cover used to compute d, shall not be taken greater than 2 inches. The quantity z in Equation (8-61) shall not exceed 170 kips per inch for members in moderate exposure conditions and 130 kips per inch for members in severe exposure conditions.

Where members are exposed to very aggressive exposure or corrosive environments, such as deicer chemicals, protection should be provided by increasing the denseness or imperviousness to water or furnishing other protection such as a waterproofing protecting system, in addition to satisfying Equation (8-61). Part D REINFORCEMENT 8.17 REINFORCEMENT OF FLEXURAL MEMBERS not less than 0.4% of the excess slab area shall be provided in the excess portions of the slab. 8.17.1 Minimum Reinforcement 8.17.2.1.2 For integral bent caps of T-girder and boxgirder construction, tension reinforcement shall be placed within a width not to exceed the web width plus an overhanging slab width on each side of the bent cap web equal to one-fourth the average spacing of the intersecting girder webs or a width as defined in Article 8.10.1.4 for integral bent caps, whichever is smaller. 8.17.1.1 At any section of a flexural member where tension reinforcement is required by analysis, the reinforcement provided shall be adequate to develop a moment at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture for normal weight concrete specified in Article 8.15.2.1.1. 8.17.1.2 The requirements ofArticle 8.17.1.1 may be waived if the area of reinforcement provided at a section is at least one-third greater than that required by analysis based on the loading combinations specified in Article 3.22. 8.17.2 Distribution of Reinforcement 8.17.2.1 Flexural Tension Reinforcement in Zones of Maximum Tension 8.17.2.1.1 Where flanges of T-girders and box-girders are in tension, tension reinforcement shall be distributed over an effective tension flange width equal to onetenth the girder span length or a width as defined in Article 8.10.1, whichever is smaller. If the actual slab width, center-to-center of girder webs, exceeds the effective tension flange width, and for excess portions of the deck slab overhang, additional longitudinal reinforcement with area 8.17.2.1.3 If the depth of the side face of a member exceeds 3 feet, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance d/2 nearest the flexural tension reinforcement. The area of skin reinforcement Askper foot of height on each side face shall be r 0.012 (d - 30). The maximum spacing of skin reinforcement shall not exceed the lesser of d/6 and 12 inches. Such reinforcement may be included in strength computations if a strain compatibility analysis is made to determine stresses in the individual bars or wires. The total area of longitudinal skin reinforcement in both faces need not exceed one-half of the required flexural tensile reinforcement. 8.17.2.2 Transverse Deck Slab Reinforcement in T-Girders and Box Girders At least one-third of the bottom layer of the transverse reinforcement in the deck slab shall extend to the exterior face of the outside girder web in each group and be an- HIGHWAY BRIDGES - chored by a standard 90" hook. If the slab extends beyond the last girder web, such reinforcement shall extend into the slab overhang and shall have an anchorage beyond the exterior face of the girder web not less than that provided by a standard hook. 8.17.2.3 Bottom Slab Reinforcement for Box Girders 8.17.2.3.1 Minimum distributed reinforcement of 0.4% of the flange area shall be placed in the bottom slab parallel to the girder span.

A single layer of reinforcement may be provided. The spacing of such reinforcementshall not exceed 18 inches. 8.17.2.3.2 Minimum distributed reinforcement of 0.5% of the cross-sectional area of the slab, based on the least slab thickness, shall be placed in the bottom slab transverse to the girder span. Such reinforcement shall be distributed over both surfaces with a maximum spacing of 18 inches. All transversereinforcement in the bottom slab shall extend to the exterior face of the outside girder web in each group and be anchored by a standard 90" hook. 8.17.3 Lateral Reinforcement of Flexural Members 8.17.3.1 Compression reinforcement used to increase the strength of flexural members shall be enclosed by ties or stirrups which shall be at least No. 3 in size for longitudinal bars that are No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, and bundled longitudinal bars.

Welded wire fabric of equivalent area may be used instead of bars. The spacing of ties shall not exceed 16 longitudinal bar diameters. Such stirrups or ties shall be provided throughout the distance where the compression reinforcement is required. This paragraph does not apply to reinforcement located in a compression zone which has not been considered as compression reinforcement in the design of the member. 8.17.3.2 Torsion reinforcement, where required, shall consist of closed stirrups, closed ties, or spirals, combined withlongitudinalbars. SeeArticle8.15.5.1.1 or8.16.6.1.1. 8.17.3.3 Closed stirrups or ties may be formed in one piece by overlapping the standard end hooks of ties or stirrups around a longitudinal bar, or may be formed in one or two pieces by splicing with Class C splices (lap of 1.7 td). 8.17.3.4 In seismic areas, where an earthquake that could cause major damage to construction has a high probability of occurrence, lateral reinforcement shall be designed and detailed to provide adequate strength and ductility to resist expected seismic movements. 8.17.4 Reinforcement for Hollow Rectangular Compression Members 8.17.4.1 The area of longitudinal reinforcement in the cross section shall not be less than 0.01 times the gross area of concrete in the cross section. 8.17.4.2 Two layers of reinforcement shall be provided in each wall of the cross section, one layer near each face of the wall. The areas of reinforcementin the two layers shall be approximately equal. 8.17.4.3 The center-to-center lateral spacing of longitudinal reinforcing bars shall be no greater than 1.5 times the wall thickness, or 18 inches, whichever is less. 8.17.4.4 The center-to-center longitudinal spacing of lateral reinforcing bars shall be no greater than 1.25 times the wall thickness, or 12 inches, whichever is less. 8.17.4.5 Cross ties shall be provided between layers of reinforcement in each wall. The cross ties shall include a standard 135" hook at one end, and a standard 90" hook at the other end. Cross ties shall be located at bar grid intersections, and the hooks of all ties shall enclose both lateral and longitudinal bars at the intersections. Each longitudinal reinforcing bar and each lateral reinforcing bar shall be enclosed by the hook of a cross tie at a spacing not to exceed 24 inches. 8.17.4.6 For segmentally constructed members, additional cross ties shall be provided along the top and bottom edges of each segment. The cross ties shall be placed so as to link the ends of each pair of internal and external longitudinal reinforcing bars in the walls of the cross section. 8.17.4.7 Lateral reinforcing bars may be joined at the comers of the cross section by overlapping 90" bends. Straight lap splices of lateral reinforcing bars are not permitted unless the overlapping bars are enclosed over the length of the splice by the hooks of at least four cross ties located at intersections of the lateral bars and longitudinal bars. 8.17.4.8 When details permit, the longitudinal reinforcing bars in the comers of the cross section shall be enclosed by closed hoops.

If closed hoops cannot be provided, then pairs of " U shaped bars with legs at least twice as long as the wall thickness, and orientated 90" to one another, may be substituted. 8.17.4.9 DIVISION I-DESIGN 215 8.17.4.9 Post-tensioning ducts located in the corners of the cross section shall be anchored into the comer regions with closed hoops, or by stirrups having a 90" bend at each end which encloses at least one longitudinal bar near the outer face of the cross section. where f, is the specified yield strength of spiral reinforcement but not more than 60,000 psi. 8.18 REINFORCEMENT OF COMPRESSION MEMBERS 8.18.2.2.4 Anchorage of spiral reinforcement shall be provided by 1 Y2 extra turns of spiral bar or wire at each end of a spiral unit. 8.18.1 Maximum and Minimum Longitudinal Reinforcement 8.18.1.1 The area of longitudinal reinforcement for compression members shall not exceed 0.08 times the gross area, A,, of the section. 8.18.1.2 The minimum area of longitudinal reinforcement shall not be less than 0.01 times the gross area, A,, of the section. When the cross section is larger than that required by consideration of loading, a reduced effective area may be used. The reduced effective area shall not be less than that which would require 1% of longitudinal reinforcement to carry the loading. The minimum number of longitudinal reinforcing bars shall be six for bars in a circular arrangement and four for bars in a rectangular arrangement.

The minimum size of bars shall be No. 5. 8.18.2 Lateral Reinforcement 8.18.2.2.3 The clear spacing between spirals shall not exceed 3 inches or be less than 1 inch or 1% times the maximum size of coarse aggregate used. 8.18.2.2.5 Spirals shall extend from top of footing or other support to the level of the lowest horizontal reinforcement in members supported above. 8.18.2.2.6 Splices in spiral reinforcement shall be lap splices of 48 bar or wire diameters but not less than 12 inches, or shall be welded. 8.18.2.2.7 Spirals shall be of such size and so assembled to permit handling and placing without distortion from designed dimensions. 8.18.2.2.8 Spirals shall be held firmly in place by attachment to the longitudinalreinforcement and true to line by vertical spacers. 8.18.2.3 Ties Tie reinforcement for compression members shall conform to the following: 8.18.2.1 General All bars shall be enclosed by lateral ties which shall be at least No. 3 in size for longitudinal bars that are No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, and bundled longitudinal bars. Deformed wire or welded wire fabric of equivalent area may be used instead of bars. 8.18.2.3.1 In a compression member that has a larger cross section than that required by conditions of loading, the lateral reinforcement requirements may be waived where structurd analysis or tests show adequate strength and feasibility of construction. 8.18.2.2 Spirals 8.18.2.3.2 Spiral reinforcement for compression members shall conform to the following: 8.18.2.2.1 Spirals shall consist of evenly spaced continuous bar or wire, with a minimum diameter of 3/8 inch.

8.18.2.2.2 The ratio of spiral reinforcement to total volume of core, p,, shall not be less than the value given by: I (8 - 63) The spacing of ties shall not exceed the least dimension of the compression member or 12 inches. When two or more bars larger than No. 10 are bundled together, tie spacing shall be one-half that specified above. 8.18.2.3.3 Ties shall be located not more than half a tie spacing from the face of a footingor from the nearest longitudinal reinforcement of a cross-framing member. 8.18.2.3.4 No longitudinal bar shall be more than 2 feet, measured along the tie, from a restrained bar on either side. A restrained bar is one which has lateral support provided by the comer of a tie having an included angle of not more than 135". Where longitudinal bars are lo- HIGHWAY BRIDGES 216 cated around the perimeter of a circle, a complete circular tie may be used. 8.18.2.3.4 (d) Combinations of stirrups and bent longitudinal reinforcement. (e) Spirals. 8.18.2.4 Seismic Requirements In seismic areas, where an earthquake which could cause major damage to construction has a high probability of occurrence, lateral reinforcement for column piers shall be designed and detailed to provide adequate strength and ductility to resist expected seismic movements. 8.19 LIMITS FOR SHEAR REINFORCEMENT 8.19.1 Minimum Shear Reinforcement 8.19.1.1 A minimum area of shear reinforcement shall be provided in all flexural members, except slabs and footings, where (a) For design by Strength Design, factored shear force V, exceeds one-half the shear strength provided by concrete +V,. 8.19.2.2 Shear reinforcement shall be developed at both ends in accordance with the requirements of Article 8.27.

8.19.3 Spacing of Shear Reinforcement Spacing of shear reinforcement placed perpendicular to the axis of the member shall not exceed d 2 or 24 inches. Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45" line extending toward the reaction from the mid-depth of the member, d 2 , to the longitudinal tension reinforcement shall be crossed by at least one line of shear reinforcement. 8.20 SHRINKAGE AND TEMPERATURE REINFORCEMENT (b) For design by Service Load Design, design shear stress v exceeds one-half the permissible shear stress carried by concrete v,. 8.20.1 Reinforcement for shrinkage and temperature stresses shall be provided near exposed surfaces of walls and slabs not otherwise reinforced. The total area of reinforcement provided shall be at least K square inch per foot in each direction. 8.19.1.2 Where shear reinforcement is required by Article 8.19.1.1, or by analysis, the area provided shall not be less than: 8.20.2 The spacing of shrinkage and temperature reinforcement shall not exceed three times the wall or slab thickness, or 18 inches. A,=- 50b,s (8 - 64) 8.21 SPACING LIMITS FOR REINFORCEMENT fY where b, and s are in inches. 8.19.1.3 Minimum shear reinforcement requirements may be waived if it is shown by test that the required ultimate flexural and shear capacity can be developed when shear reinforcement is omitted.

8.19.2 Types of Shear Reinforcement 8.19.2.1 Shear reinforcement may consist of: (a) Stirmps perpendicular to the axis of the member or making an angle of 45" or more with the longitudinal tension reinforcement. (b) Welded wire fabric with wires located perpendicular to the axis of the member. (c) Longitudinal reinforcement with a bent portion making an angle of 30" or more with the longitudinal tension reinforcement. 8.21.1 For cast-in-place concrete the clear distance between parallel bars in a layer shall not be less than 1.5 bar diameters, 1.5 times the maximum size of the coarse aggregate, or 1%inches. 8.21.2 For precast concrete (manufactured under plant control conditions) the clear distance between parallel bars in a layer shall be not less than 1 bar diameter, 1% times the maximum size of the coarse aggregate, or 1 inch. 8.21.3 Where positive or negative reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above those in the bottom layer with the clear distance between layers not less than 1 inch. 8.21.4 The clear distance limitation between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars. 8.21.4 DIVISION I-DESIGN 8.21.5 Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to 4 in any one bundle. Bars larger than No. 11 shall be limited to two in any one bundle in beams. Bundled bars shall be located within stirrups or ties. Individual bars in a bundle cut off within the span of a member shall terminate at points at least 40-bar diameters apart. Where spacing limitations are based on bar diameter, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area. 8.21.6 In walls and slabs the primary flexural reinforcement shall be spaced not farther apart than 1.5 times the wall or slab thickness, or 18 inches. 8.22 PROTECTION AGAINST CORROSION 8.22.1 The following minimum concrete cover shall be provided for reinforcement: Minimum Cover (inches) Concrete cast against and permanently exposed to earth ..... Concrete exposed to earth or weather: Primary reinforcement ..... Stirrups, ties, and spirals ..... Concrete deck slabs in mild climates: Top reinforcement ..... Bottom reinforcement ..... Concrete deck slabs which have no positive corrosion protection and are frequently exposed to deicing salts: Top reinforcement ..... Bottom reinforcement ..... Concrete not exposed to weather or in contact with ground: Primary reinforcement ..... Stirmps, ties, and spirals ..... Concrete piles cast against andlor permanently exposed to earth ..... 3 2 1% 2 1 217 TABLE 8.23.2.1 Minimum Diameters of Bend Bar Size - Minimum Diameter -- Nos. 3 through 8 Nos. 9, 10, and 11 Nos. 14 and 18 - -- - - 6-bar diameters 8-bar diameters 10-bar diameters concrete or other means.

Other means of positive corrosion protection may consist of, but not be limited to, epoxy-coated bars, special concrete overlays, and impervious membranes; or a combination of these means.* 8.22.4 Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion. 8.23 HOOKS AND BENDS 8.23.1 Standard Hooks The term "standard hook" as used herein shall mean one of the following: (1) 180" bend plus 4dbextension, but not less than 2% inches at free end of bar. (2) 90" bend plus 12dbextension at free end of bar. (3) For stirrup and tie hooks: (a) No. 5 bar and smaller, 90" bend plus 6dbextension at free end of bar, or (b) No. 6, No. 7, and No. 8 bar, 90" bend plus 12db extension at free end of bar, or (c) No. 8 bar and smaller, 135" bend plus 6dbextension at free end of bar. 2% 1 8.23.2 Minimum Bend Diameters 1% 1 8.23.2.1 Diameter of bend measured on the inside of the bar, other than for stirrups and ties, shall not be less than the values given in Table 8.23.2.1. 2 8.22.2 For bundled bars, the minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 2 inches, except for concrete cast against and permanently exposed to earth in which case the minimum cover shall be 3 inches. 8.22.3 In corrosive or marine environments or other severe exposure conditions, the amount of concrete protection shall be suitably increased, by increasing the denseness and imperviousness to water of the protecting 8.23.2.2 The inside diameter of bend for stirrups and ties shall not be less than 4 bar diameters for sizes No. 5 and smaller. For bars larger than size No. 5 diameter of bend shall be in accordance with Table 8.23.2.1. 8.23.2.3 The inside diameter of bend in smooth or deformed welded wire fabric for stirmps and ties shall not be less than 4-wire diameters for deformed wire larger than D6 and 2-wire diameters for all other wires. Bends with inside *For additional information on corrosion protection methods, refer to National Cooperative Highway Research Report 297, "Evaluation of Bridge Deck Protective Strategies." 218 HIGHWAY BRIDGES diameters of less than &wire diameters shall not be less than 4-wire diameters from the nearest welded intersection. 8.24 8.24.1 DEVELOPMENT OF FLEXURAL REINFORCEMENT General 8.24.1.1 The calculated tension or compression in the reinforcement at each section shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination thereof. Hooks may be used in developing bars in tension only. 8.23.2.3 8.24.1.4.3 For No. 11 bars and smaller, the continuing bars provide double the area required for flexure at the cutoff point and the shear does not exceed three-fourths that permitted. 8.24.1.5 Adequate end anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets; deep flexural members; or members in which the tension reinforcement is not parallel to the compression face. 8.24.2 Positive Moment Reinforcement 8.24.1.2 Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates or is bent. The provisions of Article 8.24.2.3 must also be satisfied. 8.24.2.1 At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of the member into the support. In beams, such reinforcement shall extend into the support at least 6 inches. 8.24.1.2.1 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member, 15 bar diameters, or KO of the

logarithms (Article 9.16) = average concrete compressive stress at the c.g. of the prestressing steel under full dead - load (Article 9.16) = average concrete stress at the c.g. of the prestressing steel at time of release (Article 9.16) = compressive strength of concrete at 28 days = compressive strength of concrete at time of initial prestress (Article 9.15) = average splitting tensile strength of lightweight aggregate concrete, psi = stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads (Article 9.20) = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange (In a composite member, f, is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone.)(Article 9.20) = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (Article 9.20) z. HIGHWAY BRIDGES = guaranteed ultimate tensile strength of the prestressing steel, Atf: = the modulus of rupture of concrete, as defined in Article 9.15.2.3 (Article 9.18) = total prestress loss, excluding friction (Article 9.16) = effective steel prestress after losses = average stress in prestressing steel at ultimate load = ultimate stress of prestressing steel (Articles 9.15 and 9.17) = yield stress of non-prestressed conventional reinforcement in tension (Articles 9.19 and 9.20) = yield stress of non-prestressed conventional reinforcement in compression (Article 9.19) = yield stress of prestressing steel (Article 9.15) = 0.90 f: for low-relaxation wire or strand = 0.85 f: for stress-relieved wire or strand = 0.85 f; for Type I (smooth) high-strength bat = 0.80 f: for Type 11 (deformed) high-strength bar = overall depth of member (Article 9.20) = moment of inertia about the centroid of the cross section (Article 9.20) = friction wobble coefficient per foot of prestressing steel (Article 9.16) = length of prestressing steel element from jack end to point x (Article 9.16) = moment causing flexural cracking at section due to externally applied loads (Article 9.20) = cracking moment (Article 9.18) = composite dead load moment at the section (Commentary to Article 9.18) = noncomposite dead load moment at the section (Article 9.18) = maximum factored moment at section due to externally applied loads (Article 9.20) = nominal moment strength of a section = factored moment at section 5 +M, (Articles 9.17 and 9.18) = As/bd, ratio of non-prestressed tension reinforcement (Articles 9.7 and 9.17-9.19) = AT/bd, ratio of prestressing steel (Articles 9.17 and 9.19) = Al/bd, ratio of compression reinforcement (Article 9.19) = factored tendon force = statical moment of cross-sectionalarea, above or below the level being investigated for shear, about the centroid (Article 9.20) 9.1.2 = loss of prestress due to concrete shrinkage (Article 9.16) = longitudinal spacing of the web reinforcement (Article 9.20) = noncomposite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads (Article 9.18) = composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads (Article 9.18) = average thickness of the flange of a flanged member (Articles 9.17 and 9.18) = steel stress at jacking end (Article 9.16) = steel stress at any point x (Article 9.16) = permissible horizontal shear stress (Article 9.20) = nominal shear strength provided by concrete (Article 9.20) = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment (Article 9.20) = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web (Article 9.20) = shear force at section due to unfactored dead load (Article 9.20) = factored shear force at section due to externally applied loads occurring simultaneously with M, (Article 9.20) = nominal horizontal shear strength (Article 9.20) = vertical component of effective prestress force at section (Article 9.20) = nominal shear strength provided by shear reinforcement (Article 9.20) = factored shear force at section (Article 9.20) = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension (Article 9.20) = friction curvature coefficient (Article 9.16) = total angular change of prestressing steel profile in radians from jacking end to point x (Article 9.16) = factor for concrete strength, as defined in Article 8.16.2.7 (Articles 9.17,9.18 and 9.19) = factor for type of prestressing steel (Article 9.17) = 0.28 for low-relaxation steel = 0.40 for stress-relieved steel = 0.55 for bars 9.1.3 DIVISION I--DESIGN 9.1.3 Definitions The following terms are defined for general use. Specialized definitions appear in individual articles.

Anchorage Device-The hardware assembly used for transferring a post-tensioning force from the tendon wires, strands or bars to the concrete. Anchorage Seating-Deformation of anchorage or seating of tendons in anchorage device when prestressing force is transferred from jack to anchorage device. Anchorage Spacing-Center-to-center spacing of anchorage devices. Anchorage Zone-The portion of the structure in which the concentrated prestressing force is transferred from the anchorage device into the concrete (Local Zone), and then distributed more widely into the structure (General Zone) (Article 9.21.1). Basic Anchorage Device-Anchorage device meeting the restricted bearing stress and minimum plate stiffness requirements of Articles 9.21.7.2.2 through 9.21.7.2.4; no acceptance test is required for Basic Anchorage Devices. Bonded Tendon-Prestressing tendon that is bonded to concrete either directly or through grouting.

Coating-Material used to protect prestressing tendons against corrosion, to reduce friction between tendon and duct, or to debond prestressing tendons. Couplers (Couplings)-Means by which prestressing force is transmitted from one partial-length prestressing tendon to another. Creep of Concrete-Time-dependent deformation of concrete under sustained load. Curvature Friction-Friction resulting from bends or curves in the specified prestressing tendon profile. Debonding (blanketing)-Wrapping, sheathing, or coating prestressing strand to prevent bond between strand and surrounding concrete. Diaphragm-Transverse stiffener in girders to maintain section geometry. Duct-Hole or void formed in prestressed member to accommodate tendon for post-tensioning.

Edge Distance-Distance from the center of the anchorage device to the edge of the concrete member. Effective Prestress-Stress remaining in concrete due to prestressing after all calculated losses have been deducted, excluding effects of superimposed loads and weight of member; stress remaining in prestressing tendons after all losses have occurred excluding effects of dead load and superimposed load. 227 Elastic Shortening of Concrete-Shortening of member caused by application of forces induced by prestressing. End Anchorage-Length of reinforcement, or mechanical anchor, or hook, or combination thereof, beyond point of zero stress in reinforcement. End Block-Enlarged end section of member designed to reduce anchorage stresses. Friction (post-tensioning)-Surface resistance between tendon and duct in contact during stressing. General Zone-Region within which the concentrated prestressing force spreads out to a more linear stress distribution over the cross section of the member (Saint Venant Region) (Article 9.21.2.1) Grout Opening or Vent-Inlet, outlet, vent, or drain in post-tensioning duct for grout, water, or air Intermediate Anchorage-Anchorage not located at the end surface of a member or segment; usually in the form of embedded anchors, blisters, ribs, or recess pockets Jacking Force-Temporary force exerted by device that introduces tension into prestressing tendons. Local Zone-The volume of concrete surrounding and immediately ahead of the anchorage device, subjected to high local bearing stresses (Article 9.21.2.2) Loss of Prestress-Reduction in prestressing force resulting from combined effects of strains in concrete and steel, including effects of elastic shortening, creep and shrinkage of concrete, relaxation of steel stress, and for post-tensioned members, friction and anchorage seating. Post-Tensioning-Method of prestressing in which tendons are tensioned after concrete has hardened. Precompressed Zone-Portion of flexural member cross section compressed by prestressing force. Prestressed Concrete-Reinforced concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads. Pretensioning-Method of prestressing in which tendons are tensioned before concrete is placed. Relaxation of Tendon Stress-Time-dependent reduction of stress in prestressing tendon at constant strain. Shear Lag--Nonuniform distribution of bending stress over the cross section. Shrinkage of Concrete-Time-dependentdeformation of concrete caused by drying and chemical changes (hydration process). Special Anchorage Device-Anchorage device whose adequacy must be proven experimentally in the standardized acceptance tests of Division 11, Article 10.3.2.3. 228 HIGHWAY BRIDGES Tendon-Wire, strand, or bar, or bundle of such elements, used to impart prestress to concrete.

Transfer-Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member. Transfer Length-Length over which prestressing force is transferred to concrete by bond in pretensioned members. Wobble Friction-Friction caused by unintended deviation of prestressing sheath or duct from its specified profile or alignment. Wrapping or Sheathing-Enclosure around a prestressing tendon to avoid temporary or permanent bond between prestressing tendon and surrounding concrete.

9.2 CONCRETE The specified compressive strength, f:, of the concrete for each part of the structure shall be shown on the plans. The requirements for f: shall be based on tests of cylinders made and tested in accordance with Division U, Section 8, "Concrete Structures." 9.1.3 9.3 REINFORCEMENT 9.3.1 Prestressing Steel Wire, strands, or bars shall conform to one of the following specifications. "Uncoated Stress-Relieved Wire for Prestressed Concrete," AASHTO M 204. "Uncoated Seven-WireStress-Relieved Strand for Prestressed Concrete," AASHTO M 203. "Uncoated High-Strength Steel Bar for Prestressing Concrete," ASTM A 722. Wire, strands, and bars not specifically listed in AASHTO M 204, AASHTO M 203, or ASTM A 722 may be used provided they conform to the minimum requirements of these specifications. 9.3.2 Non-Prestressed Reinforcement Non-prestressedreinforcement shall conform to the requirements in Article 8.3. Part B ANALYSIS 9.4 GENERAL 9.6 SPAN LENGTH Members shall be proportioned for adequate strength using these specifications as minimum guidelines. Continuous beams and other statically indeterminate structures shall be designed for adequate strength and satisfactory behavior. Behavior shall be determined by elastic analysis, taking into account the reactions, moments, shear, and axial forces produced by prestressing, the effects of temperature, creep, shrinkage, axial deformation, restraint of attached structural elements, and foundation settlement.

The effective span lengths of simply supported beams shall not exceed the clear span plus the depth of the beam. The span length of continuous or restrained floor slabs and beams shall be the clear distance between faces of support. Where fillets making an angle of 45" or more with the axis of a continuous or restrained slab are built monolithic with the slab and support, the span shall be measured from the section where the combined depth of the slab and the fillet is at least one and one-half times the thickness of the slab.

Maximum negative moments are to be considered as existing at the ends of the span, as above defined. No portion of the fillet shall be considered as adding to the effective depth. 9.5 EXPANSION AND CONTRACTION 9.5.1 In all bridges, provisions shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. 9.7 FRAMESAND CONTINUOUS CONSTRUCTION 9.5.2 Movements not otherwise provided for, including shortening during stressing, shall be provided for by means of hinged columns, rockers, sliding plates, elastomeric pads, or other devices. The effect of secondary moments due to prestressing shall be included in stress calculations at working load. In calculating ultimate strength moment and shear requirements, the secondary moments or shears induced by pre- 9.7.1 Cast-in-Place Post-Tensioned Bridges 9.7.1 DIVISION 1-DESIGN -- stressing (with a load factor of 1.0) shall be added algebraically to the moments and shears due to factored or ultimate dead and live loads.

9.7.2 Bridges Composed of Simple-Span Precast Prestressed Girders Made Continuous 9.7.2.1 General When structural continuity is assumed in calculating live loads plus impact and composite dead load moments, the effects of creep and shrinkage shall be considered in the design of bridges incorporating simple span precast, prestressed girders and deck slabs continuous over two or more spans. 9.7.2.2 Positive Moment Connection at Piers 9.7.2.2.1 Provision shall be made in the design for the positive moments that may develop in the negative moment region due to the combined effects of creep and shrinkage in the girders and deck slab, and due to the effects of live load plus impact in remote spans. Shrinkage and elastic shortening of the pier shall be considered when significant.

9.7.2.2.2 Non-prestressed positive moment connection reinforcement at piers may be designed at a working stress of 0.6 times the yield strength but not to exceed 36 ksi. 9.7.2.3 Negative Moments 9.7.2.3.1 Negative moment reinforcement shall be proportioned by strength design with load factors in accordance with Article 9.14. 9.7.2.3.2 The ultimate negative resisting moment shall be calculated using the compressive strength of the girder concrete regardless of the strength of the diaphragm concrete. 9.7.3 Segmental Box Girders 9.7.3.1 General 229 - - - segment weights and erection loads shall be accommodated in pier design or with auxiliary struts. Erection equipment which can eliminate these unbalanced moments may be used. 9.7.3.2 Flexure The transverse design of segmental box girders for flexure shall consider the segments as rigid box frames.

Top slabs shall be analyzed as variable depth sections considering the fillets between top slab and webs.

Wheel loads shall be positioned to provide maximum moments, and elastic analysis shall be used to determine the effective longitudinal distribution of wheel loads for each load location. (See Article 3.11.) Transverse prestressing of top slabs is generally recommended. 9.7.3.3 Torsion In the design of the cross section, consideration shall be given to the increase in web shear resulting from eccentric loading or geometry of structure. 9.8 EFFECTIVE FLANGE WIDTH 9.8.1.1 For composite prestressed construction where slabs or flanges are assumed to act integrally with the beam, the effective flange width shall conform to the provisions for T-girder flanges in Article 8.10.1. 9.8.1.2 For monolithic prestressed construction, with normal slab span and girder spacing, the effective flange width shall be the distance center-to-centerof beams. For very short spans, or where girder spacing is excessive, analytical investigations shall be made to determine the anticipated width of flange acting with the beam. 9.8.1.3 For monolithic prestressed design of isolated beams, the flange width shall not exceed 15 times the web width and shall be adequate for all design loads. 9.7.3.1.1 Elastic analysis and beam theory may be used in the design of segmental box girder structures.

9.8.2 Box Girders 9.7.3.1.2 In the analysis of precast segmental box girder bridges, no tension shall be permitted across any joint between segments during any stage of erection or service loading. 9.8.2.1 For cast-in-place box girders with normal slab span and girder spacing, where the slabs are considered an integral part of the girder, the entire slab width shall be assumed to be effective in compression. 9.7.3.1.3 In addition to the usual substructure design considerations, unbalanced cantilever moments due to 9.8.2.2 For box girders of unusual proportions, including segmental box girders, methods of analysis which 230 HIGHWAY BRIDGES consider shear lag shall be used to determine stresses in the cross section due to longitudinal bending. 9.8.2.2 9.10 DIAPHRAGMS 9.10.1 General 9.8.2.3 Adequate fillets shall be provided at the intersections of all surfaces within the cell of a box girder, except at the junction of web and bottom flange where none are required. Diaphragms shall be provided in accordance with Articles 9.10.2 and 9.10.3 except that diaphragms may be omitted where tests or structural analysis show adequate strength. 9.8.3 Precast/Prestressed Concrete Beams with Wide Top Flanges 9.8.3.1 For composite prestressed concrete where slabs or flanges are assumed to act integrally with the precast beam, the effective web width of the precast beam shall be the lesser of (1) six times the maximum thickness of the flange (excluding fillets) on either side of the web plus the web and fillets, and (2) the total width of the top flange. Diaphragms or other means shall be used at span ends to strengthen the free edge of the slab and to transmit lateral forces to the substructure. Intermediate diaphragms shall be placed between the beams at the points of maximum moment for spans over 40 feet. 9.8.3.2 The effective flange width of the cornposite section shall be the lesser of (1) one-fourth of the span length of the girder, (2) six (6) times the thickness of the slab on each side of the effective web width as determined by Article 9.8.3.1 plus the effective web width, and (3) one-half the clear distance on each side of the effective web width plus the effective web width. 9.10.3.1 For spread box beams, diaphragms shall be placed within the box and between boxes at span ends and at the points of maximum moment for spans over 80 feet.

9.9 FLANGE AND WEB THICKNESS-BOX GIRDERS 9.9.1 Top Flange The minimum top flange thickness shall be %oth of the clear distance between fillets or webs but not less than 6 inches, except the minimum thickness may be reduced for factory produced precast, pretensioned elements to 5% inches. 9.9.2 Bottom Flange 9.10.3 Box Girders 9.10.3.2 For precast box multi-beam bridges, diaphragms are required only if necessary for slab-end support or to contain or resist transverse tension ties. 9.10.3.3 For cast-in-place box girders, diaphragms or other means shall be used at span ends to resist lateral forces and maintain section geometry.

Intermediate diaphragms are not required for bridges with inside radius of curvature of 800 feet or greater. 9.10.3.4 For segmental box girders, diaphragms shall be placed within the box at span ends. Intermediate diaphragms ke not required for bridges with inside radius of curvatureof 800 feet or greater. 9.10.3.5 For all types of prestressed boxes in bridges with inside radius of curvature less than 800 feet, intermediate diaphragms may be required and the spacing and strength of diaphragms shall be given special consideration the design of the structure.

flange thickness shall be The the clear distance between fillets or webs but not less than 51/2 inches, except the minimum thickness may be reduced for factory produced precast, pretensioned elements to 5 inches. 9.11 DEFLECTIONS 9.9.3 Web 9.11.1 General Changes in girder stem thickness shall be tapered for a minimum distance of 12 times the difference in web thickness. Deflection calculations shall consider dead load, live load, prestressing, erection loads, concrete creep and shrinkage, and steel relaxation. 9.11.2 DMSION I-DESIGN 9.11.2 Segmental Box Girders 9.12 DECK PANELS Deflections shall be calculated prior to casting of segments and they shall be based on the anticipated casting and erection schedules. Calculated deflections shall be used as a guide against which actual deflection measurements are checked. 9.12.1 General 231 9.12.1.1 Precast prestressed deck panels used as permanent forms spanning between stringers may be designed compositely with the cast-in-place portion of the slabs to support additional dead loads and live loads. 9.11.3 Superstructure Deflection Limitations When making deflection computations, the following criteria are recommended. 9.11.3.1 Members having simple or continuous spans preferably should be designed so that the deflection due to service live load plus impact shall not exceed Ym of the span, except on bridges in urban areas used in part by pedestrians whereon the ratio preferably shall not exceed YIW. 9.11.3.2 The deflection of cantilever arms due to service live load plus impact preferably should be limited to Ym of the cantilever arm except for the case including pedestrian use, where the ratio preferably should be YWS. 9.12.1.2 The panels shall be analyzed assuming they support their self-weight, any construction loads, and the weight of the cast-in-place concrete, and shall be analyzed assuming they act compositely with the cast-in-place concrete to support moments due to additional dead loads and live loads. 9.12.2 Bending Moment 9.12.2.1 Live load moments shall be computed in accordance with Article 3.24.3. 9.12.2.2 In calculating stresses in the deck panel due to negative moment near the stringer, no compression due to prestressing shall be assumed to exist. Part C DESIGN 9.13 GENERAL 9.13.2.2 Before cracking, stress is linearly proportional to strain. 9.13.1 Design Theory and General Considerations 9.13.1.1 Members shall meet the strength requirements specified herein. 9.13.1.2 Design shall be based on strength (Load Factor Design) and on behavior at service conditions (Allowable Stress Design) at all load stages that may be critical during the life of the structure from the time prestressing is first applied. 9.13.1.3 Stress concentrations due to the prestressing shall be considered in the design. 9.13.1.4 The effects of temperature and shrinkage shall be considered. 9.13.2 Basic Assumptions The following assumptions are made for design purposes for monolithic members. 9.13.2.1 Strains vary linearly over the depth of the member throughout the entire load range.

9.13.2.3 After cracking, tension in the concrete is neglected. 9.13.3 Members Composite flexural members consisting of precast andlor cast-in-place concrete elements constructed in sepmate placements but so interconnected that all elements respond to superimposed loads as a unit shall conform to the provisions of Articles 8.14.2.1 through 8.14.2.4, Article 8.14.2.6, and the following. 9.13.3.1 Where an entire member is assumed to resist the vertical shear, the design shall be in accordance with the requirements of Articles 9.20.1 through 9.20.3. 9.13.3.2 The design shall provide for full transfer of horizontal shear forces at contact surfaces of interconnected elements. Design for horizontal shear shall be in accordance with the requirements of Article 9.20.4. 232 HIGHWAY BRIDGES 9.13.3.3 In structures with a cast-in-place slab on precast beams, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of the beams. Because the tensile shrinkage develops over an extended time period, the effect on the beams is reduced by creep. Differential shrinkage may influence the cracking load and the beam deflection profile. When these factors are particularly significant, the effect of differentialshrinkage should be added to the effect of loads. 9.14 LOAD FACTORS The computed strength capacity shall not be less than the largest value from load factor design in Article 3.22. For the design of post-tensioned anchorage zones a load factor of 1.2 shall be applied to the maximum tendon jacking force. The following strength capacity reduction factors shall be used: For factory produced precast prestressed concrete members 4 = 1.0 For post-tensioned cast-in-place concrete members 4 = 0.95 For shear 4 = 0.90 For anchorage zones = 0.85 for normal weight concrete and = 0.70 for lightweight concrete. + + 9.15 ALLOWABLE STRESSES The design of precast prestressed members ordinarily shall be based on f: = 5,000 psi. An increase to 6,000 psi is permissible where, in the Engineer's judgment, it is reasonable to expect that this strength will be obtained consistently. Still higher concrete strengths may be considered on an individual area basis. In such cases, the Engineer shall satisfy himself completely that the controls over materials and fabrication procedures will provide the required strengths. The provisions of this Section are equally applicable to prestressed concrete structures and components designed with lower concrete strengths. 9.15.1 Prestressing Steel Pretensioned members: Stress immediately prior to transferLow-relaxation slands ..... .0.75 f,' Stress-relieved strands ..... .0.70 f,' Post-tensioned members: Stress immediately after seatingAt anchorage . . . . . .

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. . .0.70 fi 9.13.3.3 At the end of the seating loss zone .....

0.83 q Tensioning to 0.90 q for short periods of time prior to seating may be permitted to offset seating and friction losses provided the stress at the anchorage does not exceed the above value. Stress at service loadt after losses ..... .0.80 q 9.15.2 Concrete 9.15.2.1 Temporary Stresses Before Losses Due to Creep and Shrinkage Compression: Pretensioned members .....0.60 fLi Post-tensioned members .....0.55 fLi Tension: Precompressed tensile zone ..... .No temporary allowable stresses are specified. See Article 9.15.2.2 for allowable stresses after losses. Other Areas In tension areas with no bonded reinforcement ..... .200 psi or 3 a Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section. The maximum tensile stress shall not exceed ..... .7.5- 9.15.2.2 Stress at Service Load After Losses Have Occurred Compression: (a) The compressive stresses under all load combinations, except as stated in (b) and (c), shall not exceed 0.60f:. (b) The compressive stresses due to effective prestress plus permanent (dead) loads shall not exceed 0.40fL. (c) The compressive stress due to live loads plus onehalf of the sum of compressive stresses due to prestress and permanent (dead) loads shall not exceed 0.40f:. Tension in the precompressed tensile zone: (a) For members with bonded reinforcement* .....

6 e For severe corrosive exposure conditions, such as coastal areas ..... 3 f i *Includes bonded prestressed strands. tService load consists of all loads contained in Article 3.2 but does not include overload provisions. 9.15.2.2 DIVISION I-DESIGN (b) For members without bonded reinforcement . . . . . .

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. . . . . 0 Tension in other areas is limited by allowable temporary stresses specified in Article 9.15.2.1. 233 tions. Rigid ducts shall have sufficient strength to maintain their correct alignment without visible wobble during placement of concrete.

Rigid ducts may be fabricated with either welded or interlocked seams. Galvanizing of the welded seam will not be required. 9.15.2.3 Cracking Stress* 9.16.2 Prestress Losses Modulus of rupture from tests or if not available. For normal weight concrete ..... .7.5fi For sand-lightweight concrete ..... .6.3fl For all other lightweight concrete .....5.5fi 9.15.2.4 Anchorage Bearing Stress Post-tensioned anchorage at service load (but not to exceed 0.9 fiJ .. .3,000 psi 9.16 LOSS OF PRESTRESS 9.16.1 Friction Losses Friction losses in post-tensioned steel shall be based on experimentally determined wobble and curvature coefficients, and shall be verified during stressing operations. The values of coefficients assumed for design, and the acceptable ranges of jacking forces and steel elongations shall be shown on the plans. These friction losses shall be calculated as follows: 9.16.2.1 General Loss of prestress due to all causes, excluding friction, may be determined by the following method.** The method is based on normal weight concrete and one of the steel: 250 or 270 ksi, following types of seven-wire, stress-relieved or low-relaxation strand; 240 ksi stress-relieved wires; or 145 to 160 ksi smooth or deformed bars. Refer to documented tests for data regarding the properties and the effects of lightweight aggregate concrete on prestress losses. TOTAL LOSS Af, = SH + ES + CR, + CR, (9-3) where: Af, = total loss excluding friction in pounds per square inch; To = T,eW + P). (9-1) SH square inch; + When (KL pa) is not greater than 0.3, the following equation may be used: The following values for K and p may be used when experimental data for the materials used are not available: Q p e of Steel o p e of Duct Wire or strand Rigid and semi-rigid galvanized metal sheathing ~ol~eth~le~e Rigid steel pipe 0.0002 0.15-0.25a 0.0002 0.0002 0.23 0.25b Galvanized metal sheathing 0.0002 0.15 High Strength bars = loss due to elastic shortening in pounds per CR, = loss due to creep of concrete in pounds per square inch; CR, = loss due to relaxation of prestressing steel in pounds per square inch. P 9.16.2.1.1 Shrinkage nAfriction coefficient of 0.25 is appropriate for 12 strand tendons. A lower coefficient may be used for larger tendon and duct sizes. bLubricationwill probably be required.

Friction losses occur prior to anchoring but should be for design and checked during stressing opera*Refer to Article 9.18. ES sauare inch; To= T, (1 + KL + pa) aft = loss due to concrete shrinkage in pounds per Pretensioned Members: SH = 17,000 - 150 RH (9-4) Post-tensioned Members: SH = 0.80 (17,000 - 150 RH) (9-5) **Should more exact prestress losses be desired, data representingthe materials to be used, the methods of curing, the ambient service condition and any pertinent structural details should be determined for use in accordance with a method of calculating prestress losses that is supported by appropriate research data. See also FHWA Report FHWAIRD 851045, Criteria for Designing Lightweight Concrete Bridges. 234 9.16.2.1.1 HIGHWAY BRIDGES where RH = mean annual ambient relative humidity in percent. (See Figure 9.16.2.1.1.) 9.16.2.1.3 Creep of Concrete Pretensioned and post-tensioned members 9.16.2.1.2 Elastic Shortening CR, =12 fc, - 7 fcds (9 - 9) Pretensioned Members where Es ES = -fc, Eci (9 - 6) Post-tensioned Members* fcds = concrete stress at the center of gravity of the prestressing steel due to all dea6%ds e s t the dead load present at the time the pre--stressing force is applied. 9.16.2.1.4 Relaxation of Prestressing Steel** ES = 0.5 E Eci fc. (9 - 7) where E, E,, = modulus of elasticity of prestressing steel strand, which can be assumed to be 28 X lo6 psi; = modulus of elasticity of concrete in psi at transfer of stress, which can be calculated from: Eci = 3 3 w 3 I 2 z fci, (9 - 8) in which w is the concrete unit weight in pounds per cubic foot and fLi is in pounds per square inch; = concrete stress at the center of gravity of the prestressing steel due to prestressing force and dead load of beam immediately after transfer; fckshall be computed at the section or sections of maximum moment. (At this stage, the initial stress in the tendon has been reduced by elas&cshz$er& of the concrete and tendon r F @ n during placing and curing the concrete for pretensioned members, or by elastic shortening of the concrete and tendon friction for post-tensioned members. The reductions to initial tendon stress due to these factors can be estimated, or the reduced tendon stress can be taken as 0.63 f: for stress relieved strand or 0.69 f: for low relaxation strand in typical pretensioned members.) *Certain tensioning procedures may alter the elastic shortening losses. Pretensioned Members 250 to 270 ksi Strand CR, = 20,000 - 0.4 ES - 0.2 (SH + CR,) for stress relieved strand (9-10) CR, = 5,000 - 0.10 ES - 0.05 (SH + C%) (9- 10A) for low relaxation strand Post-tensioned Members 250 to 270 ksi Strand CR, = 20,000 - 0.3 FR - 0.4 ES - 0.2 (SH + CK) for stress relieved strand (9-11) CR, = 5,000 - 0.07 FR - 0.1 ES - 0.05 (SH + CK) for low relaxation strand (9-llA) 240 ksi Wire CR, = 18,000 - 0.3 FR - 0.4 ES - 0.2 (SH + CR,) (9-12) 145- to 160-ksi Bars CR, = 3,000 where FR = friction loss stress reduction in psi below the level of 0.70 fi at the point under consideration, computed according to Article 9.16.1, ES, SH, = appropriate values as determined for and CR, either pretensioned or post-tensioned members. **The relaxationlosses are based on an initial stress equal to the stress at anchorages allowed by Article 9.15.1. 9.16.2.1.4 DIVISION I-DESIGN 235 HIGHWAY BRIDGES satisfy Equation (9-24), the design flexural strength shall be assumed as: 9.16.2.2 Estimated Losses In lieu of the preceding method, the following estimates of total losses may be used for prestressed members or structures of usual design.

These loss values are based on use of normal weight concrete, normal prestress levels, and average exposure conditions.

For exceptionally long spans, or for unusual designs, the method in Article 9.16.2.1 or a more exact method shall be used. 9.17.3 Flanged Sections TABLE 9.16.2.2 Estimate of Prestress Losses of Prestressing Steel -- Pretensioning Strand Post-Tensioninga Wire or Strand Bars Total Loss fi = 4,000 psi f: = 5,000 psi - 45,000 psi 32,000 psi 22,000 psi 33,000 psi 23,000 psi For sections having prestressing steel only, in which the depth of the equivalent rectangular stress block, defined as (Asrf~u)/(0.85f~b') is greater than the compression flange thickness "t," and which satisfy Equation (9-21), the design flexural strength shall be assumed as: "Losses due to friction are. excluded. Friction losses should be computed according to Article 9.16.1. 9.17 FLEXURAL STRENGTH + 0.85 f,' (b - bf)(t)(d - 0.5t) 9.17.1 General Prestressed concrete members may be assumed to act as uncracked members subjected to combined axial and bending stresses within specified service loads. In calculations of section properties, the transformed area of bonded reinforcement may be included in pretensioned members and in post-tensioned members after grouting; prior to bonding of tendons, areas of the open ducts shall be deducted.

(9 - 14) For sections with non-prestressed tension reinforcement included, in which the depth of the equivalent rectangular stress block, defined as (As,ftu)/(0.85fLb') is greater than the compression flange thickness "t," and which satisfy Equation (9-25), the design flexural strength shall be assumed as: 9.17.2 Rectangular Sections For rectangular or flanged sections having prestressing steel only, which the depth of the equivalent rectangular stress block, defined as (A: ffu)/(0.85fdb), is not greater than the compression flange thickness "t", and which satisfy Equation (9-20), the design flexural strength shall be assumed as: where: A,, = A: - Asfin Equation (9-14); (9-15) As, = A? + (ASfSY/EU) - As, in Equation (9-14a) (9-15a) For rectangular or flanged sections with nonprestressed tension reinforcement included, in which the depth of the equivalent rectangular stress block, defined as (At ffu + Asfs,)/(0.85 f,'b), is not greater than the compression flange thickness "t," and which Asf= the steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.

9.17.4 DIVISION I-DESIGN 237 9.17.4 Steel Stress 9.18 DUCTILITY LIMITS 9.17.4.1 Unless the value of fZUcan be more accurately known from detailed analysis, the following values may be used: 9.18.1 Maximum Prestressing Steel Bonded Members . . . with prestressing only (as defined); Prestressed concrete members shall be designed so that the steel is yielding as ultimate capacity is approached. In general, the reinforcement index shall be such that (p*Eu)/fifor rectangular sections (9-20) &,EU/(bfdfL)for flanged sections (9-21) f?; = $ [l - (y * / ~ ~) (~ * f : / f ~)] (9-17) and with non-prestressed tension reinforcement included; Y * p * f,' f?; = dt pfsy f~{l-K[fr+d(fC)]} Unbonded members . . . f", = f, (9 - 17a) + 900((d - y,)&) (9-18) but shall not exceed f;. Where does not exceed 0.36P1. (See Article 9.19 for reinforcement indices of sections with non-prestressed reinforcement.). For members with reinforcement indices greater than 0.36P1,the design flexural strength shall be assumed not greater than: For rectangular sections y, = distance from extreme compression fiber to 1, li N, the neutral axis assuming the tendon prestressing steel has yielded. = li/(l 0.5N5);effective tendon length. = tendon length between anchorages (inch). = number of support hinges crossed by the tendon between anchorages or discretely bonded points. + provided that (1) The stress-strain properties of the prestressing steel approximate those specified in Division 11, Article 10.3.1.1. (2) The effective after losses is not less than 0.5 f:. 9.17.4.2 At ultimate load, the stress in the prestressing steel of precast deck panels shall be limited to but shall not be greater than fZuas given by the equations in Article 9.17.4.1. In the above equation: D = nominal diameter of strand in inches; f,, = effective stress in prestressing strand after losses in kips per square inch; t , = distance from end of prestressing strand to center ' of panel in inches. For flanged sections +Mn = +[(0.36P1 - 0.08 P:) fib'd2 0.85 f,' (b - b') t (d - 0.5t)l + (9-23) 9.18.2 Minimum Steel 9.18.2.1 The total amount of prestressed and nonprestressed reinforcement shall be adequate to develop an ultimate moment at the critical section at least 1.2 times the cracking moment M,*. where Appropriate values for ant and Sbshall be used for any intermediate composite sections. Where beams are designed to be noncomposite, substitute Sb for Sc in the above equation for the calculation of M,*,. 9.18.2.2 The requirements of Article 9.18.2.1 may be waived if the area of prestressed and non-prestressed reinforcement provided at a section is at least one-third 238 HIGHWAY BRIDGES greater than that required by analysis based on the loading combinations specified in Article 3.22. 9.18.2.3 The requirements of Article 9.18.2.1 may be waived if the area of prestressed and non-prestressed reinforcement provided at a section is at least one-third greater than that required by analysis based on the loading combinations specified in Article 3.22. 9.18.2.4 The minimum amount of non-prestressed longitudinal reinforcement provided in the cast-in-place portion of slabs utilizing precast prestressed deck panels shall be 0.25 square inch per foot of slab width. 9.19 NON-PRESTRESSED REINFORCEMENT Non-prestressed reinforcement may be considered as contributing to the tensile strength of the beam at ultimate strength in an amount equal to its area times its yield point, provided that For rectangular sections For flanged sections (Asfsy)/(brdfL)+ (As, f?u)/(b1dfL) - (ALfi)/(brdfL)5 0.36P1 (9-25) Design flexural strength shall be calculated based on Equation (9-13a) or Equation (9-14a) if these values are met, and on Equation (9-22) or Equation (9-23) if these values are exceeded. 9.18.2.2 wire fabric with wires located perpendicular to the axis of the member. Web reinforcement shall extend to a distance d from the extreme compression fiber and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Web reinforcement shall be anchored at both ends for its design yield strength in accordance with the provisions of Article 8.27. 9.20.1.3 Members subject to shear shall be designed so that where V, is the factored shear force at the section considered, V, is the nominal shear strength provided by concrete and V, is the nominal shear strength provided by web reinforcement. 9.20.1.4 When the reaction to the applied loads introduces compression into the end regions of the member, sections located at a distance less than h/2 from the face of the support may be designed for the same shear V, as that computed at a distance hl2. 9.20.1.5 Reinforced keys shall be provided in the webs of precast segmental box girders to transfer erection shear. Possible reverse shearing stresses in the shear keys shall be investigated, particularly in segments near a pier. At time of erection, the shear stress carried by the shear key shall not exceed 2 a.

9.20.2 Shear-Strength Provided by Concrete 9.20.2.1 The shear strength provided by concrete, V,, shall be taken as the lesser of the values Vcior V,.

9.20 SHEAR* 9.20.2.2 The shear strength, Vci,shall be computed by 9.20.1 General 9.20.1.1 Prestressed concrete flexural members, except solid slabs and footings, shall be reinforced for shear and diagonal tension stresses. Voided slabs shall be investigated for shear, but shear reinforcement may be omitted if the factored shear force, V,, is less than half the shear strength provided by the concrete V,. + 9.20.1.2 Web reinforcement shall consist of stim~ps perpendicular to the axis of the member or welded *The method for design of web reinforcement presented in the 1979 Interim AASHTO Standard Spec$cations for Highway Bridges is an acceptable alternate. but need not be less than 1.7 br d and d need not be taken less than 0.8h. The moment causing flexural cracking at the section due to externally applied loads, M,,, shall be computed by: I Mcr = - (6 &+f,, Yt -f,) (9 - 28) The maximum factored moment and factored shear at the section due to externally applied loads, M, and Vi, DIVISION I-DESIGN 9.20.2.2 shall be computed from the load combination causing maximum moment at the section. 9.20.2.3 9.203.3 shall be The minimum area of web reinforcement 50 b's A, =- The shear strength, V,, shall be computed fSY by Vcw = (3.5 f i + 0.3 f,) b'd + Vp (9 - 29) 9.203.4 The design yield strength of web reinforcement, f,,, shall not exceed 60,000 psi. but d need not be taken less than 0.8h. 9.20.2.4 For a pretensioned member in which the section at a distance h12 from the face of support is closer to the end of the member than the transfer length of the prestressing tendons, the reduced prestress shall be considered when computing V,. The prestress force may be assumed to vary linearly from zero at the end of the tendon to a maximum at a distance from the end of the tendon equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire. 9.20.2.5 The provisions for computing the shear strength provided by concrete, Vciand V,, apply to normal weight concrete. When lightweight aggregate concretes are used (see definition, concrete, structural lightweight, Article 8.1.3). one of the following modifications shall apply

prestressed reinforcement. Tensile strength of the concrete shall be neglected. 9.21.4.4.2 Tension ties shall be properly detailed and shall extend beyond the nodes to develop the full tension tie force at the node. The reinforcement layout must closely follow the directions of the ties in the strut-and-tie model. 9.21.5 Elastic Stress Analysis 9.21.5.1 Analyses based on assumed elastic material properties, equilibrium, and compatibility of strains are acceptable for analysis and design of anchorage zones.
9.21.5.2 If the compressive stresses in the concrete ahead of the anchorage device are determined from a linear-elastic stress analysis, local stress maxima may be averaged over an area equal to the bearing area of the anchorage device. 9.21.5.3 Location and magnitude of the bursting force may be obtained by integration of the corresponding tensile bursting stresses along the tendon path. 9.21.6 Approximate Methods 9.21.6.1 Limitations In the absence of a more accurate analysis, concrete compressive stresses ahead of the anchorage device, location and magnitude of the bursting force, and edge tension forces may be estimated by Equations (9-35) through (9-38), provided that: (1) The member has a rectangular cross section and its longitudinal extent is at least equal to the largest transverse dimension of the cross section. (2) The member has no discontinuities within or ahead of the anchorage zone. (3) The minimum edge distance of the anchorage in the main plane of the member is at least l Y 2 times the corresponding lateral dimension, a, of the anchorage device. (4) Only one anchorage device or one group of closely spaced anchorage devices is located in the anchorage zone. Anchorage devices can be treated as closely spaced if their center-to-center spacing does not exceed 1% times the width of the anchorage devices in the direction considered. 9.21.6.1 DIVISION I-DESIGN (5) The angle of inclination of the tendon with respect to the center line of the member is not larger than 20" if the anchor force points toward the centroid of the section and for concentric anchors, and is not larger than 5" if the anchor force points away from the centroid of the section. 9.21.6.2 Compressive Stresses 9.21.6.2.1 No additional check of concrete compressive stresses is necessary for basic anchorage devices satisfying Article 9.21.7.2. 9.21.6.2.2 The concrete compressive stresses ahead of special anchorage devices at the interface between local zone and general zone shall be approximated by Equations (9-35) and (9-36). 0.6Pu fca= K Ab 1 (9 - 35) for < 2aeff (9 - 36) for s 2 2aeff 245 If a group of anchorages is closely spaced in two directions, the product of the correction factors, K, for each direction is used in Equation (9-36). 9.21.6.2.3 Effective bearing area, Abr in Equation (9-35) shall be taken as the larger of the anchor bearing plate area, Apia,, or the bearing area of the confined concrete in the local zone, &o,,f, with the following lirnitations: (1) If Apl, controls, Aplate shall not be taken larger than 4/n Ld. (2) If Ltd controls, the maximum dimension of L0,,f shall not be more than twice the maximum dimension of Apia, or three times the minimum dimension of A,,,. If any of these limits is violated the effective-bearing area, Ab,shall be based on ApIate. (3) Deductions shall be made for the area of the duct in the determination of Ab. 9.21.6.3 Bursting Forces Values for the magnitude of the bursting force, Tbunt, and for its distance from the loaded surface, dbmrr shall be estimated by Equations (9-37) and (9-38), respectively. In the application of Equations (9-37) and (9-38) the specified stressing sequence shall be considered if more than one tendon is present. + 0.5Pu sin a (9 - 37) where: f, = the concrete compressive stress ahead of the an- K = a correction factor for closely spaced anchor- chorage device; ages; Ab = an effective bearing area as defined in Article 9.21.6.2.3; hff =the lateral dimension of the effective bearing area measured parallel to the larger dimension of the cross section or in the direction of closely spaced anchors; beff= the lateral dimension of the effective bearing area measured parallel to the smaller dimension of the cross section; 4, = the longitudinal extent of confining reinforcement for the local zone, but not more than the larger of 1.15 &ffor 1.15 beff; Pu = the factored tendon load; t = the thickness of the section; s = the center-to-center spacing of multiple anchorages; n = the number of anchorages in a row. where, XPu = the sum of the total factored tendon loads for the stressing arrangement considered; a = the lateral dimension of the anchorage device or group of devices in the direction considered; e = the eccentricity (always taken as positive) of the anchorage device or group of devices with respect to the centroid of the cross section; h = the lateral dimension of the cross section in the direction considered; a = the angle of inclination of the resultant of the tendon forces with respect to the center line of the member. 9.21.6.4 Edge-Tension Forces 9.21.6.4.1 For multiple anchorages with a center-tocenter spacing of less than 0.4 times the depth of the section, the spalling forces shall be given by Article 246 HIGHWAY BRIDGES 9.21.3.4.8. For larger spacings, the spalling forces shall be determined from a more detailed analysis, such as strutand-tie models or other analytical procedures. 9.21.6.4.2 If the centroid of all tendons considered is located outside of the kern of the section both spalling forces and longitudinal edge tension forces are induced. The longitudinal edge-tension force shall be determined from an axial-flexural beam analysis at a section located at one-half the depth of the section away from the loaded surface. The spalling force shall be taken as equal to the longitudinal edge-tension force but not less than specified in Article 9.21.3.4.8. 9.21.7 Design of the Local Zone 9.21.7.1 Dimensions of the Local Zone 9.21.7.1.1 When no independently verified manufacturer's edge-distance recommendations for a particular anchorage device are available, the transverse dimensions of the local zone in each direction shall be taken as the larger of: (1) The corresponding bearing plate size plus twice the minimum concrete cover required for the particular application and environment. (2) The outer dimension of any required confining reinforcement plus the required concrete cover over the confining reinforcing steel for the particular application and environment. 9.21.7.1.2 When independently verified manufacturer's recommendations for minimum cover, spacing and edge distances for a particular anchorage device are available, the transverse dimensions of the local zone in each direction shall be taken as the smaller of: (1) Twice the edge distance specified by the anchorage device supplier.
(2) The center-to-center spacing specified by the anchorage device supplier. The manufacturer's recommendations for spacing and edge distance of anchoragesshall be considered minimum values. 9.21.7.1.3 The length of the local zone along the tendon axis shall be taken as the greater of: (1) The maximum width of the local zone. (2) The length of the anchorage device confining reinforcement. (3) For anchorage devices with multiple-bearing surfaces, the distance from the loaded concrete surface to 9.21.6.4.1 the bottom of each bearing surface plus the maximum dimension of that bearing surface. In no case shall the length of the local zone be taken as greater than 1% times the width of the local zone. 9.21.7.1.4 For closely spaced anchorages an enlarged local zone enclosing all individual anchorages shall also be considered. 9.21.7.2 Bearing Strength 9.21.7.2.1 Anchorage devices may be either basic anchorage devices meeting the bearing compressive strength limits of Articles 9.21.7.2.2 through 9.21.7.2.4 or spec$ anchorage devices meeting the requirements of Article 9.21.7.3. 9.21.7.2.2 The effective concrete bearing compressive strength fb used for design shall not exceed that of Equations (9-39) or (9-40). but, fb S 2.25 4 fLi (9-40) where: fb = the maximum factored tendon load, P,, divided by the effective bearing area Ab; fLi= the concrete compressive strength at stressing; A = the maximum area of the portion of the supporting surface that is geometrically similar to the loaded area and concentric with it; A, = the gross area of the bearing plate if the requirements of Article 9.21.7.2.3 are met, or is the area calculated in accordance with Article 9.21.7.2.4; Ab = the effective net area of the bearing plate calculated as the area A, minus the area of openings in the bearing plate. Equations (9-39) and (9-40) are only valid if general zone reinforcement satisfying Article 9.21.3.4 is provided and if the extent of the concrete along the tendon axis ahead of the anchorage device is at least twice the length of the local zone as defined in Article 9.21.7.1.3. 9.21.7.2.3 The full bearing plate area may be used for A, and the calculation of Abif the anchorage device is sufficiently rigid. To be considered sufficiently rigid, the slenderness of the bearing plate (nlt) must not exceed the value given in Equation (9-41). The plate must also be checked to ensure that the plate material does not yield. 9.21.7.2.3 DIVISION I-DESIGN nlt 5 0.08 (9-41) where: n = the largest distance from the outer edge of the wedge plate to the outer edge of the bearing plate. For rectangular-bearing plates this distance is measured parallel to the edges of the bearing plate. If the anchorage has no separate wedge plate, the size of the wedge plate shall be taken as the distance between the extreme wedge holes in the corresponding direction. t = the average thickness of the bearing plate. Eb = the modulus of elasticity of the bearing-plate material.
9.21.7.2.4 For bearing plates that do not meet the stiffness requirements of Article 9.21.7.2.3, the effective gross-bearing area, A,, shall be taken as the area geometrically similar to the wedge plate (or to the outer perimeter of the wedge-hole pattern for plates without separate wedge plate) with dimensions increased by assuming load spreading at a 45' angle. A larger effective-bearing area may be calculated by assuming an effective area and checking the new fb and & values for conformancewith Articles 9.21.7.2.2 and 9.21.7.2.3. 9.21.7.3 Special Anchorage Devices Special anchorage devices that do not meet the requirements of Article 9.21.7.2 as well as other devices that do meet the requirements of Article 9.21.7.2 but which the engineer of record requires to have tested may be used provided that they have been tested by an independent testing agency acceptable to the engineer of record according to the procedures described in Division 11, Article 10.3.2 (or equivalent)and meet the acceptancecriteria specified in Division 11, Article 10.3.2.3.10. For a series of similar special anchoragedevices, tests are only required for representative samples unless tests for each capacity of the anchorages in the series are required by the engineer of record. 247 9.22 PRETENSIONED ANCHORAGE ZONES 9.22.1 In pretensioned beams, vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4% of the total prestressing force shall be placed within the distance of dl4 of the end of the beam. 9.22.2 For at least the distance d from the end of the beam, nominal reinforcement shall be placed to enclose the prestressing steel in the bottom flange. 9.22.3 For box girders, transverse reinforcement shall be provided and anchored by extending the leg into the web of the girder. 9.22.4 Unless otherwise specified, stress shall not be transferred to concrete until the compressive strength of the concrete as indicated by test cylinders, cured by methods identical with the curing of the member, is at least 4,000 psi. 9.23 CONCRETE STRENGTH AT STRESS TRANSFER Unless otherwise specified, stress shall not be transferred to concrete until the compressive strength of the concrete as indicated by test cylinders, cured by methods identical with the curing of the members, is at least 4,000 psi for pretensioned members (other than piles) and 3,500 psi for post-tensioned members and prestensioned piles. 9.24 DECK PANELS 9.24.1 Deck panels shall be prestressed with pretensioned strands. The strands shall be in a direction transverse to the stringers when the panels are placed on the supporting stringers. The top surface of the panels shall be roughened in such a manner as to ensure composite action between the precast and cast-in-place concrete. 9.24.2 Reinforcing bars, or equivalent mesh, shall be placed in the panel transverse to the strands to provide at least 0.11 square inches per foot of panel. Part D DETAILING 9.25 FLANGE REINFORCEMENT 9.26 COVER AND SPACING OF STEEL Bar reinforcement for cast-in-place T-beam and box girder flanges shall conform to the provisions in Articles 8.17.2.2 and 8.17.2.3 exceptthat the minimumreinforcement in bottom flanges shall be 0.3% of the flange section. 9.26.1 Minimum Cover The following minimum concrete cover shall be provided for prestressing and conventional steel: 248 HIGHWAY BRIDGES 9.26.1.1 Prestressing Steel and Main Reinforcement .....1YZ inch 9.26.1.2 Slab Reinforcement 9.26.1.2.1 Topofslab .....1Yzinch When deicers are used .....2 inch 9.26.1.2.2 Bottom of Slab .....1 inch 9.26.1.3 Stirrups and Ties .....1 inch 9.26.1.4 When deicer chemicals are used, drainage details shall dispose of deicer solutions without constant contact with the prestressed girders. Where such contact cannot be avoided, or in locations where members are exposed to salt water, salt spray, or chemical vapor, additional cover should be provided. 9.26.2 Minimum Spacing 9.26.2.1 The minimum clear spacing of prestressing steel at the ends of beams shall be as follows: Pretensioning steel: The clear distance between strands shall not be less than 1% times the maximum size of the concrete aggregate. Also, the minimum spacing center-to-center of strand shall be as follows: Strand Size inch special, 9/16 inch, 9/16 inch special, and 6/10 inch 7/16 inch and l/2 inch 3/8 inch 1/2 Spacing 2 inches 13/4 inches 1 1/2 inches Post-tensioning steel: 1% inches or 1%times the maximum size of the concrete aggregate, whicheveris greater.
9.26.2.2 Prestressing strands in deck panels shall be spaced symmetrically and uniformly across the width of the panel.
They shall not be spaced farther apart than ~ Y z times the total composite slab thickness or more than 18 inches.
9.26.3 Bundling 9.26.3.1 When post-tensioning steel is draped or deflected, post-tensioning ducts may be bundled in groups of three maximum, provided that the spacing specified in Article 9.26.2 is maintained in the end 3 feet of the member. 9.26.3.2 Where pretensioning steel is bundled, all bundling shall be done in the middle third of the beam length and the deflection points shall be investigated for secondary stresses. 9.26.1.1 9.26.4 Size of Ducts 9.26.4.1 For tendons made up of a number of wires, bars, or strands, duct area shall be at least twice the net area of the prestressing steel. 9.26.4.2 For tendons made up of a single wire, bar, or strand, the duct diameter shall be at least % inch larger than the nominal diameter of the wire, bar, or strand. 9.27 POST-TENSIONING ANCHORAGES AND COUPLERS 9.27.1 Anchorages, couplers, and splices for bonded post-tensionedreinforcementshall develop at least 95% of the minimum specified ultimate strength of the prestressing steel, tested in an unbonded state without exceeding anticipated set. Bond transfer lengths between anchorages and the zone where full prestressing force is required under service and ultimate loads shall normally be sufficient to develop the minimum specified ultimate strength of the prestressing steel.
Couplers and splices shall be placed in areas approved by the Engineer and enclosed in a housing long enough to pennit the necessary movements. When anchorages or couplers are located at critical sections under ultimate load, the ultimate strength required of the bonded tendons shall not exceed the ultimate capacity of the tendon assembly, including the anchorage or coupler, tested in an unbonded state.
9.27.2 The anchorages of unbonded tendons shall develop at least 95% of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The total elongation under ultimate load of the tendon shall not be less than 2% measured in a minimum gauge length of 10 feet. 9.27.3 For unbonded tendons, a dynamic test shall be performed on a representative specimen and the tendon shall withstand, without failure, 500,000 cycles from 60% to 66% of its minimum specified ultimate strength, and also 50 cycles from 40% to 80% of its minimum specified ultimate strength. The period of each cycle involves the change from the lower stress level to the upper stress level and back to the lower. The specimen used for the second dynamic test need not be the same used for the first dynamic test.
Systems utilizing multiple strands, wires, or bars may be tested utilizing a test tendon of smaller capacity than the full size tendon. The test tendon shall duplicate the behavior of the full size tendon and generally shall not have less than 10% of the capacity of the full size tendon. Dynamic tests are not required on bonded tendons, unless the anchorage is located or used in such manner that repeated load applications can be expected on the anchorage. 9.27.4 249 DIVISION I-DESIGN 9.27.4 Couplings of unbonded tendons shall be used only at locations specifically indicated and/or approved by the Engineer. Couplings shall not be used at points of sharp tendon curvature. All couplings shall develop at least 95% of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The coupling of tendons shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplings and/or coupling components shall be enclosed in housings long enough to permit the necessary movements. All the coupling components shall be completely protected with a coating material prior to final encasement in concrete. 9.27.5 Anchorages, end fittings, couplers, and exposed tendons shall be permanently protected against corrosion. eu where D is the nominal diameter in inches, and f,, are in kips per square inch, and the parenthetical expression is considered to be without units.
9.28.2 Investigations may be limited to those cross sections nearest each end of the member which are required to develop their full ultimate capacity. 9.28.3 Where strand is debonded at the end of a member and tension at service load is allowed in the precompressed tensile zone, the development length required above shall be doubled. 9.28 EMBEDMENT OF PRESTRESSED STRAND 9.29 BEARINGS 9.28.1 Three- or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length in inches not less than Bearing devices for prestressed concrete structures shall be designed in accordance with Article 10.29 and Section 14. Section 10 STRUCTURAL STEEL Part A GENERAL REQUIREMENTS AND MATERIALS 10.1 APPLICATION 10.1.1 Notations = area Afc A, of cross section (Articles 10.37.1.1, 10.34.4, 10.48.1.1, 10.48.2.1, 10.48.4.2, 10.48.5.3, and 10.55.1) =bending moment coefficient (Article 10.50.1.1.2) = effective area of a flange or splice plate with holes or a tension member with holes (Articles 10.12, 10.18.2.2.1, 10.18.2.2.3, 10.18.2.2.4, and 10.18.4.1) = amplification factor (Articles 10.37.1.1 and 10.55.1) = product of area and yield point for bottom flange of steel section (Article 10.50.1.1.1) = product of area and yield point of that part of reinforcing which lies in the compression zone of the slab (Article 10.50.1.1.1) = product of area and yield point for top flange of steel section (Article 10.50.1.1.1) = product of area and yield point for web of steel section (Article 10.50.1.1.1) = area of flange (Articles 10.39.4.4.2, 10.48.2.1, 10.53.1.2, and 10.56.3) = the sum of the area of filler plates on the top and bottom of the connected plate (Article 10.18.1.2.1) = area of compression flange (Articles 10.48.4.1 and 10.50.1.2.1) = gross area of a flange, splice plate or tension member (Articles 10.18.2.2.2, 10.18.2.2.4, and 10.18.4.1) = net section of a tension member (Article 10.18.4.1) = the smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (Article 10.18.1.2.1) = total area of longitudinal reinforcing steel at, the interior support within the effective flange width (Article 10.38.5.1.2) = total area of longitudinal slab reinforcement steel for each beam over interior support (Article 10.38.5.1.3) = area of steel section (Articles 10.38.5.1.2, 10.54.1.1, and 10.54.2.1) = cross-sectionalarea of a stud shear connector (Article 10.38.5.1.2) = area of web of beam (Article 10.53.1.2) = distance from center of bolt under consideration to edge of plate, in. (Articles 10.32.3.3.2 and 10.56.2) = spacing of transverse stiffeners (Article 10.39.4.4.2) = depth of stress block (Figure 10.50A) = ratio of numerically smaller to the larger end moment (Article 10.54.2.2) = constant based on the number of stress cycles (Article 10.38.5.1.1) = constant for stiffeners (Articles 10.34.4.7 and 10.48.5.3) = compression flange width (Table 10.32.1A and Articles 10.34.2.1, 10.48, 10.48.1.1, 10.48.2, 10.48.2.1, and 10.61.4) = distance from center of bolt under consideration to toe of fillet of connected part, in. (Articles 10.32.3.3.2 and 10.56.2) = effective width of slab (Article 10.50.1.1.1) = effective flange width (Articles 10.38.3 and 10.38.5.1.2) = widest flange width (Article 10.15.2.1) = distance from edge of plate or edge of perforation to the point of support (Article 10.35.2.3) = unsupported distance between points of support (Article 10.35.2.7) = flange width between webs (Articles 10.37.3.1, 10.39.4.2, 10.51.5.1, and 10.55.3) HIGHWAY BRIDGES 10.1.1 = width of stiffeners (Articles 10.34.5.2, = depth of beam or girder, in. (Table 10.32.1A 10.34.6, 10.37.2.4, 10.39.4.5.1, and 10.55.2) = width of a projecting flange element, angle, or stiffener (Articles 10.34.2.2, 10.34.4.7, 10.37.3.2, 10.39.4.5.1, 10.48.5.3, 10.51.5.5, and 10.55.3) = web buckling coefficient (Articles 10.34.4, 10.38.1.7, 10.48.5.3, and 10.48.8) = compressive force in the slab (Article 10.50.1.1.1) = equivalent moment factor (Article 10.54.2.1) = compressive force in top portion of steel section (Article 10.50.1.1.1) = bending coefficient (Table 10.32.1A and Articles 10.48.4.1 and 10.50.2.2) = column slenderness ratio dividing elastic and inelastic buckling (Table 10.32.1A) = coefficient about X axis (Article 10.36) = coefficient about the Y axis (Article 10.36) = buckling stress coefficient (Article 10.51.5.2) = clear distance between fl'anges, in. (Article 10.15.2) = clear unsupported distance between flange components (Articles 10.18.2.3.4,10.18.2.3.7, 10.18.2.3.8, 10.18.2.3.9, 10.34.3, 10.34.4, 10.34.5, 10.37.2, 10.48.1, 10.48.2, 10.48.4, 10.48.5,10.48.6,10.48.8,10.49.2,10.49.3.2, 10.50.1.1.2, 10.50.2.1, 10.55.2, and 10.61.1) = distance from the top of the slab to the neutral axis at which a composite section in positive bending theoretically reaches its plasticmoment capacity when the maximum strain in the slab is at 0.003 (Article 10.50.1.1.2) = clear distance between the neutral axis and the compression flange (Articles 10.34.3.2.1, 10.34.5.1, 10.48.4.1, 10.49.2, 10.49.3, 10.50(b), 10.57, and 10.61.1) = moments caused by dead load acting on composite girder (Article 10.50.1.2.2) = depth of the web in compression at the plastic moment (Articles 10.50(b), 10.50.1.1.2, and 10.50.2.1) = depth of the web in compression of the noncomposite steel beam or girder (Articles 10.34.5.1 and 10.49.3.2(a)) = distance from the top of the slab to the plastic neutral axis, in. (Article 10.50.1.1.2) = moments caused by dead load acting on steel girder (Article 10.50.1.2.2) = bolt diameter (Table 10.32.3B) = diameter of stud, in. (Article 10.38.5.1) and Articles 10.13, 10.48.2, 10.48.4.1, and 10.50.1.1.2) = diameter of rocker or roller, in.
(Article 10.32.4.2) = beam depth (Article 10.56.3) = column depth (Article 10.56.3) = spacing of intermediate stiffener (Articles 10.34.4, 10.34.5, 10.48.5.3, 10.48.6.3, and 10.48.8) = distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or the leg of the compression flange component (Articles 10.34.3.2.1, 10.34.5.1, 10.48.4.1, 10.49.3.2(a), and 10.61.1) = modulus of elasticity of steel, psi (Table 10.32.1AandArticles 10.15.3, 10.36, 10.37, 10.39.4.4.2, 10.54.1, and 10.55.1) = modulus of elasticity of concrete, psi (Article 10.38.5.1.2) = distance from the centerline of a splice to the centroid of the connection on the side of the joint under consideration(Articles 10.18.2.3.3, 10.18.2.3.5, and 10.18.2.3.7) = maximum induced stress in the bottom flange (Article 10.20.2.1) = maximum compressive stress, psi (Article 10.41.4.6) = allowable axial unit stress (Table 10.32.1A and Articles 10.36, 10.37.1.2, and 10.55.1) = allowable bending unit stress (Table 10.32.1A and Articles 10.18.2.2.3, 10.37.1.2, and 10.55.1) = compressive bending stress permitted about the X axis (Article 10.36) = compressive bending stress permitted about the Y axis (Article 10.36) = buckling stress of the compression flange plate or column (Articles 10.48.2, 10.50.2.2, 10.51.1, 10.51.5, 10.54.1.1, and 10.54.2.1) = local buckling stress of a stiffener (Articles 10.34.4.7 and 10.48.5.3) = design stress for the controlling flange at a point of splice (Articles 10.18.2.2.3 and 10.18.2.3.8) = design stress for the controlling flange at a point of splice (Articles 10.18.2.2.1 and 10.18.2.3.4) = maximum horizontal force (Article 10.20.2.2) = Euler buckling stress (Articles 10.37.1, 10.54.2.1, and 10.55.1) = Euler stress divided by a factor of safety (Ar- F : F.S. Fu ticle 10.36) = design stress for the noncontrolling flange at a point of splice (Article 10.18.2.2.3) = design stress for the noncontrolling flange at a point of splice (Article 10.18.2.2.1) = computed bearing stress due to design load (Table 10.32.3B) = limiting bending stress (Article 10.34.4) = allowable range of stress (Table 10.3.1A) = reduced allowable tensile stress on rivet or bolt due to the applied shear stress, ksi (Articles 10.32.3.3.4and 10.56.1.3.3) = specified minimum yield point of the reinforcing steel (Article 10.38.5.1.2) = factor of safety (Table 10.32.1A and Articles 10.32.1 and 10.36) = specified minimum tensile strength (Tables 10.2A, 10.32.1A and 10.32.3B and Article 10.18.4) = tensile strength of electrode classification (Table 10.56Aand Article 10.32.2) = maximum bending strength of the flange (Articles 10.48.8.2, 10.50.1.2.1, and 10.50.2.2) = allowable shear stress (Table 10.32.1A and 10.32.3B and Articles 10.18.2.3.6, 10.32.2, 10.32.3,10.34.4, 10.38.1.7,and 10.40.2.2) = shear strength of a fastener (Article 10.56.1.3) = combined tension and shear in bearing-type connections (Article 10.56.1.3) = design shear stress in the web at a point of splice (Articles 10.18.2.3.6,10.18.2.3.7,and 10.18.2.3.9) = specified minimum yield point of steel (Articles 10.15.2.1, 10.15.3, 10.16.11, 10.32.1, 10.32.4, 10.34, 10.35, 10.37.1.3, 10.38.1.7, fa 10.55.3) fb fb Fy stiffener Fy web f = computed compressive bending stress (Articles 10.34.2, 10.34.3, 10.34.5.2, 10.37, 10.39, and 10.55) = factored bending stress in the compression flange (Articles 10.48, 10.48.2.1@), 10.48.4.1, 10.50.1.2.1,10.50.2.2,10.53, and 10.53.1.2) fb f: fd fcu f, DL f, fdel 10.38.5,10.39.4,10.40.2.2,10.41.4.6,10.46, 10.48, 10.49, 10.50, 10.51.5, 10.54, and 10.61.4) = specified minimum yield strength of the flange (Articles 10.18.2.2.1, 10.48.1.1, 10.53.1, 10.57.1, and 10.57.2) = specified minimum yield strength of a transverse stiffener (Articles 10.34.4.7and 10.48.5.3) = specified minimum yield strength of the web (Articles 10.18.2.2.1,10.18.2.2.2,10.18.2.3.4, 10.53.1, and 10.61.1) = specified mimimum yield strength of the web (Articles 10.34.4.7 and 10.48.5.3) = the lesser of (f a) or Fy (Articles 10.48.2.1@),10.48.2.2,and 10.53) = computed axial compression stress (Articles 10.35.2.10, 10.36, 10.37, 10.55.2, and fWf fnCu f. f0f = maximum factored noncomposite dead load compressive bending stress in the web (Article 10.61.1) = unit ultimate compressive strength of concrete as determined by cylinder tests at age of 28 days, psi (Articles 10.38.1, 10.38.5.1.2, 10.45.3, and 10.50.1.1.1) = maximum flexural stress at the mid-thickness of the flange under consideration at a point of splice (Articles 10.18.2.2.3and 10.18.2.3.8) = maximum flexural stress due to the factored loads at the mid-thickness of the controlling flange at a point of splice (Articles 10.18.2.2.1 and 10.18.2.3.4) = noncomposite dead load stress in the compression flange (Articles 10.34.5.1 and 10.49.3.2(a)) = top flange compressive stress due to the factored noncomposite dead load divided by the factor & (Article 10.61.4) = total noncomposite and composite dead-load plus composite live-load stress in the compression flange at the most highly stressed section of the web (Articles 10.34.5.1 and 10.49.3.2(a)) = top flange compressive stress due to noncomposite dead load (Articles 10.34.2.1 and 10.34.2.2) = flexural stress at the mid-thickness of the noncontrolling flange concurrent with fcf(Articles 10.18.2.2.3and 10.18.2.3.8) = flexural stress due to the factored loads at the mid-thickness of the noncontrolling flange at a point of splice concurrent with f, (Articles 10.18.2.2.1and 10.18.2.3.4) = maximum flexural stress due to D + (L + I) at the mid-thickness of the flange under consideration at a point of splice (Articles 10.18.2.2.2and 10.18.2.3.5) = flexural stress due to D + (L + I) at the midthickness of the other flange at a point of splice concurrent with f, in the flange under consideration (Article 10.18.2.3.5) range of stress due to live load plus impact, in the slab reinforcement over the support (Article 10.38.5.1.3) = modulus of rupture of concrete specified in Article 8.15.2.1.1 (Article 10.50.2.3) = maximum longitudinal bending stress in the flange of the panels on either side of the transverse stiffener (Article 10.39.4.4) = factored bending stress in either the top or bottom flange, whichever flange has the larger ratio of (fp") (Article 10.48.8.2) = tensile stress due to applied loads (Articles 10.32.3.3.3 and 10.56.1.3.2) = allowable tensile stress in the concrete specified in Article 8.15.2.1.1 (Article 10.38.4.3) = unit shear stress (Articles 10.32.3.2.3, 10.34.4.4, and 10.34.4.7) = maximum shear stress in the web at a point of splice (Article 10.18.2.3.6) = computed compressive bending stress about the x axis (Article 10.36) = computed compressive bending stress about the y axis (Article 10.36) = gage between fasteners, in. (Articles 10.16.14,10.24.5, and 10.24.6) = height of stud, in.
(Article 10.38.5.1.1) = horizontal design force resultant in the web at a point of splice (Articles 10.18.2.3.8 and 10.1 8.2.3.9) = overload horizontal design force resultant in the web at a point of splice (Article 10.18.2.3.5) = horizontal design force resultant in the web at a point of splice (Articles 10.18.2.3.4 and 10.18.2.3.5) = average flange thickness of the channel flange, in. (Article 10.38.5.1.2) = moment of inertia, in.4 (Articles 10.34.4, 10.34.5, 10.38.5.1.1, 10.48.5.3, and 10.48.6.3) = moment of inertia of stiffener (Articles 10.37.2, 10.39.4.4.1, and 10.51.5.4) = moment of inertia of transverse stiffeners (Article 10.39.4.4.2) = moment of inertia of member about the vertical axis in the plane of the web, in4 (Article 10.48.4.1) = moment of inertia of compression flange about the vertical axis in the plane of the web, in4 (Table 10.32.1A and Article 10.48.4.1) = required ratio of rigidity of one transverse stiffener to that of the web plate (Articles 10.34.4.7 and 10.48.5.3) 4 = St. Venant torsional constant, in (Table 10.32.1A and Article 10.48.4.1) = fbx K = effective length factor in plane of buckling (Table 10.32.1A and Articles 10.37, 10.54.1, and 10.54.2) = effective length factor in the plane of bendKb ing (Article 10.36) k = constant: 0.75 for rivets; 0.6 for highstrength bolts with thread excluded from shear plane (Article 10.32.3.3.4) k = buckling coefficient (Articles 10.34.3.2.1, 10.34.4, 10.39.4.3, 10.48.4.1, 10.48.8, 10.51.5.4, and 10.61.1) k = distance from outer face of flange to toe of web fillet of member to be stiffened (Article 10.56.3) = buckling coefficient (Article 10.39.4.4) k, L = distance between bolts in the direction of the applied force (Table 10.32.3B) L = actual unbraced length (Table 10.32.1A and Articles 10.7.4, 10.15.3, and 10.55.1) L = 112 of the length of the arch rib (Article 10.37.1) L = distance between transverse beams (Article 10.41.4.6) = unbraced length (Table 10.48.2.1 .A and ArtiLb cles 10.36, 10.48.1.1, 10.48.2.1, 10.48.4.1, and 10.53.1.3) = length of member between points of support, LC in. (Article 10.54.1.1) = clear distance between the holes, or between L the hole and the edge of the material in the direction of the applied bearing force, in. (Table 10.32.3B and Article 10.56.1.3.2) = limiting unbraced length (Article 10.48.4.1) L~ = limiting unbraced length (Article 10.48.4.1) Lr e = member length (Table 10.32.1A and Article 10.35.1) M = maximum bending moment (Articles 10.48.8, 10.54.2.1, and 10.50.1.1.2) = smaller moment at the end of the unbraced MI length of the member (Article 10.48.1.l(c)) MI & M2 = moments at two adjacent braced points (Tables 10.32.1A and 10.36A and Articles 10.48.4.1 and 10.50.2.2) = column moment (Article 10.56.3.2) I% = full plastic moment of the section (Articles M~ 10.50.1.1.2 and 10.54.2.1) = lateral torsional buckling moment or yield Mr moment (Articles 10.48.2, 10.48.4.1, 10.50.1.2.1, 10.50.2.2, and 10.53.1.3) = elastic pier moment for loading producing Ms maximum positive moment in adjacent span (Article 10.50.1.1.2) Mu = maximum bending strength (Articles 10.18.2.2.1, 10.48, 10.49, 10.50.1, 10.50.2, 10.51.1, 10.53.1, 10.54.2.1, and 10.61.3) = design moment due to the eccentricity of the design shear at a point of splice (Articles 10.18.2.3.7 and 10.18.2.3.9) = overload design moment due to the eccentricity of the overload design shear at a point of splice (Article 10.18.2.3.5) = design moment due to the eccentricity of the design shear at a point of splice (Articles 10.18.2.3.3 and 10.18.2.35) = design moment at a point of splice representing the portion of the flexural moment assumed to be resisted by the web (Articles 10.18.2.3.8 and 10.18.2.3.9) = overload design moment at a point of splice representing the portion of the flexural moment assumed to be resisted by the web (Article 10.18.2.3.5) = design moment at a point of splice representing the portion of the flexural moment assumed to be resisted by the web (Articles 10.18.2.3.4 and 10.18.2.3.5) = moment capacity at first yield (Articles 10.18.2.2.1, 10.50.1.1.2, and 10.61.3) = number of shear connectors (Article 10.38.5.1.2) = number of additional connectors for each beam at point of contraflexure (Article 10.38.5.1.3) = number of slip planes in a slip-critical connection (Articles 10.32.3.2.1 and 10.57.3.1) = number of roadway design lanes (Article 10.39.2) = ratio of modulus of elasticity of steel to that of concrete (Article 10.38.1) = number of longitudinal stiffeners (Articles 10.39.4.3, 10.39.4.4, and 10.51.5.4) = allowable compressive axial load on members (Article 10.35.1) = axial compression on the member (Articles 10.48.1.1, 10.48.2.1, and 10.54.2.1) = force in the slab (Article 10.38.5.1.2) = design force in the controlling flange at a point of splice (Article 10.18.2.2.3) Ps Pu = allowable slip resistance (Article 10.32.3.2.1) = maximum axial compression capacity (Arti- Pu = design force for checking the strength of a cle 10.54.1.1) P Q Q R R R R Rb & %, Rev R Rw r rb ry rr = design force for the controlling flange at a point of splice (Article 10.18.2.2.1) = overload design force in the flange at a point of splice (Article 10.18.2.2.2) = design force for the noncontrolling flange at a point of splice (Article 10.18.2.2.3) = design force in the noncontrolling flange at a point of splice (Article 10.18.2.2.1) = design force for checking slip of abolted splice in a tension member (Article 10.18.4.2) S S S sr bolted splice in a tension member (Article 10.18.4.1) = allowable bearing (Article 10.32.4.2) = prying tension per bolt (Articles 10.32.3.3.2 and 10.56.2) = statical moment about the neutral axis (Article 10.38.5.1.1) = radius (Article 10.15.2.1) = number of design lanes per box girder (Article 10.39.2.1) = reduction factor for hybrid girders (Articles 10.18.2.2.1, 10.18.2.2.2, 10.18.2.2.3, 10.18.2.3.4, 10.18.2.3.8, 10.40.2.1.1, 10.53.1.2, and 10.53.1.3) = reduction factor applied to the design shear strength of fasteners passing through fillers (Article 10.18.1.2.1) = bending capacity reduction factor (Articles 10.48.2, 10.48.4.1, 10.50.1.2.1, 10.50.2.2, 10.53.1.2, 10.53.1.3, and 10.61.4) = absolute value of the ratio of Fcfto fd for the controlling flange at a point of splice (Articles 10.18.2.2.3 and 10.18.2.3.8) = the absolute value of the ratio of F, to f, for the controlling flange at a point of splice (Articles 10.18.2.2.1 and 10.18.2.3.4) = a range of stress involving both tension and compression during a stress cycle (Table 10.3.1B) = vertical force at connections of vertical stiffeners to longitudinal stiffeners (Article 10.39.4.4.8) = vertical web force (Article 10.39.4.4.7) = radius of gyration, in (Articles 10.35.1, 10.37.1, 10.41.4.6, 10.48.6.3, 10.54.1.1, 10.54.2.1, and 10.55.1) = radius of gyration in plane of bending, in.
(Article 10.36) = radius of gyration with respect to the Y-Y axis, in. (Article 10.48.1.1) = radius of gyration of the compression flange about the axis in the plane of the web, in. (Table 10.32.1A and Article 10.48.4.1) = allowable rivet or bolt unit stress in shear (Article 10.32.3.3.4) = section modulus, in.' (Articles 10.48.2, 10.51.1, 10.53.1.2, and 10.53.1.3) = pitch of any two successive holes in the chain (Article 10.16.14.2) =range of horizontal shear (Article 10.38.5.1.1) = section modulus of transverse stiffener, in.' (Articles 10.39.4.4 and 10.48.6.3) = section modulus of longitudinal or transverse stiffener, in.3(Article 10.48.6.3) = ultimate strength of the shear connector (Article 10.38.5.1.2) = section modulus with respect to the compression flange, in.' (Table 10.32.1A and Articles 10.48.2, 10.48.4.1, 10.50.1.2.1, 10.50.2.2 and 10.53.1.2) = section modulus with respect to the tension flange, in.' (Articles 10.48.2 and 10.53.1.2) = computed rivet or bolt unit stress in shear (Article 10.32.3.3.4) = range in tensile stress (Table 10.3.1B) = direct tension per bolt due to external load (Articles 10.32.3 and 10.56.2) = arch rib thrust at the quarter point from dead +live+ impact loading (Articles 10.37.1 and 10.55.1) = thickness of the thinner outside plate or shape (Article 10.35.2) = thickness of members in compression (Article 10.35.2) = thickness of thinnest part connected, in (Articles

radius of gyration shall not exceed 200 for main members, shall not exceed 240 for bracing members, and shall not exceed 140 for main members subject to a reversal of stress. 10.8 MINIMUM THICKNESS OF METAL 500,000 "Average Daily Truck Traffic (one direction). bLmgitudinalmembers should also be checked for truck loading. also be investigated for "over stress cycles produced by placing a single truck on the bridge distributed to the girders as designated in Article 3.23.2 for one traffic lane loadina. The she& in steel girder webs shall not exceed 0.58 F,.D&C for this single truck loading. 10.7.2 In determining the radius of gyration, r, for the purpose of applying the limitations of the KLlr ratio, the area of any portion of a member may be neglected provided that the strength of the member as calculated without using the area thus neglected and the strength of the member as computed for the entire section with the KLlr ratio applicable thereto, both equal or exceed the computed total force that the member must sustain. 10.7.3 The radius of gyration and the effective area for canying stress of a member containing perforated cover plates shall be computed for a transverse section through the maximum width of perforation. When perforations are staggered in opposite cover plates, the cross-sectional area of the member shall be considered the same as for a section having perforations in the same transverse plane. 10.7.4 Actual unbraced length, L, shall be assumed as follows: For the top chords of half-through trusses, the length between panel points laterally supported as indicated under Article 10.16.12; for other main members, the 10.8.1 Structural steel (including bracing, cross frames, and all types of gusset plates), except for webs of certain rolled shapes, closed ribs in orthotropic decks, fillers, and in railings, shall be not less than 5/16 inch in thickness. The web thickness of rolled beams or channels shall not be less than 0.23 inches. The thickness of closed ribs in orthotropic decks shall not be less than y16 inch. 10.8.2 Where the metal will be exposed to marked corrosive influences, it shall be increased in thickness or specially protected against corrosion. 10.8.3 It should be noted that there are other provisions in this section pertaining to thickness for fillers, segments of compression members, gusset plates, etc. As stated above, fillers need not be 5/16 inch minimum. 10.8.4 For compression members, refer to "Trusses" (Article 10.16). 10.8.5 For stiffeners and other plates, refer to "Plate Girders" (Article 10.34). 10.8.6 For stiffeners and outstanding legs of angles, etc., refer to Article 10.10. 10.9 EFFECTIVE AREA OF ANGLES AND TEE SECTIONS IN TENSION 10.9.1 The effective area of a single angle tension member, a tee section tension member, or each angle of a dou- 266 HIGHWAY BRIDGES ble angle tension member in which the shapes are connected back to back on the same side of a gusset plate shall be assumed as the net area of the connected leg or flange plus one-half of the area of the outstanding leg. 10.9.2 If a double angle or tee section tension member is connected with the angles or flanges back to back on opposite sides of a gusset plate, the full net area of the shapes shall be considered effective. 10.9.3 When angles connect to separate gusset plates, as in the case of a double-webbed truss, and the angles are connected by stay plates located as near the gusset as practicable, or by other adequate means, the full net area of the angles shall be considered effective. If the angles are not so connected, only 80% of the net areas shall be considered effective. 10.9.1 under Strength Design as specified in Articles 10.48.1, 10.50.1.l, and 10.50.2.1. When computing the strength of a flexural member at a section with holes in the tension flange, an effective flange area, &,specified by Equation (10-4g) shall be used for that flange in computing the elastic section properties. The diameter of the holes shall be taken as specified in Article 10.16.14.6. In the case of the strength design method, the strength of compact sections with holes in the tension flange shall not be taken greater than the moment capacity at first yield. 10.13 COVER PLATES 10.13.1 The length of any cover plate added to a rolled beam shall be not less than (2di-3) feet, where (d) is the depth of the beam in feet. 10.9.4 Lug angles may be considered as effective in transmitting stress, provided they are connected with at least one-third more fasteners than required by the stress to be carried by the lug angle. 10.13.2 Partial length welded cover plates shall not be used on flanges more than 0.8 inches thick for nonredundant load path structures subjected to repetitive loadings that produce tension or reversal of stress in the member. 10.10 OUTSTANDING LEGS OF ANGLES 10.13.3 The maximum thickness of a single cover plate on a flange shall not be greater than two times the thickness of the flange to which the cover plate is attached. The total thickness of all cover plates should not be greater than 2% times the flange thickness. The widths of outstanding legs of angles in compression (except where reinforced by plates) shall not exceed the following: In main members carrying axial stress, 12 times the thickness. In bracing and other secondary members, 16 times the thickness. For other limitations, see Article 10.35.2. 10.11 EXPANSION AND CONTRACTION In all bridges, provisions shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. Provisions shall be made for changes in length of span resultingfrom live load stresses. In spans more than 300 feet long, allowance shall be made for expansion and contraction in the floor. The expansion end shall be secured against lateral movement. 10.12 FLEXURAL MEMBERS Flexural members shall be designed using the elastic section modulus except when utilizing compact sections 10.13.4 Any partial length welded cover plate shall extend beyond the theoretical end by the terminal distance, and it shall extend to a section where the stress range in the beam flange is equal to the allowable fatigue stress range for base metal adjacent to or connected by fillet welds. The theoretical end of the cover plate, when using service load design methods, is the section at which the stress in the flange without that cover plate equals the allowable service load stress, exclusive of fatigue considerations. When using strength design methods, the theoretical end of the cover plate is the section at which the flange strength without that cover plate equals the required strength for the design loads, exclusive of fatigue requirements.

The terminal distance is two times the nominal cover plate width for cover plates not welded across their ends, and 1% times for cover plates welded across their ends.

The width at ends of tapered cover plates shall be not less than 3 inches. The weld connecting the cover plate to the flange in its terminal distance shall be continuous and of sufficient size to develop a total stress of not less than the computed stress in the cover plate at its theoretical end.

All welds connecting cover plates to beam flanges shall be continuous and shall not be smaller than the minimum size permitted by Article 10.23.2. 10.13.5 DIVISION I-DESIGN - - -- 267 -- 10.13.5 Any partial length end-bolted cover plate shall extend beyond the theoretical end by a terminal distance equal to the length of the end-bolted portion, and the cover plate shall extend to a section where the stress range in the beam flange is equal to the allowable fatigue stress range for base metal at ends of partial length welded cover plates with high-strength bolted, slip-critical end connections (Table 10.3.1B). Beams with end-bolted cover plates shall be fabricated in the following sequence: drill holes; clean faying surfaces; install bolts; weld. The theoretical end of the end-bolted cover plate is determined in the same manner as that of a welded cover plate, as is specified in Article 10.13.4. The bolts in the slip-critical connections of the cover plate ends to the flange, shall be of sufficient numbers to develop a total force of not less than the computed force in the cover plate at the theoretical end. The slip resistance of the end-bolted connection shall be determined in accordance with Article 10.32.3.2 for service load design, and Article 10.56.1.4 for load factor design. The longitudinal welds connecting the cover plate to the beam flange shall be continuous and stop a distance equal to one bolt spacing before the first row of bolts in the endbolted portion. 10.14 CAMBER Girders should be cambered to compensate for dead load deflections and vertical curvature required by profile grade. R= 10.15.1 Scope This section pertains to rolled beams and welded I-section plate girders heat-curved to obtain a horizontal curvature. Steels that are manufactured to a specified minimum yield point greater than 50,000 psi, except for Grade HPS70W steel, shall not be heat-curved.

10.15.2 Minimum Radius of Curvature 10.15.2.1 For heat-curved beams and girders, the horizontal radius of curvature measured to the center line of the girder web shall not be less than 150 feet and shall not be less than the larger of the values calculated (at any and all cross sections throughout the length of the girder) from the following two equations: &W (10 - 1) tw In these equations, F, is the specified minimum yield point in kips per square inch of steel in the girder web, $ is the ratio of the total cross-sectional area to the crosssectional area of both flanges, b is the widest flange width in inches, D is the clear distance between flanges in inches, t,is the web thickness in inches, and R is the radius in inches. 10.15.2.2 In addition to the above requirements, the radius shall not be less than 1,000 feet when the flange thickness exceeds 3 inches or the flange width exceeds 30 inches. 10.15.3 Camber To compensate for possible loss of camber of heatcurved girders in service as residual stresses dissipate, the amount of camber in inches, A at any section along the length L of the girder shall be equal to: 1,000- R 850 EYO AR = 0 for radii greater than 1,000 AR = 10.15 HEAT-CURVED ROLLED BEAMS AND WELDED PLATE GIRDERS 14bD 0.02 L'F, (where AD, is the camber in inches at any point along the length L calculated by usual procedures to compensate for deflection due to dead loads or any other specified loads; AMis the maximum value of ADLin inches within the length L; E is the modulus of elasticity in ksi; F, is the specified minimum yield point in h i of the girder flange; Yois the distance from the neutral axis to the extreme outer fiber in inches (maximum distance for nonsyrnmetrical sections); R is the radius of curvature in feet; and L is the span length for simple spans or for continuous spans, the distance between a simple end support and the dead load contraflexure point, or the distance between points of dead load contraflexure. (L is measured in inches.) Camber loss between dead load contraflexure points adjacent to piers is small and may be neglected.

Note: Part of the camber loss is attributable to construction loads and will occur during construction of the 270 10.16.9.2 HIGHWAY BRIDGES with Article 10.3. Terminations of butt welds shall be ground smooth. 10.16.9.3 The separate segments of tension members composed of shapes may be connected by perforated plates or by stay plates or end stay plates and lacing.

End stay plates shall have the same minimum length as specified for end stay plates on main compression members, and intermediate stay plates shall have a minimum length of % of that specified for intermediate stay plates on main compression members. The clear distance between stay plates on tension members shall not exceed 3 feet. 10.16.9.4 The thickness of stay plates shall be not less than %O of the distance between points of support for main members, and Ym of that distance for bracing members. Stay plates shall be connected by not less than three fasteners on each side, and in members having lacing bars the last fastener in the stay plates preferably shall also pass through the end of the adjacent bar. 10.16.10 Lacing Bars When lacing bars are used, the following provisions shall govern their design. 10.16.10.1 Lacing bars of compression members shall be so spaced that the slenderness ratio of the portion of the flange included between the lacing bar connections will be not more than 40 or more than Y3 of the slenderness ratio of the member. 10.16.10.2 The section of the lacing bars shall be determined by the formula for axial compression in which L is taken as the distance along the bar between its connections to the main segments for single lacing, and as 70% of that distance for double lacing. 10.16.10.3 If the distance across the member between fastener lines the flanges is more than l5 inches and a bar with a single fastener in the connection is used, the lacing shall be double and fastened at the intersections. 10.16.10.4 The angle between the lacing bars and the axis of the member shall be approximately 45" for double lacing and 60" for single lacing. 10.16.10.5 Lacing bars may be shapes or flat bars. For main members, the minimum thickness of flat bars shall be YN of the distance along the bar between its connections for single lacing and Ym for double lacing. For o single lacing bracing members, the limits shall be Y ~for and Y75 for double lacing. 10.16.10.6 The diameter of fasteners in lacing bars shall not exceed one-third the width of the bar. There shall be at least two fasteners in each end of lacing bars connected to flanges more than 5 inches in width. 10.16.11 Gusset Plates 10.16.11.1 Gusset or connection plates preferably shall be used for connecting main members, except when the members are pin-connected. The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the full development of the elements of the member shall be given consideration. The gusset plates shall be of ample thickness to resist shear, direct stress, and flexure acting on the weakest or critical section of maximum stress. 10.16.11.2 Re-entrant cuts, except curves made for appearance, shall be avoided as far as practicable. 10.16.11.3 If the length of unsupported edge of a gusset plate exceeds the value of the expression 11,000/* times its thickness, the edge shall be stiffened. 10.16.11.4 Listed below are the values of the expression 11,000/* for the following grades of steel: 36,000 psi, Y.P. Min 58 50,000 psi, Y.P. Min 49 70,000 psi, Y.P. Min 42 90,000 psi, Y.P. Min 37 100,000 psi, Y.P. Min 35 10.16.12 Half-Through Truss Spans 10.16.12.1 The vertical truss members and the floor beams and their connections in half-through mss spans shall be proportioned to resist a lateral force of not less than 300 pounds per linear foot applied at the top chord panel points of each truss. 10.16.123 The top chord shall be considered as a column with elastic lateral supports at the panel points. The critical buckling force of the column, so determined, shall exceed the maximum force from dead load, live load, and impact in any panel of the top chord by not less than 50%.* *For a discussion of columns with elastic lateral supports, refer to Timoshenko& Gere, "Theory of Elastic Stability,"McGtaw-HillBook CO., Fit Edition, p. 122. DIVISION I-DESIGN 10.16.13 271 10.16.13 Fastener Pitch in Ends of Compression Members shall be considered in determining the unit stress on the net section. In the ends of compression members, the pitch of fasteners connecting the component parts of the member shall not exceed four times the diameter of the fastener for a length equal to 1% times the maximum width of the member. Beyond this point, the pitch shall be increased gradually for a length equal to 1% times the maximum width of the member until the maximum pitch is reached. 10.16.14.6 The diameter of the hole shall be taken as inch greater than the nominal diameter of the rivet or in achigh-strength bolt, unless larger holes are d with ~ ~ 10.24. ~ i ~ l ~ cor ance 10.16.14 Net Section of Riveted or High-Strength Bolted Tension Members 10.16.14.1 The net section of a riveted or highstrength bolted tension member is the sum of the net sections of its component parts. The net section of a part is the product of the thickness of the part multiplied by its lea; net width. 10.16.14.2 The net width for any chain of holes extending progressively across the part shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the chain and adding, for each gage space in the chain, the quantity: f/8 10.17 BENTS AND TOWERS 10.17.1 General Bents preferably shall be composed of two supporting columns, and the bents usually shall be united in pairs to form towers. The design of members for bents and towers is governed by applicable articles. 10.17.2 Single Bents Single bents shall have hinged ends or else shall be designed to resist bending. 10.17.3 Batter s2 - (10 - 4) 4g where: S = pitch of any two successive holes in the chain; g = gage of the same holes. The net section of the part is obtained from the chain that gives the least net width.

10.16.14.3 For angles, the gross width shall be the sum of the widths of the legs less the thickness. The gage for holes in opposite legs shall be the sum of gages from back of angle less the thickness.

10.16.14.4 At a splice, the total stress in the member being spliced is transferred by fasteners to the splice material.

10.16.14.5 When determining the unit stress on any least net width of either splice material or member being spliced, the amount of the stress previously transferred by fasteners adjacent to the section being investigated Bents preferably shall have a sufficient spread at the base to prevent uplift under the assumed lateral loadings. In general, the width of a bent at its base shall be not less than one-third of its height. 10.17.4 Bracing 10.17.4.1 Towers shall be braced, both transversely and longitudinally, with stiff members having either welded, high-strength bolted or riveted connections. The sections of members of longitudinal bracing in each panel shall not be less than those of the members in corresponding panels of the transverse bracing. 10.17.4.2 The bracing of long columns shall be designed to fix the column about both axes at or near the same point. 10.17.4.3 Horizontal diagonal bracing shall be placed in all towers having more than two vertical panels, at alternate intermediate panel points. 272 HIGHWAY BRIDGES 10.17.5 Bottom Struts The bottom struts of towers shall be strong enough to slide the movable shoes with the structure unloaded, the coefficient of friction being assumed at 0.25. Provision for expansion of the tower bracing shall be made in the column bearings. 10.18 SPLICES 10.18.1 General 10.18.1.1 Design Strength Splices may be made by rivets, by high-strength bolts or by the use of welding. In general, splices whether in tension, compression, bending, or shear, shall be designed in the case of the service load design or strength design methods for a capacity based on not less than the average of the required design strength at the point of splice and the design strength of the member at the same point but, in any event, not less than 75% of the design strength of the member, except as specified herein. Bolted splices in flexural members shall satisfy the requirements of Article 10.18.2. Bolted splices in compression members shall satisfy the requirements of Article 10.18.3. Bolted splices in tension members shall satisfy the requirements of Article 10.18.4. Welded splices shall satisfy the requirements of Article 10.18.5. Where a section changes at a splice, the smaller section is to be used to satisfy the above splice requirements. 10.18.1.2 Fillers 10.18.1.2.1 For fillers % inch and thicker in bolted or riveted axially loaded connections, including girder flange splices, additional fasteners shall be required to distribute the total stress in the member uniformly over the combined section of the member and the filler. The filler shall either be extended beyond the splice material and secured by additional bolts, or as an alternate to extending the filler, an equivalent number of bolts may be included in the connection. Fillers '/4 inch and thicker need not be extended and developed provided that the design shear strength of the fasteners, specified in Article 10.56.1.3.2 in the case of the strength design method and in Table 10.32.3B in the case of the service load design method, is reduced by the following factor R: where: y =' A A, 10.17.5 Af = sum of the area of the fillers on the top and bottom of the connected plate A, = smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate The design slip force, specified in Article 10.57.3.1 in the case of the strength design method and in Article 10.32.3.2.1 in the case of the service load design method, for slip-critical connections shall not be adjusted for the effect of the fillers. Fillers % inch or more in thickness shall consist of not more than two plates, unless special permission is given by the Engineer.

10.18.1.2.2 For bolted web splices with thickness differences of %6 inch or less, no filler plates are required. 10.18.1.2.3 Fillers for welded splices shall conform to the requirements of the ANSVAASHTO/AWS D1.5 Bridge Welding Code. 10.18.1.3 Design Force for Flange Splice Plates For a flange splice with inner and outer splice plates, the flange design force may be assumed to be divided equally to the inner and outer plates and their connections when the areas of the inner and outer plates do not differ by more than 10%. When the areas of the inner and outer plates differ by more than lo%, the design force in each splice plate and its connection shall be determined by multiplying the flange design force by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates. For this case, the shear strength of the connection shall be checked for the maximum calculated splice plate force acting on a single shear plane. The slip resistance of high-strength bolted connections for a flange splice with inner and outer splice plates shall always be checked for the flange design force divided equally to the two slip planes. 10.18.1.4 Truss Chords and Columns Splices in truss chords and columns shall be located as near to the panel points as practicable and usually on the side where the smaller stress occurs. The arrangement of plates, angles, or other splice elements shall be such as to make proper provision for the stresses, both axial and bending, in the component parts of the members spliced. 10.18.2 DIVISION I-DESIGN 10.18.2 Flexural Members 10.18.2.1 General 10.18.2.1.1 In continuous spans, splices shall preferably be made at or near points of dead-load contraflexure. 10.18.2.1.2 In both flange and web splices, there shall be not less than two rows of bolts on each side of the joint. 10.18.2.1.3 Oversize or slotted holes shall not be used in either the member or the splice plates at bolted splices. 10.18.2.1.4 In both flange and web splices, highstrength bolted connections shall be proportioned to prevent slip during erection of the steel and during the casting or placing of the deck. 10.18.2.1.5 In the case of the strength design method, the strength of compact sections at the point of splice shall not be taken greater than the moment capacity at first yield, computed by accounting for the holes in the tension flange as specified in Article 10.12. 10.18.2.1.6 Flange and web splices in areas of stress reversal shall be checked for both positive and negative flexure. 10.18.2.1.7 Riveted and bolted flange angle splices shall include two angles, one on each side of the flexural member. 10.18.2.2.1 As a minimum, in the case of the strength design method, the splice plates on the controlling flange shall be proportioned for a design force, PC,.The controlling flange shall be taken as the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its mid-thickness due to the factored loads to its maximum strength. PC,shall be taken equal to a design stress, Fcu, times the smaller effective flange area, A,, on either side of the splice. A, is defined in Article 10.18.2.2.4 and Fcuis defined as follows: (Ifcu/~I+ @fl) 2 0.75% 2 Mu= maximum bending strength of the section in positive or negative flexure at the point of splice, whichever causes the maximum compressive stress due to the factored loads at the mid-thickness of the flange under consideration My= moment capacity at first yield for the section at the point of splice used to compute Mu.For composite sections, Myshall be calculated in accordance with Article 10.50(c). For hybrid sections, Myshall be computed in accordance with Article 10.53. fcu = maximum elastic flexural stress due to the factored loads at the mid-thickness of the controlling flange at the point of splice. R = reduction factor for hybrid girders specified in Article 10.53.1.2. R shall be taken equal to 1.0 when f,, is less than or equal to F, where F, is equal to the specified minimum yield strength of the web. For homogeneous girders, R shall always be taken equal to 1.0. Ffl = specified minimum yield strength of the flange As a minimum, the splice plates for the noncontrolling flange shall be proportioned for a design force, Pncu.Pncu shall be taken equal to a design stress, Fncu,times the smaller effective flange area, A,, on either side of the splice. Fncuis defined as follows: Fncu= Rc, (If,,, / Rl) 2 0.75aFfl (10 - 4c) where: Ru= the absolute value of the ratio of Fcuto fcufor the 10.18.2.2 Flange Splices Fcu = 273 (10 - 4b) where: a = 1.0 except that a lower value equal to (MUMy) may be used for flanges in compression at sections where Muis less than My. controlling flange. fncu= flexural stress due to the factored loads at the mid-thickness of the noncontrolling flange at the point of splice concurrent with fcu In calculating fcu,f,,,, Mu, My and R, holes in the flange subject to tension shall be accounted for as specified in Article 10.12. For a flange splice with inner and outer splice plates, the flange design force shall be proportioned to the inner and outer plates and their connections as specified in Article 10.18.1.3. The effective area, A,, of each splice plate shall be sufficient to prevent yielding of the splice plate under its calculated portion of the design force. A, of each splice plate shall be taken as defined in Article 10.18.2.2.4. As a minimum, the connections for both the top and bottom flange splices shall be proportioned to develop the design force in the flange through shear in the bolts and bearing at the bolt holes, as specified in Article 10.56.1.3.2. Where filler plates are required, the requirements of Article 10.18.1.2.1 shall also be satisfied. 274 HIGHWAY BRIDGES 10.18.2.2.2 As a minimum, in the case of the strength design method, high-strength bolted connections for both top and bottom flange splices shall be proportioned to prevent slip at an overload design force, Pfo.For the flange under consideration, Pfoshall be computed as follows: where: , f, = maximum flexural stress due to D + P,(L + I) at the mid-thickness of the flange under consideration for the smaller section at the point of splice, where p, is defined in Article 3.22 R = reduction factor for hybrid girders specified in Article 10.53.1.2. R shall be taken equal to 1.0 when fois less than or equal to F, where F, is equal to the specified minimum yield strength of the web. For homogeneous girders, R shall always be taken equal to 1.0. A, = smaller gross flange area on either side of the splice foand R shall be computed using the gross section of the member. The slip resistance of the connection shall be computed from Equation (10-172).

10.18.2.2.3 As a minimum, in the case of the service load design method, the splice plates on the controlling flange shall be proportioned for a design force, PC,.The controlling flange shall be taken as the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its mid-thickness to its allowable stress. Pcf shall be taken equal to a design stress, Fd, times the smaller effective flange area, 4,on either side of the splice. 4 is defined in Article 10.18.2.2.4 and Fcfis defined as follows: 10.18.2.2.2 As a minimum, the splice plates for the noncontrolling flange shall be proportioned for a design force, Pncf.Pncf shall be taken equal to a design stress, FnC,times the smaller effective flange area, A,, on either side of the splice. Fncfis defined as follows: where: Rf = the absolute value of the ratio of Fcfto fcffor the controlling flange fncf= flexural stress at the mid-thickness of the noncontrolling flange at the point of splice concurrent with fcf In calculating Fcf,fmfand R, holes in the flange subject to tension shall be accounted for as specified in Article 10.12. For a flange splice with inner and outer splice plates, the flange design force shall be proportioned to the inner and outer plates and their connections as specified in Article 10.18.1.3. The effective area, 4 , of each splice plate shall be sufficient to ensure that the stress in the splice plate does not exceed the allowable flexural stress under its calculated portion of the design force. A, of each splice plate shall be taken as defined in Article 10.18.2.2.4. As a minimum, the connections for both the top and bottom flange splices shall be proportioned to develop the design force in the flange through shear in the bolts and bearing at the bolt holes, as specified in Table 10.32.3B. Where filler plates are required, the requirements of Article 10.18.1.2.1 shall also be satisfied. As a minimum, high-strength bolted connections shall also be proportioned to prevent slip at a force equal to the maximum elastic flexural stress due to D + (L + I) at the midthickness of the flange under consideration for the smaller section at the point of splice times the smaller value of the gross flange area on either side of the splice.

The slip resistance of the connection shall be determined as specified in Article 10.32.3.2.1. A where: fcf= maximum elastic flexural stress at the mid-thickness of the controlling flange at the point of splice. Fb= allowable flexural stress for the flange under consideration at the point of splice R = reduction factor for hybrid girders specified in Article 10.40.2.1. R shall be taken equal to 1.0 when fcfis less than or equal to the allowable flexural stress for the web steel. For homogeneous girders, R shall always be taken equal to 1.O. 10.18.2.2.4 For checking the strength of flange splices, an effective area, 4 , shall be used for the flange and for the individual splice plates as follows: For flanges and their splice plates subject to tension: where: W, = least net width of the flange or splice plate computed as specified in Article 10.16.14 DIVISION I-DESIGN 10.18.2.2.4 t = flange or splice plate E k n e s s A, = gross area of the flange or splice plate p = 0.0 for M 270 Grade 100/100W steels, or when holes exceed 1 % inch in diameter. = 0.15 for all other steels and when holes are less than or equal to 1 % inch in diameter. 275 A -S a minimum, in the case of the strengfh----design method, web splice plates and their connections shall be proportioned for a design moment, M,, due to the eccentricity of the design shear at the point of splice defined as Mvu= Vwue The diameter of the holes shall be taken as specified in Article 10.16.14.6. For the flanges and their splice plates subject to compression: A, =A, (10 - 4h) 10.18.2.3 Web Splices 10.18.2.3.1 In general, web splice plates and their connections shall be proportioned for shear, a moment due to the eccentricity of the shear at the point of splice, and a portion of the flexural moment that is assumed to be resisted by the web at the point of splice.* Webs shall be spliced symmetrically by plates on each side. The web splice plates shall extend as near as practical for the full depth between flanges. where: V, = design shear in the web at the point of splice defined in Article 10.18.2.3.2 e = distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration 10.18.2.3.4 As arninimum, in the case of the strength design method, web splice plates and their connections shall be proportioned for a design moment at the point of splice, M, representing the portion of the flexural moment that is assumed to be resisted by the web. M, shall be applied at the mid-depth of the web. For sections where the neutral axis is not located at mid-depth of the web, a horizontal design force resultant in the web at the point of splice, H, shall also be applied at the mid-depth of the web. M, and H, may be computed as follows: t D~ M ~ = 12 - IRKu- Rcufncu I 10.18.2.3.2 As a minimum, in the case of the strength design method, web splice plates and their connections shall be proportioned for a design shear in the web at the point of splice, V, defined as follows: For V < 0.5Vu: For V 2 0.5Vu: where: V = maximum shear in the web at the point of splice due to the factored loads Vu= shear capacity of the web at the point of splice *For an alternative approach for compact steel sections, reference is made to Firas I. Sheikh-Ibrahim and Karl H. Frank, "The Ultimate Strength of Symmetric Beam Bolted Splices," AISC Engineering Journal, 3rd Quarter, 1998, and 'The Ultimate Strength of Unsymmetric Beam Bolted Splices,"AISC Engineering J o u m l , 2nd Quader, 2001. (10 - 4k) (10 - 41) where: F, = design stress for the controlling flange at the point of splice defined in Article 10.18.2.2.1 (positive for tension; negative for compression) R = reduction factor for hybrid girders specified in Article 10.53.1.2. R shall be taken equal to 1.0 when fcuis less than or equal to F, where F, is equal to the specified minimum yield strength of the web. For homogeneous girders, R shall always be taken equal to l .O. Ku=the absolute value of the ratio of F,, to fcufor the controlling flange fncu=flexural stress due to the factored loads at the mid-thickness of the noncontrolling flange at the point of splice concurrent with fcu(positive for tension; negative for compression) 10.18.2.3.5 As a minimum, in the case of the strength design method, web splice plates and their connections shall be proportioned to develop the most critical combi- 276 HIGHWAY BRIDGES nation of V, M,, M, and H., The connections shall be proportioned as eccentrically loaded connections to develop the resultant design force through shear in the bolts and bearing at the bolt holes, as specified in Article 10.56.1.3.2. In addition, as a minimum, high-strength bolted connections for web splices shall be proportioned as eccentrically loaded connections to prevent slip under the most critical combination of: 1) an overload design shear, V,,, 2) an overload design moment, Mvo,due to the eccentricity of the overload design shear, 3) an overload design moment, M,,, applied at mid-depth of the web representing the portion of the flexural moment that is assumed to be resisted by the web, and 4) for sections where the neutral axis is not located at the mid-depth of the web, an overload horizontal design force resultant, H,,, applied at mid-depth of the web, as follows: (10 - 4n) VWO = VO where: + V, = maximum shear in the web due to D PL(L+I) at the point of the splice, where P, is defined in Article 3.22 (10 - 40) Mvo = Vwoe 10.18.2.3.6 As a minimum, in the case of the service load design method, web splice plates and their connections shall be proportioned for a design shear stress in the web at the point of splice, F,, defined as follows: For fvc 0.5Fv: For f, 2 0.5Fv: where: fv = maximum shear stress in the web at the point of splice Fv= allowable shear stress in the web at the point of splice 10.18.2.3.7 As a minimum, in the case of the service load design method, web splice plates and their connections shall be proportioned for a design moment, Mv,due to the eccentricity of the design shear at the point of splice defined as follows: M,, and H,, may be computed as follows: MV= FwDtwe M wo ="-t D~ If0 - fof I (10 - 4 ~) t D Hwo=L(fo+f 2 (10 - 4 4 12 of) where: f, + maximum flexural stress due to D PL(L+I)at the mid-thickness of the flange under consideration for the smaller section at the point of splice (positive for tension; negative for compression) fof= flexural stress due to D PL(L+I) at the midthickness of the other flange at the point of splice concurrent with f, in the flange under consideration (positive for tension; negative for compression) = 10.18.2.3.5 + f, and fofshall be computed using the gross section of the member. The maximum resultant force on the eccentrically loaded connection shall not exceed the slip resistance computed from Equation (10-172) with Nb taken equal to 1.O. (10 - 4t) where: F, = design shear stress in the web at the point of splice defined in Article 10.18.2.3.6 D = web depth t, = web thickness 10.18.2.3.8 As a minimum, in cases of the service load design method, web splice plates and their connections shall be proportioned for a design moment at the point of splice, M,, representing the portion of the flexural moment that is assumed to be resisted by the web. M, shall be applied at the mid-depth of the web. For sections where the neutral axis is not located at the mid-depth of the web, a horizontal design force resultant in the web at the point of splice, H,, shall also be applied at the middepth of the web. M, and H, may be computed as follows: 10.18.2.3.8 DIVISION I-DESIGN where: FCf= design stress at the point of splice for the controlling flange defined in Article 10.18.2.2.3 (positive for tension; negative for compression) R = reduction factor for hybrid girders specified in Article 10.40.2.1. R shall be taken equal to 1.0 when Fcfis less than or equal to the allowable flexural stress for the web steel. For homogeneous girders, R shall always be taken equal to 1.o. Rf = the absolute value of the ratio of Fcfto fcffor the controlling flange fnCf= flexural stress at the mid-thickness of the noncontrolling flange at the point of splice concurrent with fcf(positive for tension; negative for compression) 10.18.2.3.9 As a minimum, in the case of the service load design method, web splice plates and their connections shall be proportioned to develop the most critical combination of FwDtw,M,, M, and H,.

The connections shall be proportioned as eccentrically loaded connections to develop the resultant design force through shear in the bolts and bearing at the bolt holes, as specified in Table 10.32.3B. In addition, as a minimum, high-strength bolted connections for web splices shall be proportioned as eccentrically loaded connections to prevent slip under the most critical combination of shear, moment, and horizontal force due to D + (L + I) at the point of splice. The portion of the flexural moment that is assumed to be resisted by the web and the horizontal force resultant shall be computed using the gross section of the member. The maximum resultant force on the eccentrically loaded connection shall not exceed the slip resistance computed from Article 10.32.3.2.1 with Nb taken to equal 1.O. 10.18.3 Compression Members Compression members such as columns and chords shall have ends in close contact at riveted and bolted splices. Splices of such members which will be fabricated and erected with close inspection and detailed with milled ends in full contact bearing at the splices may be held in place by means of splice plates and rivets or high-strength bolts proportioned for not less than 50% of the lower allowable design strength of the sections spliced. The strength of compression members connected by highstrength bolts or rivets shall be determined using the gross section. 277 10.18.4 Tension Members 10.18.4.1. As a minimum, splices in tension members shall be proportioned for a design force, P,, equal to the allowable design strength specified in Article 10.18.1.1 times the effective area of the member, A,, defined as follows: where: A, = net section of the member computed as specified in Article 10.16.14 = p 0.0 for AASHTO M 270 Grade 100/100W (ASTMA709 Grade 100/100W)steels, or when holes exceed 1 % inch in diameter = 0.15 for all other steels and when holes are less than or equal to 1%inch in diameter. A, = gross area of the member The diameter of the holes shall be taken as specified in Article 10.16.14.6. As a minimum, the connection shall be proportioned to develop the design force through shear in the bolts and bearing at the bolt holes, as specified in Article 10.56.1.3.2 in the case of the strength design method and in Table 10.32.3B in the case of the service load design method. 10.18.4.2 As a minimum, in the case of the strength design method, high-strength bolted connections for splices in tension members shall be proportioned to prevent slip at an overload design force, Po,equal to the maximum tensile stress in the member due to D + p, (L + I) times the gross area of the member, where p, is defined in Article 3.22. The slip resistance of the connection shall be computed from Equation (10-172). In the case of the service load design method, high-strength bolted connections shall be proportioned to prevent slip at a force equal to the maximum tensile stress in the member due to D + (L + I) times the gross area of the member. The slip resistance of the connection shall be determined as specified in Article 10.32.3.2.1. 10.18.5 Welded Splices 10.18.5.1 Tension and compression members may be spliced by means of full penetration butt welds, preferably without the use of splice plates. 10.18.5.2 Welded field splices preferably should be arranged to minimize overhead welding.

278 HIGHWAY BRIDGES 10.18.5.3 Material of different widths spliced by butt welds shall have transitions conforming to Figure 10.18.5A. The type transition selected shall be consistent with the Fatigue Stress Category from Table 10.3.1B for the Groove Welded Connection used in the design of the member. At butt-welded splices joining pieces of different thicknesses, there shall be a uniform slope between the offset surfaces, including the weld, of not more than 1 in 2%. 10.18.5.3 design shall be designed for not less than the average of the required strength at the point of connection and the strength of the member at the same point, but, in any event, not less than 75% of the strength of the member.

10.19.1.2 Connections shall be made symmetrical about the axis of the members insofar as practicable. Connections, except for lacing bars and handrails, shall contain not less than two fasteners or equivalent weld. 10.19 STRENGTH OF CONNECTIONS 10.19.1.3 Members, including bracing, preferably shall be so connected that their gravity axes will intersect in a point. Eccentric connections shall be avoided, if practicable, but if unavoidable the members shall be so proportioned that the combined fiber stresses will not exceed the allowed axial design stress. 10.19.1 General 10.19.1.1 Except as otherwise provided herein, connections for main members shall be designed in the case of service load design for a capacity based on not less than the average of the calculated design stress in the member at the point of connection and the allowable stress of the member at the same point, but, in any event, not less than 75% of the allowable stress in the member. Connections for main members in the case of load factor 10.19.1.4 In the case of connections which transfer total member shear at the end of the member, the gross section shall be taken as the gross section of the connected elements. s"! f Q 1 7 2 2" Butt JointA 0 DETAIL OF WIDTH TRANSITION L Butt Joint Narrower Plate I Narrower Plate L ~ i d t of h Wider Plate (b) Straight Tapered Transition 4 (a) 2'-0" Radius Transition FIGURE 10.18.5A Splice Details d . 10.19.2 DIVISION I-DESIGN 10.19.2 End Connections of Floor Beams and Stringers 10.19.2.1 The end connection shall be designed for the calculated member loads. The end connection angles of floor beams and stringers shall be not less than 3/s inch in finished thickness. Except in cases of special end floor beam details, each end connection for floor beams and stringers shall be made with two angles. The length of these angles shall be as great as the flanges will permit. Bracket or shelf angles which may be used to furnish support during erection shall not be considered in determining the number of fasteners required to transmit end shear. 10.19.2.2 End-connection details shall be designed with special care to provide clearance for making the field connection. 279 be at least YZand preferably % the girder depth. Cross frames shall be as deep as practicable. Intermediate cross frames shall preferably be of the cross type or vee type. End cross frames or diaphragms shall be proportioned to adequately transmit all the lateral forces to the bearings. Intermediate cross frames shall be normal to the main members when the supports are skewed more than 20". Cross frames on horizontally curved steel girder bridges shall be designed as main members with adequate provisions for transfer of lateral forces from the girder flanges. Cross frames and diaphragms shall be designed for horizontal wind forces as described in Article 10.21.2. 10.20.2 Stresses Due to Wind Loading When Top Flanges Are Continuously Supported 10.20.2.1 Flanges 10.19.2.3 End connections of stringers and floor beams preferably shall be bolted with high-strength bolts; however, they may be riveted or welded. In the case of welded end connections, they shall be designed for the vertical loads and the end-bending moment resulting from the deflection of the members. 10.19.2.4 Where timber stringers frame into steel floor beams, shelf angles with stiffeners shall be provided to carry the total reaction. Shelf angles shall be not less than %a inch thick. 10.19.3 End Connections of Diaphragms and Cross Frames The maximum induced stresses, I?, in the bottom flange of each girder in the system can be computed from the following: where: R = [0.2272L - 111si2I3 when no bottom lateral bracing is provided R = [0.059L- 0.641 s,"~ when bottom lateral bracing is provided 10.19.3.2 The end connections for diaphragms or cross frames in straight rolled-beam and plate-girder bridges shall be designed for the calculated member loads. 10.19.3.2 Vertical connection plates such as transverse stiffeners

symmetrical about the central plane of the truss and as nearly parallel as practicable. Bars shall be as close together as practicable and held against lateral movement, but they shall be so arranged that adjacent bars in the same panel will be separated by at least % inch. 10.29.1.1 Fixed ends shall be firmly anchored. Bearings for spans less than 50 feet need have no provision for deflection. Spans of 50 feet or greater shall be provided with a type of bearing employing a hinge, curved bearing plates, elastomeric pads, or pin arrangement for deflection purposes. 10.29.1.2 Spans of less than 50 feet may be arranged to slide upon metal plates with smooth surfaces and no provisions for deflection of the spans need be made. Spans of 50 feet and greater shall be provided with rollers, rockers, or sliding plates for expansion purposes and shall also be provided with a type of bearing employing a hinge, curved bearing plates, or pin arrangement for deflection purposes.
10.29.1.3 In lieu of the above requirements, elastomeric bearings may be used. See Section 14 of this specification. 10.29.2 Bronze or Copper-Alloy Sliding Expansion Bearings Bronze or copper-alloy sliding plates shall be chamfered at the ends. They shall be held securely in position, usually by being inset into the metal of the pedestals or sole plates. Provisions shall be made against any accumulation of dirt which will obstruct free movement of the span. 10.29.3 Rollers Expansion rollers shall be connected by substantial side bars and shall be guided by gearing or other effectual 286 HIGHWAY BRIDGES means to prevent lateral movement, skewing, and creeping. The rollers and bearing plates shall be protected from dirt and water as far as practicable, and the design shall be such that water will not be retained and that the roller nests may be inspected and clean easily. 10.29.4 Sole Plates and Masonry Plates 10.29.4.1 Sole plates and masonry plates shall have a minimum thickness of Y 4 inch. 10.29.4.2 For spans on inclined grades greater than 1% without hinged bearings, the sole plates shall be beveled so that the bottom of the sole plate is level, unless the bottom of the sole plate is radially curved. 10.29.5 Masonry Bearings Beams, girders, or trusses on masonry shall be so supported that the bottom chords or flanges will be above the bridge seat, preferably not less than 6 inches. 10.29.3 bearings, this distance shall be measured from the center of the pin. In built-up pedestals and shoes, the web plates and angles connecting them to the base plate shall be not less than % inch thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely.The minimum thickness of the metal in cast steel pedestals shall be 1 inch. Pedestals and shoes shall be so designed that the load will be distributed uniformly over the entire bearing. 10.29.7.2 Webs and pin holes in the webs shall be arranged to keep any eccentricity to a minimum. The net section through the hole shall provide 140% of the net section required for the actual stress transmitted through the pedestal or shoe.
Pins shall be of sufficient length to secure a full bearing. Pins shall be secured in position by appropriate nuts with washers. All portions of pedestals and shoes shall be held against lateral movement of the pins. 10.30 FLOOR SYSTEM 10.30.1 Stringers 10.29.6 Anchor Bolts 10.29.6.1 Trusses, girders, and rolled beam spans preferably shall be securely anchored to the substructure. Anchor bolts shall be swedged or threaded to secure a satisfactory grip upon the material used to embed them in the holes.
10.29.6.2 The following are the minimum requirements for each bearing: For rolled beam spans the outer beams shall be anchored at each end with 2 bolts, 1 inch in diameter, set 10 inches in the masonry. For trusses and girders: Spans 50 feet in length or less; 2 bolts, 1 inch in diameter, set 10 inches in the masonry. Spans 51 to 100 feet; 2 bolts, 1% inches in diameter, set 12 inches in the masonry. Stringers preferably shall be framed into floor beams. Stringers supported on the top flanges of floor beams preferably shall be continuous over two or more panels. 10.30.2 Floor Beams Floor beams preferably shall be at right angles to the trusses or main girders and shall be rigidly connected thereto. Floor beam connections preferably shall be located so the lateral bracing system will engage both the floor beam and the main supporting member. In pin-connected trusses, if the floor beams are located below the bottom chord pins, the vertical posts shall be extended sufficiently below the pins to make a rigid connection to the floor beam. 10.30.3 Cross Frames Spans 101 to 150 feet; 2 bolts, 1% inches in diameter, set 15 inches in the masonry. In bridges with wooden floors and steel stringers, intermediate cross frames (or diaphragms) shall be placed between stringers more than 20 feet long. Spans greater than 150 feet; 4 bolts, 1%inches in diameter, set 15 inches in the masonry. 10.30.4 Expansion Joints 10.29.7 Pedestals and Shoes 10.30.4.1 To provide for expansion and contraction movement, floor expansion joints shall be provided at all expansion ends of spans and at other points where they may be necessary. 10.29.7.1 Pedestals and shoes preferably shall be made of cast steel or structural steel. The difference in width between the top and bottom bearing surfaces shall not exceed twice the distance between them. For hinged 10.30.4.2 Apron plates, when used, shall be designed to bridge the joint and to prevent, so far as practicable, the accumulation of roadway debris upon the bridge seats.
Preferably, they shall be connected rigidly to the end floor beam. 10.29.6.3 Anchor bolts shall be designed to resist uplift as specified in Article 3.17. 10.30.5 DIVISION I-DESIGN - 287 - 10.30.5 End Floor Beams 10.30.8 Stay-in-Place Deck Forms There shall be end floor beams in all square-ended trusses and girder spans and preferably in skew spans. End floor beams for truss spans preferably shall be designed to permit the use of jacks for lifting the superstructure. For this case, the allowable stresses may be increased 50%. 10.30.6 End Panel of Skewed Bridges 10.30.8.1 Concrete Deck Panels When precast prestressed deck panels are used as permanent forms spanning between beams, stringers, or girders, the requirementsof Article 9.12, Deck Panels, and Article 9.23, Deck Panels, shall be met. 10.30.8.2 Metal Stay-in-Place Forms In skew bridges without end floor beams, the end panel stringers shall be secured in correct position by end struts connected to the stringers and to the main truss or girder. The end panel lateral bracing shall be attached to the main trusses or girders and also to the end struts. Adequate provisions shall be made for the expansion movement of stringers. 10.30.7 Sidewalk Brackets Sidewalk brackets shall be connected in such a way that the bending stresses will be transferred directly to the floor beams. When metal stay-in-place forms are used as permanent forms spanning between beams, stringers, or girders, the forms shall be designed to support, as a minimum, the weight of the concrete (including that in the corrugations, if applicable), a construction load of 50 psf, and the weight of the form.
The forms shall be designed to be elastic under construction loads. The elastic deformation caused by the dead load of the forms, plastic concrete and reinforcement shall not exceed a deflection of greater than L1180 or % inch for form work spans (L) of 10 feet or less, or a deflection of L1240 or % inch for form work spans (L) over 10 feet. Part C SERVICE LOAD DESIGN METHOD ALLOWABLE STRESS DESIGN 10.31 SCOPE Allowable stress design is a method for proportioning structural members using design loads and forces, allowable stresses, and design limitations for the appropriate material under service conditions. See Part D-Strength Design Method-Load Factor Design for an alternate design procedure. 10.32 ALLOWABLE STRESSES 10.32.1 Steel Allowable stresses for steel shall be as specified in Table 10.32.1A. 10.32.2 Weld Metal Unless otherwise specified, the yield point and ultimate strength of weld metal shall be equal to or greater than minirnun specified value of the base metal.
Allowable stresses on the effective areas of weld metal shall be as follows: Butt Welds: The same as the base metal joined, except in the case of joining metals of different yields when the lower yield material shall govern.
Fillet Welds: Fv = 0.27 F, (10-12) where, Fv = allowable basic shear stress; F, = tensile strength of the electrode classification When detailing fillet welds for quenched and tempered steels-the designer may use electrode classifications with strengths less than the base metal provided that this requirement is clearly specified on the plans. Plug Welds: Fv= 12,400 psi for resistance to shear stresses only, where, Fv= allowable basic shear stress. HIGHWAY BRIDGES 288 10.32.2 TABLE 10.32.1A Allowable Stresses- Structural Steel (In pounds per square inch) Structural Carbon Steel 5F+3 High-Strength Low-Alloy Steel Quenched and Tempered Low-Alloy Steel High-Yield Strength Quenched and Tempered Alloy Steela AASHTO ~esignation~" M 270 Grade 36 M 270 Grade 50 M 270 M 270 M 270 Grade SOW Grade HPS7OW & Grades 100/100W Grade 70W Equivalent ASTM DesignationC A 709 Grade 36 A 709 Grade 50 A 709 A 709 A 709 Grade SOW Grade HPS70W& Grades 100/100W Grade 70W Thickness of Plates Up to 4 in. incl. Up to 4 in. incl. Shapes All Groups All Groups Axial tension in members with no 0.55Fy holes 0.46Fu Axial tension in members with holes Gross and tension in extreme fiber of Section rolled shapes, girders, and built-up 0.55Fy sections with holes subject to bending. Satisfy both Gross and Net Section criterion. Net Section 0.46Fu Axial compression, gross section: stiffeners of plate girders.
Compression in splice material, gross section Compression in extreme fibers of rolled shapes, girders, and built-up sections subject to bending.
Gross section, when compression flange is: (A) Supported laterally its full length 0.55Fy byembedment in concrete (B) Partially supported or is unsupporteddve 20,000 Up to 4 in. incl.
Upto 4 in. incl. Upto Over2Xin. 2 X in.
incl. to 4 in.
incl. All Groups Not Applicable Not Applicable 27,000 38,000 Not Applicable 27,000 50,600 Not Applicable 46,000 20,000 27,000 27,000 38,000 26,700 29,900 32,200 41,400 50,600 46,000 20,000 27,000 27,000 38,000 55,000 49,000 20,000 27,000 27,000 38,000 55,000 49,000 Not Applicable Cb = 1.75 + 1.05 (M1/M2) + 0.3 (MI/M$ 5 2.3 where MI is the smaller andM2 the larger end moment in the unbraced segment of the beam; MlM2 is positive when the moments cause reverse curvature and negative when bent in single curvature. Cb = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments. Compression in concentrically loaded columns with C, = (2 . r r 2 ~ ~ ~ y=) 1 1 2 when KLIr ES. 5 C, f 126.1 107.0 107.0 90.4 75.7 79.8 10.32.2 289 DIVISION I-DESIGN TABLE 10321A Allowable Stresses- Structural Steel (Jn pounds per square inch) (Continued) Structural Carbon Steel .me High-Strength Low-Alloy Steel Quenched and Tempered Low-Alloy Steel High-Yield Strength Quenched and Tempered a Alloy Steel when KLlr > C, with ES. = 2.12 Shear in girder webs, gross section F. = 0.33Fy Bearing on milled stiffeners and other 0.80Fy steel parts in contact (rivets and bolts excluded) g Stress in extreme fiber of pins 0.80Fy F, = 0.40Fy Shear in pins Bearing on pins not subject to rotationh 0.80Fy Bearing o n pins subject to rotation (such as used in rocken and hinges) 0.40Fy Bearing on connected material at Low Carbon Steel Bolts (ASTM A 307). lbmed Bolts,Ribbed Bolts, and Rivets (ASTM A 502 Grades 1 and 2)Governed bv Table 10.32.3A Quenched and tempend alloy steel structural shapes and seamless mechanical t u b i i meeting all mechanical and chemical requirements of A 709 Grades 1W100W except that the s p c c i f i maximum tensile strength may be 140.000 psi for sln~cturalshapes and 145,000 psi for seamless mpanical tubing, shall be wnsidacd as A 709 Grades 1W100W s t d . Except for the mandatory mtch toughness and weldability requkements, the ASRvI designations are similar to the AASHTO designations. Steels meeting the AASIITO quiremenu are p r q d i f ~for i use in welded bridges M270G~36andA709Gr.36arcwuivalenttoM 183andA36 ~270GrMaad~709~~50are&~to~223GrMand~572~r.50 M 270 Gc SOW and A 709 Gr 50W are eauivalent to M 222 and A 588 M 270 GL 70W a d A 709 GL70W are e&ivalent to A 852 M 270 GL 1W100W and A 709 Gr.lWlOOW arc equivalent to M 244 and A 514 For theuse of larger Cbvalues, see Structural Stability Raearch Council Gvidc to Stubiliiy Design Criteriufw Metal Smhrns. 3rd Ed., p. 135. I€cover plates arc used, the allowable static stress at the point oftheoretical cutoff shall be as determined by the formula. 'C = length in inches, of unsupported flange between lateral connections, knee braces, or other points of support. = moment of inertiaof compression flange about the vertical axis in the plane of the web i n 4 I, d = depth of girder, ia J = w) ~Dc 1 where b and t repsent the flange width and thickness of the compression and tension flange, respectively (in-3 + 3 S, = section modulus with respect to compression flange Cm.3 = modulus of elasticity of steel f E = governing radius of gyration r L = actual unbraced length K = effective length factor (see Appendix C.) ES. = factor of safety = 2.12 For graphic representationof these formulas, see Appendix C. The formulas do not apply to members with variable moment of inertia.
F'rccedures for designing members with variable moments of inertia can be found in the following references: "Engiaeering Journal," American Institute of Steel Construction, January 1969, Volume 6. No. 1, and October 1972, Volume 9, No. 4; and "Steel Structures," by Wdliam McGuire. 1968, Rentice-Hall, Inc., Englewood Cliffs.
New Jersey. For members with eccentric loading, see Article 10.36. Singly symmetric and unsymmehic compression members, such as angles or tees, and doubly symmehic compression members, such as cruciform or built-up members with very thin walls, may also require consideration of flexural-torsional and torsional buckline. Refer to the Manual of Steel Combucrion. Ninth Edition, 1989, American Institute of Steel C~ll~truction. g see-also Article 10.32.4. This shall apply to pins used primarily in axially loaded members, such as truss members and cable adjusting links. It shall not apply to pins used in members having rotation caused by expansion or deflection. 290 HIGHWAY BRIDGES TABLE 10323A Allowable Stresses for Low-Carbon Steel Bolts and Power Driven Rivets (in psi) 'Qpe of Fastener Tensiona (A) Low-Carbon Steel 18,000 BoltsC 'nulled Bolts (ASTM A 307) Ribbed Bolts (B)Power-Driven Rivets (rivets driven by pneumatically or electrically operated hammers are coosidered power driven) Structural Steel Rivet Grade 1 (ASTM A 502 Grade 1) Structural Steel Rivet (high-strength) Grade 2 (ASTM A 502 Grade 2) Shear Bearing-'Qpe 13earingb Connectiona 20,000 1 1,000* 40.000 13,500 40,000 20,000 10.32.3.1.4 In bearing-type connections, pull-out shear in a plate should be investigated between the end of the plate and the end row of fasteners. (See Table 10.32.3B, footnote g.) 10.32.3.1.5 All bolts except high-strength bolts, tensioned to the requirements of Division11. Table 11.5A or Table 11.5B, shall have single self-locking nuts or double nuts. 10.32.3.1.6 Joints, utilizing high-strength bolts, required to resist shear between their connected parts are designated as either slip-critical (See Article 10.24.1.4) or bearing-type connections. Shear connections subjected to stress reversal, or where slippage would be undesirable, shall be slip-critical connections. Potential slip TABLE 10323B Allowable Stresses on High-Strength Bolts o r Connected Material (ksi) 'Applies to fastener cross-sectional a m based u p nominal body diameter. bAppliesto nominal diameter of fastener multiplied by the thickness of the metal. 'ASTM A 307 bolts shall not be used in connections subject to fatigue h m ~ o transmitting m axial force whose length beween extmne fastenersmeasured parallel to thc line of force excctdr50inches,the tabulated value shall k reduced 20 percent. 10.32.3 Fasteners (Rivets abd Bolts) Allowable stresses for fasteners shall be as listed in Tables 10.32.3.A and 10.32.3.B, and the allowable force on a slip-critical connection shall be as provided by Article 10.32.3.2.1. 10.32.3.1 General 10.32.3.1.1 In proportioning fasteners for shear or tension, the cross-sectional area based upon the nominal diameter shall be used except as otherwise noted.
10.32.3.1.2 The effective bearing area of a fastener shall be its diameter multiplied by the thickness of the metal on which it bears. In metal less than 31s inch thick, countersunk fasteners shall not be assumed to carry stress. In metal 3/8 inch thick and over, one-half of the depth of the countersink shall be omitted in calculating the bearing area. 10.32.3.1.3 In determining whether the bolt threads are excluded from the shear planes of the contact surfaces, thread length of bolts shall be calculated as two thread pitches greater than the specified thread length as an allowance for thread runout. 10.32.3 Load Condition Applied Static TensionbL AASHTO M 164 AASm M 253 (ASM A 325)' (ASTM A 490)' 3gd 47 lgd 24 Shear. F . on bolt with threads in shear planed Bearing, F,,, on connected material m standard. a.e*, ShOrt-sloned holes loaded m any direction, or O.SIP. long-sloned holes parallel to the applied bearing force Bearing. F,, on connected material in long-dotted holes perpendicular to the applied bearing fonr !k!&d d = F.LU 5 o.~F.U .AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490) high-strength bolb arc waikble in three types, designated as t y p 1 , 2 , or 3. l j p c 3 shaU be required on the plans when k g ~ n r n t cA d A S M~no ~ r a d eMW (ASTUA 709 ~ r a d SOW e). bBoltsmust be tensioned to requirements of Table 11.5A. Div 11. =See Aside 10.32.3.4 for bolts subject to tensile fatigue. dThetensilestrength of M 164 (A 325) bolts dtertases for diameters greater than 1 inch. The design values listed are for bolts up to 1 inch diameter. The design values shall be multiplied by 0.875 for diameters greater than 1 inch.
'"' g adal force whose length between ex'In connections r treme fastenersmeasured parallel to the Linc of force exQedsSOinches.
tabulated values shall bc reduced 20 percent.
For flange splices, the 50-inch length is to be measlned between the exIreme b o b on only one side of the connection. If material thickness or joint details preclude threads m the shear plane, multiply tabulated values by 1.25. IF. = @ed minimum tensile strength of c o ~ e ~ t material. ed h&&om using high-strength boltsin sloned hols with the load mlied in adirectionotherthan approximatelynormal (betwax 80 and fbb degrees) to the ads of thihole and connections with bolts in wersized holes shall bc designed for resistance agaiust slip in accordance with Artide 10.32.3.2.1. & is equal to the dear distance between the holes or between the hole and the edge of the material in the direction of the applied bearing force, in. and d is the nominal diameter of the bolt, in. flkallowable bearing force for the connection is equal to the sum of the allowable bearing f o m for the individual bolts in the wnnection. ' 10.32.3.1.6 DIVISION I-DESIGN Where of joints should be investigated at intermediate load stages especially those joints located in composite regions. F, = nominal slip resistance per unit of bolt area from Table 10.32.3C, ksi. Ab= area corresponding to the nominal body area of the bolt sq in. Nb= number of bolts in the joint. N, = number of slip planes. 10.32.3.1.7 The percentage of unit stress increase shown in Article 3.22, Combination of Loads, shall apply to allowable stresses in bolted slip-critical connections using high-strength bolts, except that in no case shall the percentage of allowable stress exceed 133%, and the requirements of Article 10.32.3.3 shall not be exceeded. Class A, B, or C surface conditions of the bolted parts as defined- in Table 10.32.3C shall be used in joints designated as slip-critical except as permitted in Article 10.32.3.2.2. 10.32.3.1.8 Bolted bearing-type connections shall be limited to members in compression and secondary members. 10.32.3.2 The allowable stress in shear, bearing and tension for AASHTO M 164 (ASTM A 325) andAASHT0 M 253 (ASTM A 490) bolts shall be as listed in Table 10.32.3B. 10.32.3.2.1 In addition to the allowable stress requirements of Article 10.32.3.2 the force on a slip-critical connection as defined in Article 10.24.1.4 shall not exceed the allowable slip force (P,) of the connection according to TABLE 10.32.3C 291 10.32.3.2.2 Subject to the approval of the Engineer, coatings providing a slip coefficient less than 0.33 may be used provided the mean slip coefficient is established by test in accordance with the requirements of Article 10.32.3.2.3, and the slip resistance per unit area are established. The slip resistance per unit area shall be taken as equal to the slip resistance per unit area from Table 10.32.3C for Class A coatings as appropriate for the hole type and bolt type times the slip coefficient determined by test divided by 0.33. 10.32.3.2.3 Paint, used on the faying surfaces of connections specified to be slip-critical, shall be qualified by test in accordance with "Test Method to Determine the Nominal Slip Resistance for Slip-Critical Connections (Slip Resistance per Unit of Bolt Area, F,, ksi) Hole and Direction of Load Application Any Direction Standard Oversized and Short Slot Transverse Parallel Long Slots Long Slots AASHTO AASHTO AASHTO AASHTO AASHTO AASHTO M 164 M 253 M 164 M 253 M 164 M 253 (ASTM (ASTM (ASTM (ASTM (ASTM (ASTM Contact Surface of Bolted Parts A 325)' A 490) A 325)' A 490) A 325)' A 490) Claw A (Slip Coefficient 0.33) Clean mill scale and blastcleaned surfaces with Class A coatingsb Claw B (Slip Coefficient 0.50) Blast-cleaned surfaces and blast-cleaned surfaces with Class B coatingsb Claw C (Slip Coefficient 0.33) Hot-dip galvanized surfaces and roughened by wire brushing after galvanizing AASHTO AASHTO M 164 M 253 (ASTM A 325)' (ASTM A 490) 15 19 13 16 11 13 9 11 23 29 19 24 16 20 14 17 15 19 13 16 11 13 9 11 "The tensile strength of M 164 (A325)bolts decreases for diameters greater than 1 inch. The design values listed are for bolts up to 1 inch diameter. The design values shall be multiplied by 0.875 for diameters greater than 1 inch. bCoatings classified as Class A or Class B include those coatings which provide a mean slip coefficient not less than 0.33 or 0.50, respectively,as determined by Testing Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints. See Article 10.32.3.2.3. 292 HIGHWAY BRIDGES Slip Coefficient for Coatings Used in Bolted Joints" as adopted by the Research Council on Structural Connections. See Appendix A of Allowable Stress Design Specification for Structural Joints Using ASTM A 325 or A 490 Bolts published by the Research Council on Structural Connections. 10.32.3.3 Applied Tension, Combined Tension, and Shear 10.32.3.3.1 High-strength bolts preferably shall be used for fasteners subject to tension or combined tension and shear. 10.32.3.3.2 Bolts required to support applied load by means of direct tension shall be so proportioned that their average tensile stress computed on the basis of nominal bolt area will not exceed the appropriate stress in Table 10.32.3B. The applied load shall be the sum of the external load and any tension resulting from prying action. The tension due to the prying action shall be 10.32.3.2.3 = 105 ksi for M 164 (A 325) bolts over 1-inch diameter; = 150 ksi for M 253 (A 490) bolts. 10.32.3.3.4 Where rivets or high-strength bolts are subject to both shear and tension, the tensile stress shall not exceed the value obtained from the following equations: for fvFv5 0.33 Fi = F, (10-16) for fvFv> 0.33 where fv = computed rivet or bolt shear stress in shear, ksi; Fv = allowable shear stress on rivet or bolt from Table 10.32.3A or Table 10.32.3B, ksi; F, = allowable tensile stress on rivet or bolt from Table 10.32.3A or Table 10.32.3B, ksi; F: = reduced allowable tensile stress on rivet or bolt due to the shear stress, ksi. Note: Equation (10-18) has been removed.
where Q = the prying tension per bolt (taken as zero when negative); T = the direct tension per bolt due to external load; a = distance from center of bolt to edge of plate in inches; b = distance from center of bolt under consideration to toe of fillet of connected part in inches; t = thickness of thinnest part connected in inches.
10.32.3.3.3 For combined shear and tension in slip-critical joints using high-strength bolts where applied forces reduce the total clamping force on the friction plane, the slip resistance per unit area of bolt, fv,shall not exceed the value obtained from the following equation: where: f, = computed tensile stress in the bolt due to applied loads including any stress due to prying action, ksi; F, = nominal slip resistance per unit of bolt area from Table 10.32.3C, ksi; F,= 120 ksi for M 164 (A 325) bolts up to 1-inch diameter; 10.32.3.4 Fatigue When subject to tensile fatigue loading, the tensile stress in the bolt due to the service load plus the prying force resulting from application of service load shall not exceed the following design stresses in kips per square inch. The nominal diameter of the bolt shall be used in calculating the bolt stress. The prying force shall not exceed 60% of the externally applied load. Number of Cycles Not more than 20,000 From 20,000 to 500,000 More than 500,000 AASHTO M 164 (ASTM A 325) AASHTO M 253 (ASTM A 490) 38 35.5 27.5 47 44.0 34.0 10.32.4 Pins, Rollers, and Expansion Rockers 10.32.4.1 The effective bearing area of a pin shall be its diameter multiplied by the thickness of the material on 10.32.4.1 DIVISION I-DESIGN 293 TABLE 10.32.4.38 Allowable Stresses-Steel Bars and Steel Forgings AASHTO Designation with Size Limitations - M 169 4 in. in M 102 To 20 M 102 To 20 M 102 To 10 M 102 To 20 dia. or less in. in dia. in. in dia. in. in dia. in. in dia. ASTM Designation Grade or Class - A 108 Grades 1016 1030 incl. Minimum Yield Point, psi F~ Bearing on Pins .notSubject to Rotation, psiC Bearing on Pins Subject to Rotation, psi (such as used in rockers and hinges) A 668 Class C A 668 Class D A 668 Class F A 668" Class G 36,OoOb 33,000 37,500 50,000 50,000 0.80Fy 29,000b 26,000 30,000 40,000 40,000 0.40Fy 14,OoOb 13,000 15,000 20,000 20,000 Stress in Extreme Fiber, psi Shear, psi - - =Maysubstitute rolled material of the same properties. bFordesign purposes only. Not a part of the A 108 specifications. Supplementary material requirements should provide guarantee that material will meet these values. 'This shall apply to pins used primarily in axially loaded members, such as truss members and cable adjusting links. It shall not apply to pins used in members having rotation caused by expansion or deflection. which it bears. When parts in contact have different yield points, Fyshall be the smaller value. 10.32.4.2 Design stresses for Steel Bars, Carbon Cold Finished Standard Quality, AASHTO M 169 (ASTM A 108), and Steel Forgings, Carbon and Alloy, for General Industrial Use, AASHTO M 102 (ASTM A 668), are given in Table 10.32.4.3A.
10.32.5 Cast Steel, Ductile Iron Castings, Malleable Castings, and Cast Iron 10.32.5.1 Cast Steel and Ductile Iron 10.32.5.1.1 For cast steel conforming to specifications for Steel Castings for Highway Bridges, AASHTO M 192 (ASTMA 486), Mild-to-~ediimStrength Carbon-Steel Castings for General Application, AASHTO M 103 (ASTM A 27), and Corrosion-Resistant Iron-Chromium, Iron-Chromium-Nickel and NickelBased Alloy Castings for General Application, AASHTO M 163 (ASTM A 743), and for Ductile Iron Castings (ASTM A 536), the allowable stresses in pounds Per square inch shall be in accordance with 10.32.5.1A. 10.32.5.1.2 When in contact with castings - or steel of a different yield point, the allowable unit bearing stress of the material with the lower yield point shall govern. For riveted or bolted connections,Article 10.32.3 shall govern. 10.32.5.2 Malleable Castings Malleable castings shall conform to specifications for Malleable Iron Castings, ASTM A 47 Grade 35018. The following allowable stresses in pounds per square inch shall be used: Tension ..... 18,000 Bending in Extreme Fiber ..... 18,000 Modulus of Elasticity ..... 25,000,000 10.32.5.3 Cast Iron Cast iron castings shall conform to specifications for Gray Iron Castings, AASHTO M 105 (ASTM A 48), Class 30B. The following allowable stresses in pounds per sauare inch shall be used: Bending in Extreme Fiber . . . . . .
. . . . . 3,000 Shear ..... 3,000 Direct Compression, Short Columns ..... 12,000 10.32.5.4 Bronze or Copper-Alloy 10.32.5.4.1 Bronze castings, AASHTO M 107 (ASTM B 22), Copper Alloys 913 or 911, or CopperAlloy Plates, AASHTO M 108 (ASTM B loo), shall be specified. 10.32.5.4.2 The allowable unit-bearing stress in pounds per square inch on bronze castings or copper-alloy plates shall be 2,000. 294 HIGHWAY BRIDGES 10.32.6 TABLE 10.32.5.1A Allowable Stresses-Cast Steel and Ductile Iron AASHTO Designation ASTM Designation Class or Grade Minimum Yield Point, F, Axial Tension Tension in Extreme Fibers Axial Compression, Short Columns Compression in Extreme Fibers Shear Bearing, Steel Parts in Contact Bearing on Pins not Subject to Rotation Bearing on Pins Subject to Rotation (such as used in rockers and hinges) M 163 A743 CA-15 65,000 24,000 24,000 32,000 32,000 14,000 48,000 43,000 21,500 10.32.6 Bearing on Masonry 10.34 PLATE GIRDERS 10.32.6.1 The allowable unit-bearing stress in pounds per square inch on the following types of masonry shall be: 10.34.1 General Granite . . . . . .
. . . . . 800 Sandstone and Limestone . . . .
. . . . . .
400 10.32.6.2 The above bridge seat unit stress will apply only where the edge of the bridge seat projects at least 3 inches (average) beyond the edge of shoe or plate. Otherwise, the unit stresses permitted will be 75% of the above amounts. None A536 60-40-18 40,000 16,000 16,000 22,000 22,000 10,000 33,000 28,000 14,000 10.34.1.1 Girders shall be proportioned by the moment of inertia method. For members primarily in bending, the entire gross section shall be used when calculating tensile and compressive stresses. Holes for high-strength bolts or rivets andlor open holes not exceeding 1% inches, may be neglected provided the area removed from each flange does not exceed 15% of that flange. That area in excess of 15%shall be deducted from the gross area. 10.33 ROLLED BEAMS 10.34.1.2 The compression flanges of plate girders supporting timber floors shall not be considered to be laterally supported by the flooring unless the floor and fastenings are specially designed to provide support. 10.33.1 General 10.34.2 Flanges 10.32.6.3 For allowable unit-bearing stress on concrete masonry, refer to Article 8.15.2.1.3. 10.33.1.1 Rolled beams, including those with welded cover plates, shall be designed by the moment of inertia method. Rolled beams with riveted cover plates shall be designed on the same basis as riveted plate girders. 10.33.1.2 The compression flanges of rolled beams supporting timber floors shall not be considered to be laterally supported by the flooring unless the floor and fastenings are specially designed to provide adequate support. 10.33.2 Bearing Stiffeners Suitable stiffeners shall be provided to stiffen the webs of rolled beams at bearings when the unit shear in the web adjacent to the bearing exceeds 75% of the allowable shear for girder webs. See the related provisions of Article 10.34.6. 10.34.2.1 Welded Girders 10.34.2.1.1 Each flange may comprise a series of plates joined end to end by full penetration butt welds. Changes in flange areas may be accomplished by varying the thickness andlor width of the flange plate, or by adding cover plates. Where plates of varying thicknesses or widths are connected, the splice shall be made in accordance with Article 10.18 and welds ground smooth before attaching to the web. The compression-flange width, b, on fabricated I-shaped girders preferably shall not be less than 0.2 times the web depth, but in no case shall it be less than 0.15 times the web depth. If the area of the compression flange is less than the area of the tension flange, the minimum flange width may be based on two times the depth of the web in compression rather than the web depth.
The compression-flange thickness, t, preferably shall not be less than 1.5 times the web thickness.
The 10.34.2.1 DIVISION I-DESIGN width-to-thickness ratio, blt, of flanges subject to tension shall not exceed 24. 10.34.2.1.2 When cover plates are used, they shall be designed in accordance with Article 10.13. 10.34.2.1.3 The ratio of compression flange plate width to thickness shall not exceed the value determined by the formula -=- 3y250 but in no case shall & (10 -19) blt exceed 24 10.34.2.1.4 Where the calculated compressive bending stress equals .55 Fy the (bit) ratios for the various grades of steel shall not exceed the following: 36,000 psi, Y.P. Min. blt = 23 50,000 psi, Y.P. Min. b/t = 20 70,000 psi, Y.P. Min. blt = 17 90,000 psi, Y.P. Min. blt = 15 100,000 psi, Y.P. Min. blt = 14 295 10.34.2.2.3 Where the calculated compressive bending stress equals 0.55 Fy, the b'lt ratios for the various grades of steel shall not exceed the following: 36,000 psi, Y.P. Min. b'lt 50,000 psi, Y.P. Min.
b'lt 70,000 psi, Y.P. Min. b'/t 90,000 psi, Y.P. Min. b'lt 100,000 psi, Y.P. Min. b'lt = 11 = 10 = 8.5 = 7.5 = 7 10.34.2.2.4 In the case of a composite girder the width of outstanding legs of top flange angles in compression, except those reinforced by plates, shall not exceed the value determincd by the following formula b' 1,930 -=t a but in no case shall b'lt exceed 12 - 22) In the above b' is the width of a flange angle, t is the thickness, fb is the calculated maximum compressive stress, and fda is the top flange compressive stress due to noncomposite dead load. In the above b is the flange plate width, t is the thickness, and fb is the calculated maximum compressive bending stress. (See Article 10.40.3 for Hybrid Girders.) 10.34.2.2.5 The gross area of the compression flange, except for composite design, shall be not less than the gross area of the tension flange. 10.34.2.1.5 In the case of a composite girder the ratio of the top compression flange plate width to thickness shall not exceed the value determined by the formula 10.34.2.2.6 Flange plates shall be of equal thickness, or shall decrease in thickness from the flange angles outward.
No plate shall have a thickness greater than that of the flange angles. (10 - 20) 10.34.2.2.7 At least one cover plate of the top flange shall extend the full length of the girder except when the flange is covered with concrete. Any cover plate that is not full length shall extend beyond the theoretical cutoff point far enough to develop the capacity of the plate or shall extend to a section where the stress in the remainder of the girder flange is equal to the allowable fatigue stress, whichever is greater. The theoretical cutoff point of the cover plate is the section at which the stress in the flange without that cover plate equals the allowable stress, exclusive of fatigue considerations. b 37 860 t = a but in no case shall blt exceed 24 where fdais the top flange compressive stress due to noncomposite dead load. 10.34.2.2 Riveted or Bolted Girders 10.34.2.2.1 Flange angles shall form as large a part of the area of the flange as practicable. Side plates shall not be used except where flange angles exceeding 7/s inch in thickness otherwise would be required. 10.34.2.2.2 Width of outstanding legs of flange angles in compression, except those reinforced by plates, shall not exceed the value determined by the formula b' - 1,625 t butinnocaseshall b'lt exceed 12 10.34.2.2.8 The number of fasteners connecting the flange angles to the web plate shall be sufficient to develop the increment of flange stress transmitted to the flange angles, combined with any load that is applied directly to the flange. 10.34.2.2.9 Legs of angles 6 inches or greater in width, connected to web plates, shall have two lines of HIGHWAY BRIDGES 296 10.34.2.2.9 fasteners. Cover plates over 14 inches wide shall have four lines of fasteners. 10.34.3 Thickness of Web Plates 10.34.3.1 Girders Not Stiffened Longitudinally 10.34.3.1.1 The web plate thickness of plate girders without longitudinal stiffeners shall not be less than that determined by the formula t, =- DA (See Figure 10.34.3.1A.) (10-23) 23,000 but in no case shall the thickness be less than Dl170. 10.34.3.1.2 Where the calculated compressive bending stress in the flange equals the allowable bending stress, the thickness of the web plate (with the web stiffened or not stiffened, depending on the requirements for transverse stiffeners) shall not be less than (where the Y.P. is for the flange material) 36,000 psi, Y.P. Min. Dl165 50,000 psi, Y.P. Min. Dl140 70,000 psi, Y.P. Min. Dl115 90,000 psi, Y.P. Min. Dl105 100,000 psi, Y.P. Min. Dl100 10.34.3.2 Girders Stiffened Longitudinally 10.34.3.2.1 The web plate thickness of plate girders equipped with longitudinal stiffeners shall not be less than that determined by the formula d for S20.4 Dc k=5.17 but in no case shall the thickness be less than Dl340. For symmetrical girders see Figure 10.34.3.1 .A. In the above, D (depth of the web) is the clear unsupported distance in inches between the flange components, t, is the web thickness, k is the buckling coefficient, d, is the distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or the leg of the fb (ksi) WEB THICKNESS AND GIRDER DEPTH ,(a function of bending stress) D = depth of web tw = thickness of web fb = calculated compressive bending stress in flange FIGURE 10.343.1A Web Thickness vs. Girder Depth for Noncomposite Symmetrical Sections compression flange component, D, is the depth of the web in compression calculated by summing the stresses from the applicable stages of loading, and fbis the calculated flange bending stress in the compression flange. The depth of web in compression, D,, in composite sections subjected to negative bending may be taken as the depth of the web in compression of the composite section without summing the stresses from the various stages of loading. When both edges of the web are in compression, k shall be taken equal to 7.2. 10.34.3.2.2 Where the calculated bending stress in the flange equals the allowable bending stress, the thickness of the web plate in a symmetrical girder stiffened with transverse stiffeners in combination with one longitudinal stiffener located a distance Dl5 from the compression flange shall not be less than (where the Y.P. is for the flange material) 36,000 psi, Y.P. Min. Dl327 50,000 psi, Y.P. Min. Dl278 70,000 psi, Y.P. Min. Dl235 10.34.3.2.2 DIVISION I-DESIGN 90,000 psi, Y.P. Min. Dl207 100,000 psi, Y.P. Min. Dl196 297 for In the above, D (depth of web) is the clear unsupported distance in inches between flange components. 10.34.4 Transverse Intermediate Stiffeners 10.34.4.1 Transverse intermediate stiffeners may be omitted if the average calculated unit-shearing stress in the gross section of the web plate at the point considered, f,, is less than the value given by the following equation: where D = unsupported depth of web plate between flanges in inches; t, = thiclmess of the web plate in inches; F, = allowable shear stress in psi. 10.34.4.2 Where transverse intermediate stiffeners are required, the spacing of the transverse intermediate stiffener shall be such that the actual shearing stress will not exceed the value given by the following equation; the maximum spacing is further limited to 3D and is subject to the handling requirement below: The constant C is equal to the buckling shear stress divided by the shear yield

10.38.5.1.1 Fatigue The range of horizontal shear shall be computed by the formula where S, range of horizontal shear, in kips per inch, at the junction of the slab and girder at the point in the span under consideration; V, = range of shear due to live loads and impact in kips; at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads); Q = statical moment about the neutral axis of the composite section of the transformed concrete area, in3. Between points of dead-load contraflexure, the statical moment about the neutral axis of the composite section of the area of reinforcement embedded in the concrete may be used unless the transformed concrete area is considered to be fully effective for negative moment in computing the longitudinal range of stress; I = moment of inertia of the transformed composite section, in4. Between points of dead-load contraflexure, the moment of inertia of the steel girder including the area of reinforcement embedded in the concrete may be used unless the transformed concrete area is considered to be fully effective for negative moment in computing the longitudinal range of stress. = (In the formula, the concrete area is transformed into an equivalent area of steel by dividing the effective concrete flange width by the modular ratio, n.) The allowable range of horizontal shear, Z, in pounds on an individual connector is as follows: Channels Welded studs (for Wd 2 4) where length of a channel shear connector, in inches, measured in a transverse direction on the flange of a girder; d = diameter of stud in inches; a = 13,000 for 100,000 cycles 10,600 for 500,000 cycles 7,850 for 2,000,000 cycles 5,500 for over 2,000,000 cycles; w = 306 HIGHWAY BRIDGES 10.38.5.1.1 The number of connectors, N2, required between the points of maximum positive moment and points of adjacent maximum negative moment shall equal or exceed the number given by the formula B = 4,000 for 100,000 cycles 3,000 for 500,000 cycles 2,400 for 2,000,000 cycles 2,100 for over 2,000,000 cycles; H = height of stud in inches.

The required pitch of shear connectors is determined by dividing the allowable range of horizontal shear of all connectors at one transverse girder cross-section (2%)by the horizontal range of shear S,, but not to exceed the maximum pitch specified in Article 10.38.5.1. Over the interior supports of continuous beams the pitch may be modified to avoid placing the connectors at locations of high stresses in the tension flange provided that the total number of connectors remains unchanged. 10.38.5.1.2 Ultimate Strength The number of connectors so provided for fatigue shall be checked to ensure that adequate connectors are provided for ultimate strength. The number of shear connectors required shall equal or exceed the number given by the formula At points of maximum negative moment the force in the slab is taken as where A: = total area of longitudinal reinforcing steel at the interior support within the effective flange width; *:?I = specified minimum yield point of the reinforcing steel. The ultimate strength of the shear connector is given as follows: Channels Welded studs (for Wd > 4) where N, = number of connectors between points of maximum positive moment and adjacent end supports; S, = ultimate strength of the shear connector as given below; = reduction factor = 0.85; P = force in the slab as defined hereafter as PI or p2. + At points of maximum positive moment, the force in the slab is taken as the smaller value of the formulas PI = A,Fy (10-62) P2 = 0.85flbG (10-63) or where A, = total area of the steel section including coverplates; Fy = specified minimum yield point of the steel being used; f: = compressive strength of concrete at age of 28 days; b = effective flange width given in Article 10.38.3; t, = thickness of the concrete slab. where E, = modulus of elasticity of the concrete in pounds per square inch; S, = ultimate strength of individual shear connector in pounds; A,, = cross-sectional area of a stud shear connector in square inches; h = average flange thickness of the channel flange in inches; t = thickness of the web of a channel in inches; W = length of a channel shear connector in inches; f: = compressive strength of the concrete in 28 days in pounds per square inch; d = diameter of stud in inches; w = unit weight of concrete in pounds per cubic foot. *When reinforcement steel embedded in the top slab is not used in computing section properties for negative moments, P3is equal to zero. 10.38.5.1.3 10.38.5.1.3 DIVISION I-DESIGN Additional Connectors to Develop Slab Stresses The number of additional connectors required at points of contraflexure when reinforcing steel embedded in the concrete is not used in computing section properties for negative moments shall be computed by the formula where Nc = number of additional connectors for each beam at point of contraflexure; As = total area of longitudinal slab reinforcing steel for each beam over interior support; = range of stress due to live load plus impact in f, the slab reinforcement over the support (in lieu of more accurate computations, f, may be taken as equal to 10,000 psi); Z, = allowable range of horizontal shear on an individual shear connector. The additional connectors, N,, shall be placed adjacent to the point of dead load contraflexure within a distance equal to one-third the effective slab width, i.e., placed either side of this point or centered about it. It is preferable to locate field splices so that they clear the connectors. 307 by two or more single cell composite box girders. The distance center-to-center of flanges of each box should be the same and the average distance center-to-center of flanges of adjacent boxes shall be not greater than 1.2 times and not less than 0.8 times the distance center-to-center of flanges of each box.

In addition to the above, when nonparallel girders are used, the distance center-to-center of adjacent flanges at supports shall be not greater than 1.35 times and not less than 0.65 times the distance center-tocenter of flanges of each box. The cantilever overhang of the deck slab, including curbs and parapets, shall be limited to 60% of the average distance center-to-center of flanges of adjacent boxes, but shall in no case exceed 6 feet. 10.39.1.2 The provisions of Division I, Design, shall govern where applicable, except as specifically modified by Articles 10.39.1 through 10.39.8. 10.39.2 Lateral Distribution of Loads for Bending Moment 10.39.2.1 The live load bending moment for each box girder shall be determined by applying to the girder, the fraction WLof a wheel load (both front and rear), determined by the following equation: 10.38.5.2 Vertical Shear The intensity of unit-shearing stress in a composite girder may be determined on the basis that the web of the steel girder carries the total external shear, neglecting the effects of the steel flanges and of the concrete slab. The shear may be assumed to be uniformly distributed throughout the gross area of the web. 10.38.6 Deflection 10.38.6.1 The provisions of Article 10.6 in regard to deflections from live load plus impact also shall be applicable to composite girders. 10.38.6.2 When the girders are not provided with falsework or other effective intermediatesupport during the placing of the concrete slab, the deflection due to the weight of the slab and other permanent dead loads added before the concrete has attained 75% of its required 28-day strength shall be computed on the basis of noncomposite action. 10.39 COMPOSITE BOX GIRDERS 10.39.1 General 10.39.1.1 This section pertains to the design of simple and continuous bridges of moderate length supported where R= Nw Number of Box Girders (10-71) N, = WJ12 reduced to the nearest whole number; Wc= roadway width between curbs in feet, or barriers if curbs are not used. R shall not be less than 0.5 or greater than 1.5. 10.39.2.2 The provision of Article 3.12, Reduction of Load Intensity, shall not apply in the design of box girders when using the design load WL given by the above equation. 10.39.3 Design of Web Plates 10.39.3.1 Vertical Shear The design shear V, for a web shall be calculated using the following equation: 308 HIGHWAY BRIDGES where V, = vertical shear; 0 = angle of inclination of the web plate to the vertical. 10.39.3.1 10.39.4.2.2 For greater blt ratios, but not exceeding 13,300/&, the stress in an unstiffened bottom flange shall not exceed the value determined by the use of the formula 10.39.3.2 Secondary Bending Stresses 10.39.3.2.1 Web plates may be plumb (90" to bottom of flange) or inclined. If the inclination of the web plates to a plane normal to the bottom flange is no greater than 1 to 4, and the width of the bottom flange is no greater than 20% of the span, then the transverse bending stresses resulting from distortion of the span, and the transverse bending stresses resulting from distortion of the girder cross section and from vibrations of the bottom plate need not be considered. For structures in this category transverse bending stresses due to supplementary loadings, such as utilities, shall not exceed 5,000 psi. 10.39.3.2.2 For structures exceeding these limits, a detailed evaluation of the transverse bending stresses due to all causes shall be made. These stresses shall be limited to a maximum stress or range of stress of 20,000 psi. 10.39.4 Design of Bottom Flange Plates 10.39.4.1 Tension Flanges 10.39.4.1.1 In cases of simply supported spans, the bottom flange shall be considered completely effective in resisting bending if its width does not exceed one-fifth the span length.

If the flange plate width exceeds one-fifth of the span, an amount equal to one-fifth of the span only shall be considered effective. 10.39.4.1.2 For continuous spans, the criteria above shall be applied to the lengths between points of contraflexure. 10.39.4.2 Compression Flanges Unstiffened Unstiffened compression flanges designed for the basic allowable stress of 0.55 Fyshall have a width to thickness ratio equal to or less than the value obtained by the use of the formula 10.39.4.2.1 10.39.4.2.3 For values of blt exceeding 13,3001*, the stress in the flange shall not exceed the value given by the formula 10.39.4.2.4 The blt ratio preferably should not exceed 60 except in areas of low stress near points of dead load contraflexure. 10.39.4.2.5 Should the blt ratio exceed 45, longitudinal stiffeners should be considered. 10.39.4.2.6 Unstiffened compression flanges shall also satisfy the provisions of Article 10.39.4.1. The effective flange plate width shall be used to calculate the flange bending stress. The full flange plate width shall be used to calculate the allowable bending stress. 10.39.4.3 Compression Flanges Stiffened Longitudinally* 10.39.4.3.1 Longitudinal stiffeners shall be at equal spacings across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener is at least equal to where + = 0.07 k n for values of n greater than 1; 3 4 4 tf w n k 0.125 k3 for avalue of n = 1; = thickness of the flange; = width of flange between longitudinal stiffeners or distance from a web to the nearest longitudinal stiffener; = number of longitudinal stiffeners; = buckling coefficient which shall not exceed 4. = where b = flange width between webs in inches; t = flange thickness in inches. *In solving these equations a value of k between 2 and 4 generally should be assumed. 10.39.4.3.1 DIVISION I-DESIGN p fiY FIGURE 10.39.4.3A. Longitudinal Stiffeners-Box Girder Compression Flange 309 310 HIGHWAY BRIDGES 10.39.4.3.1 10.39.4.3.2 DMSION I-DESIGN 10.39.4.3.2 For the flange, including stiffeners, to be designed for the basic allowable stress of 0.55 Fy,the ratio wlt shall not exceed the value given by the formula 10.39.4.3.3 For greater values of wlt but not exceeding 60 or (6,650 a)/*, whichever is less, the stress in the flange, including stiffeners, shall not exceed the value determined by the formula 311 10.39.4.4.2 The transverse stiffeners shall be proportioned so that the moment of inertia of each stiffener about an axis through the centroid of the section and parallel to its bottom edge is at least equal to where Af = area of bottom flange including longitudinal stiffeners; a = spacing of transverse stiffeners; f, = maximum longitudinal bending stress in the flange of the panels on either side of the transverse stiffener; E = modulus of elasticity of steel. 10.39.4.3.4 For values of wlt exceeding (6,650 a) / but not exceeding 60, the stress in the flange, including stiffeners, shall not exceed the value given by the formula 10.39.4.3.5 When longitudinal stiffeners are used, it is preferable to have at least one transverse stiffener placed near the point of dead load contraflexure. The stiffener should have a size equal to that of a longitudinal stiffener. 10.39.4.3.6 Ifthe longitudinal stiffeners are placed at their maximum wlt ratio to be designed for the basic allowable design stresses of 0.55 Fyand the number of longitudinal stiffeners exceeds 2, then transverse stiffeners should be considered. 10.39.4.3.7 Compression flanges stiffened longitudinally shall also satisfy the provisions of Article 10.39.4.1. The effective flange plate width shall be used to calculate the flange bending stress. The full flange plate width shall be used to calculate the allowable bending stress. 10.39.4.4 Compression Flanges Stiffened Longitudinally and Transversely 10.39.4.4.1 The longitudinal stiffeners shall be at equal spacings across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener is at least equal to 10.39.4.4.3 For the flange, including stiffeners, to be designed for the basic allowable stress of 0.55 Fy,the ratio wlt for the longitudinal stiffeners shall not exceed the value given by the formula where 10.39.4.4.4 For greater values of wlt, but not exceeding 60 or (6,650 a) / * , whichever is less, the stress in the flange, including stiffeners, shall not exceed the value determined by the formula [6,650 fi- 1 - sin (2 X 3380% "1 (10-84) 10.39.4.4.5 For values of wlt exceeding (6,650 %)I* but not exceeding 60, the stress in the flange, including stiffeners, shall not exceed the value given by the formula HIGHWAY BRIDGES 312 10.39.4.4.5 10.39.4.5.2 Longitudinal stiffeners shall be extended to locations where the maximum stress in the flange does not exceed that allowed for base metal adjacent to or connected by fillet welds.

10.39.4.4.6 The maximum value of the buckling coefficient, k,, shall be 4. When k, has its maximum value, the transverse stiffeners shall have a spacing, a, equal to or less than 4w. If the ratio a/b exceeds 3, transverse stiffeners are not necessary. 10.39.4.4.7 The transverse stiffeners need not be connected to the flange plate but shall be connected to the webs of the box and to each longitudinal stiffener. The connection to the web shall be designed to resist the vertical force determined by the formula 10.39.5 Design of Flange to Web Welds The total effective thickness of the web-flange welds shall be not less than the thickness of the web, except, when two or more interior intermediate diaphragms per span are provided, the minimum size fillet welds specified in Article 10.23.2.2 may be used. Regardless of the type weld used, welds shall be deposited on both sides of the connecting flange or web plate. 10.39.6 Diaphragms where S, = section modulus of the transverse stiffener. 10.39.4.4.8 The connection to each longitudinal stiffener shall be designed to resist the vertical force determined by the formula 10.39.6.1 Diaphragms, cross-frames, or other means shall be provided within the box girders at each support to resist transverse rotation, displacement, and distortion. 10.39.6.2 Intermediate diaphragms or cross-frames are not required for steel box girder bridges designed in accordance with this specification. 10.39.7 Lateral Bracing 10.39.4.5 Compression Flange Stiffeners, General Generally, no lateral bracing system is required between box girders. A horizontal wind load of 50 pounds per square foot shall be applied to the area of the superstructure exposed in elevation. Half of the resulting force shall be applied in the plane of the bottom flange. The section assumed to resist the horizontal load shall consist of the bottom flange acting as a web and 12 times the thickness of the webs acting as flanges. A lateral bracing system shall be provided if the combined stresses due to the specified horizontal force and dead load of steel and deck exceed 150% of the allowable design stress. 10.39.4.5.1 The width to thickness ratio of any out- 10.39.8 Access and Drainage 10.39.4.4.9 Compression flanges stiffened longitudinally and transversely shall also satisfy the provisions of Article 10.39.4.1. The effective flange plate width shall be used to calculate the flange bending stress. The full flange plate width shall be used to calculate the allowable bending stress. standing element of the flange stiffeners shall not exceed the value determined by the formula 10.40 HYBRID GIRDERS where b' t' F, Consistent with climate, location, and materials, consideration shall be given to the providing of manholes, or other openings, either in the deck slab or in the steel box for form removal, inspection, maintenance, drainage, etc. = width of any outstanding stiffener element = thickness of outstanding stiffener element = yield strength of outstanding stiffener ele- ment. 10.40.1 General 10.40.1.1 This section pertains to the design of girders that utilize a lower strength steel in the web 10.40.1.1 DIVISION I-DESIGN 313 10.40.1.2 The provisions of Division I, Design, shall govern where applicable, except as specifically modified by Articles 10.40.1 through 10.40.4. 10.40.2 Allowable Stresses 10.40.2.1 Bending 10.40.2.1.1 The bending stress in the web may exceed the allowable stress for the web steel provided that the stress in each flange does not exceed the allowable stress from Articles 10.3 or 10.32 for the steel in that flange multiplied by the reduction factor, R. (See Figure 10.40.2.1A and 10.40.2.1B.) RATIO OF WEB AREA TO TENSION FLANGE AREA# FIGURE 10.40.2.1A where: a = minimum specified yield strength of the web divided by the minimum specified yield strength of the tension flange;* fi = area of the web divided by the area of the tension flange;" $ = distance from the outer edge of the transformed section for composite girders) divided by the depth of the steel section. 10.40.2.1.2 The bending stress in the concrete slab in composite girders shall not exceed the allowable stress for the concrete multiplied by R. 10.40.2.1.3 R shall be taken as 1.0 at sections where the bending stress in both flanges does not exceed the allowable stress for the web. RATIO OF WEB AREA TO TENSION FLANGE AREA,B FIGURE 10.40.2.1B than in one or both of the flanges. It applies to composite and noncomposite plate girders, and composite box girders. At any cross section where the bending stress in either flange exceeds 55% of the minimum specified yield strength of the web steel, the compression-flange area shall not be less than the tension-flange area. The top-flange area shall include the transformed area of any portion of the slab or reinforcing steel that is considered to act compositely with the steel girder. 10.40.2.1.4 Longitudinal web stiffeners preferably shall not be located in yielded portions of the web. 10.40.2.2 Shear The design of the web for a hybrid girder shall be in compliance with Article 10.34.3 except that Equation (10-26) of Article 10.34.4.2 for the allowable average shear stress in the web of transversely stiffened nonhybrid girders shall be replaced by the following equation for the allowable average shear stress in the web of transversely stiffened hybrid girders: *Bottom flange of orthotropic deck bridges. 314 HIGHWAY BRIDGES 10.40.2.2 10.41.1.3 For an alternate design method (Strength Design), see Article 10.60. where F, is equal to the specified minimum yield strength of the web. The provisions of Article 10.34.4.4, and the equation for A in Article 10.34.4.7 are not applicable to hybrid girders. 10.40.2.3 Fatigue Hybrid girders shall be designed for the allowable fatigue stress range given in Article 10.3 and Table 10.3.1A.

10.40.3 Plate Thickness Requirements In calculating the maximum width-to-thickness ratio of the flange plate according to Article 10.34.2, fbshall be taken as the lesser of the calculated bending stress in the compression flange divided by the reduction factor, R, or the allowable bending stress for the compression flange. 10.40.4 Bearing Stiffener Requirements In designing bearing stiffeners at interior supports of continuous hybrid girders for which a is less than 0.7, no part of the web shall be assumed to act in bearing. 10.41 ORTHOTROPIC-DECK SUPERSTRUCTURES 10.41.1 General 10.41.1.1 This section pertains to the design of steel bridges that utilize a stiffened steel plate as a deck. Usually the deck plate is stiffened by longitudinal ribs and transverse beams; effective widths of deck plate act as the top flanges of these ribs and beams. Usually the deck including longitudinal ribs, acts as the top flange of the main box or plate girders. As used in Articles 10.41.1 through 10.41.4.10, the terms rib and beam refer to sections that include an effective width of deck plate. 10.41.1.2 The provisions of Division I, Design, shall govern where applicable, except as specifically modified by Articles 10.41.1 through 10.41.4.10. An appropriate method of elastic analysis, such as the equivalent-orthotropic-slabmethod or the equivalent-grid method, shall be used in designing the deck. The equivalent stiffness properties shall be selected to correctly simulate the actual deck. An appropriate method of elastic analysis, such as the thin-walled-beam method, that accounts for the effects of torsional distortions of the crosssectional shape shall be used in designing the girders of orthotropic-deck box-girder bridges. The box-girder design shall be checked for lane or truck loading arrangements that produce maximum distortional (torsional) effects. 10.41.2 Wheel Load Contact Area The wheel loads specified in Article 3.7 shall be uniformly distributed to the deck plate over the rectangular area defined below: Width Length Wheel Load Perpendicular in Direction (kip) to Traffic (inches) of Traffic (inches) 8 12 16 20 20 24 + 2t + 2t + 2t 8 + 2t 8 + 2t 8 2t + In the above table, t is the thickness of the wearing surface in inches. 10.41.3 Effective Width of Deck Plate 10.41.3.1 Ribs and Beams The effective width of deck plate acting as the top flange of a longitudinal rib or a transverse beam may be calculated by accepted approximate methods.* 10.41.3.2 Girders 10.41.3.2.1 The full width of deck plate may be considered effective in acting as the top flange of the girders if the effective span of the girders is not less than: (1) 5 times the maximum distance between girder webs and (2) 10 times the maximum distance from edge of the deck to the nearest girder web. The effective span shall be taken as the actual span for simple spans and the distance between points of contraflexure for continuous spans. Alternatively, the effective width may be determined by accepted analytical methods. 10.41.3.2.2 The effective width of the bottom flange of a box girder shall be determined according to the provisions of Article 10.39.4.1.

10.41.4 Allowable Stresses 10.41.4.1 Local Bending Stresses in Deck Plate The term local bending stresses refers to the stresses caused in the deck plate as it carries a wheel load to the ribs and beams. The local transverse bending stresses caused in the deck plate by the specified wheel load plus 30% impact shall not exceed 30,000 psi unless a higher allowable stress is justified by a detailed fatigue analysis or *"Design Manual for Orthotropic Steel Plate Deck Bridges," AISC, 1963, or "Orthotropic Bridges, Theory and Design," by M.S. Troitsky, Lincoln Arc Welding Foundation, 1967. 10.41.4.1 DNISION I-DESIGN p p l p conhguratlons by a in which the spacing of transverse beams is at least 3 times the spacing of longitudinal-rib webs, the local longitudinal and transverse bending stresses in the deck plate need not be combined with the other bending stresses covered in Articles 10.41.4.2 and 10.41.4.3. 10.41.4.2 Bending Stresses in Longitudinal Ribs The total bending stresses in longitudinal ribs due to a combination of (1) bending of the rib and (2) bending of the girders may exceed the allowable bending stresses in Article 10.32 by 25%. The bending stress due to each of the two individual modes shall not exceed the allowable bending stresses in Article 10.32. 10.41.4.3 Bending Stresses in Transverse.Beams The bending stresses in transverse beams shall not exceed the allowable bending stresses in Article 10.32. 10.41.4.4 Intersections of Ribs, Beams, and Girders Connections between ribs and the webs of beams, holes in the webs of beams to permit passage of ribs, connections of beams to the webs of girders, and rib splices may affect the fatigue life of the bridge when they occur in regions of tensile stress. Where applicable, the number of cycles of maximum stress and the allowable fatigue stresses given in Article 10.3 shall be applied in designing these details; elsewhere, a rational fatigue analysis shall be made in designing the details. Connections between webs of longitudinal ribs and the deck plate shall be designed to sustain the transverse bending fatigue stresses caused in the webs by wheel loads.

10.41.4.5 Thickness of Plate Elements 10.41.4.5.1 Longitudinal Ribs and Deck Plate Plate elements comprising longitudinal ribs, and deck-plate elements between webs of these ribs, shall meet the minimum thickness requirements of Article 10.35.2. The quantity fa may be taken as 75% of the sum of the compressive stresses due to (1) bending of the rib and (2) bending of the girder, but not less than the compressive stress due to either of these two individual bending modes. 315 19;41;4.6 Maximum Slenderness or" Longitudinal Ribs The slenderness, L/r, of a longitudinal rib shall not exceed the value given by the following formula unless it can be shown by a detailed analysis that overall buckling of the deck will not occur as a result of compressive stress induced by bending of the girders where L = distance between transverse beams; r = radius of gyration about the horizontal centroidal axis of the rib including an effective width of deck plate; F = maximum compressive stress in psi in the deck plate as a result of the deck acting as the top flange of the girders; this stress shall be taken as positive; F, = yield strength of rib material in psi. 10.41.4.7 Diaphragms Diaphragms, cross frames, or other means shall be provided at each support to transmit lateral forces to the bearings and to resist transverse rotation, displacement, and distortion. Intermediate diaphragms or cross frames shall be provided at locations consistent with the analysis of the girders. The stiffness and strength of the intermediate and support diaphragms or cross frames shall be consistent with the analysis of the girders. 10.41.4.8 Stiffness Requirements 10.41.4.8.1 Dejections The deflections of ribs, beams, and girders due to live load plus impact may exceed the limitations in Article 10.6 but preferably shall not exceed 1/500 of their span. The calculation of the deflections shall be consistent with the analysis used to calculate the stresses. To prevent excessive deterioration of the wearing surface, the deflection of the deck plate due to the specified wheel load plus 30% impact preferably shall be less than 1/3m of the distance between webs of ribs. The stiffening effect of the wearing surface shall not be included in calculating the deflection of the deck plate. 10.41.4.5.2 Girders and Transverse Beams Plate elements of box girders, plate girders, and transverse beams shall meet the requirements of Articles 10.34.2 to 10.34.6 and 10.39.4. 10.41.4.8.2 Vibrations The vibrational characteristics of the bridge shall be considered in arriving at a proper design. 316 HIGHWAY BRIDGES 10.41.4.9 Wearing Surface A suitable wearing surface shall be adequately bonded to the top of the deck plate to provide a smooth, nonskid riding surface and to protect the top of the plate against corrosion and abrasion. The wearing surface material shall provide (1) sufficient ductility to accommodate, without cracking or debonding, expansion and contraction imposed by the deck plate, (2) sufficient fatigue strength to withstand flexural cracking due to deck-plate deflections, (3) sufficient durability to resist rutting, shoving, and wearing, (4) imperviousness to water and motor- 10.41.4.9 vehicle fuels and oils, and (5) resistance to deterioration from deicing salts, oils, gasolines, diesel fuels, and kerosenes. 10.41.4.10 Closed Ribs Closed ribs without access holes for inspection, cleaning, and painting are permitted. Such ribs shall be sealed against the entrance of moisture by continuously welding (1) the rib webs to the deck plate, (2) splices in the ribs, and (3) diaphragms, or transverse beam webs, to the ends of the ribs. Part D STRENGTH DESIGN METHOD LOAD FACTOR DESIGN 10.42 SCOPE Load factor design is a method of proportioning structural members for multiples of the design loads. To ensure serviceability and durability, consideration is given to the control of permanent deformations under overloads, to the fatigue characteristics under service loadings, and to the control of live load deflections under service loadings. See Part C--Service Load Design Method-Allowable Stress Design for an alternate design procedure. their computed maximum strengths shall be at least equal to the total effects of design loads multiplied by their respective load factors specified in Article 3.22. 10.44.3 Service behavior shall be investigated as specified in Articles 10.57 through 10.59. 10.45 ASSUMPTIONS 10.45.1 Strain in flexural members shall be assumed directly proportional to the distance from the neutral axis. 10.43 LOADS 10.43.1 Service live loads are vehicles which may operate on a highway legally without special load permit. 10.43.2 For design purposes, the service loads are taken as the dead, live, and impact loadings described in Section 3. - 10.43.3 Overloads are the live loads that can be allowed on a structure on infrequent occasions without causing permanent damage. For design purposes, the maximum overload is taken as 5(L + I)/3. 10.43.4 The maximum loads are the loadings specified in Article 10.47. 10.44 DESIGN THEORY 10.44.1 The moments, shears, and other forces shall be determined by assuming elastic behavior of the structure except as modified in Article 10.48.1.3. 10.44.2 The members shall be proportioned by the methods specified in Articles 10.48 through 10.56 so that 10.45.2 Stress in steel below the yield strength, F,, of the grade of steel used shall be taken as 29,000,000 psi times the steel strain. For strain greater than that corresponding to the yield strength, F,, the stress shall be considered independent of strain and equal to the yield strength, F,. This assumption shall apply also to the longitudinal reinforcement in the concrete floor slab in the region of negative moment when shear connectors are provided to ensure composite action in this region.

10.45.3 At maximum strength the compressive stress in the concrete slab of a composite beam shall be assumed independent of strain and equal to 0.85fl. 10.45.4 Tensile strength of concrete shall be neglected in flexural calculations,except as permitted under the provisions of Articles 10.57.2, 10.58.1, and 10.58.2.2. 10.46 DESIGN STRESS FOR STRUCTURAL STEEL The design stress for structural steel shall be the specified minimum yield point or yield strength, F,, of the steel used as set forth in Article 10.2. 1n47 DIVISION I-DESIGN 317 10.47 MAXIMUM DESIGN LOADS where b is the flange width and t is the flange thickness. The maximum moments, shears, or forces to be sustained by a stress-carrying member shall be computed for the load combinations specified in Article 3.22. Each part of the structure shall be proportioned for the group loads that are applicable and the maximum design required by the group loading combinations shall be used. (b) Web thickness 10.48 FLEXURAL MEMBERS Flexural members are subject to the following requirements in this article in addition to any applicable requirements from Articles 10.49 through 10.61 that may supersede these requirements. The compression-flange width, b, on fabricated I-shaped girders preferably shall not be less than 0.2 times the web depth, but in no case shall it be less than 0.15 times the web depth.

If the area of the compression flange is less than the area of the tension flange, the minimum flange width may be based on two times the depth of the web in compression rather than the web depth. The compression-flange thickness, t, preferably shall not be less than 1.5 times the web thickness. The width-to-thickness ratio, bit, of flanges subject to tension shall not exceed 24. 10.48.1 Compact Sections Sections of properly braced constant-depth flexural members without longitudinal web stiffeners, without holes in the tension flange and with high resistance to local buckling qualify as compact sections. Sections of rolled or fabricated flexural members meeting the requirements of Article 10.48.1.1 below shall be considered compact sections and the maximum strength shall be computed as where D is the clear distance between the flanges and t,is the web thickness. When both b/t and D/t, exceed 75% of the above limits, the following interaction equation shall apply where Fflis the yield strength of the compression flange.

(c) Spacing of lateral bracing for compression flange where Lb is the distance between points of bracing of the compression flange, ry is the radius of gyration of the steel section with respect to the Y-Y axis, M1 is the smaller moment at the end of the unbraced length of the member, and Muis the ultimate moment from Equation (10-92) at the other end of the unbraced length: (MIA4,,) is positive when moments cause single curvature between brace points. (Ml/M,) is negative when moments cause reverse curvature between brace points. The required lateral bracing shall be provided by braces capable of preventing lateral displacement and twisting of the main members or by embedment of the top and sides of the compression flange in concrete. (d) Maximum axial compression where Fy is the specified yield point of the steel being used, and Z is the plastic section modulus.* 10.48.1.1 Compact sections shall meet the following requirements: (For certain frequently used steels these requirements are listed in Table 10.48.1.2A.) (a) Compression flange *Values for rolled sections are listed in the Manual of Steel Construction, Ninth Edition, 1989, American Institute of Steel Construction. Appendix D shows the method of computing Z as presented in the Commentary of AISI Bulletin 15. where A is the area of the cross section. Members with axial loads in excess of 0. 15FyAshould be designed as beam-columns as specified in Article 10.54.2. 10.48.1.2 Article 10.48.1 is applicable to steels with a demonstrated ability to reach M,. Steels such as AASHTO M 270 Grades 36,50, and 50W (ASTM A 709 Grades 36, 50, and 50W), and AASHTO M 270 Grade HPS70W (ASTM A 709 Grade HPS70W) meet these requirements. The limitations set forth in Article 10.48.1 are given in Table 10.48.1.2A. 10.48.1.3 In the design of a continuous beam with compact negative-moment support sections of AASHTO 318 HIGHWAY BRIDGES TABLE 10.48.1.2A Limitations for Compact Sections Fy (psi) 36,000 50,000 70,000 blt 21.7 18.4 15.5 Ldr, (MIIMu= 0*) 100 72 51 Ldry(MI/MU = 1 *) 39 28 20 10.48.1.3 fb = factored bending stress in the compression flange, but not to exceed Fy 10.48.2.1 The above equations are applicable to sections meeting the following requirements: (a) Compression flange * For values of M W ,other than 0 and 1, use Equation (10-96). (b) Web thickness M 270 Grades 36,50 and 50W (ASTM A 709 Grades 36, 50, and 50W) steel complying with the provision of Article 10.48.1.1, negative moments over such supports at Overload and Maximum Load determined by elastic analysis may be reduced by a maximum of 10%. Such reductions shall be accompanied by an increase in moments throughout adjacent spans statically equivalent and opposite in sign to the decrease of negative moments at the adjacent supports. For example, the increase in moment at the center of the span shall equal the average decrease of the moments at the two adjacent supports. The reduction shall not apply to the negative moment of a cantilever. This 10% redistribution of moment shall not apply to compact sections of AASHTO M 270 Grade HPS7OW or Grade 70W (ASTM A 709 Grade HPS70W or Grade 70W) steel. The web thickness shall meet the requirement given by Equation (10-104) or Equation (10- log), as applicable, subject to the corresponding requirements of Article 10.49.2 or 10.49.3. For unstiffened webs, the web thickness shall not be less than Dl150. (c) Spacing of lateral bracing for compression flange where d is the depth of beam or girder, and Af is the flange area.

If Equation (10-101) is not satisfied, Mu calculated from Equation (10-99) shall not exceed Mu calculated from the provisions of Article 10.48.4.1. (d) Maximum axial compression 10.48.2 Braced Noncompact Sections For sections of rolled or fabricated flexural members not meeting the requirements of Article 10.48.1.1 but meeting the requirements of Article 10.48.2.1 below, the maximum strength shall be computed as the lesser of Members with axial loads in excess of 0.15 F,A should be designed as beam-columns as specified in Article 10.54.2. 10.48.2.2 The limitations set forth in Article 10.48.2.1 above are given in Table 10.48.2.1A. 10.48.3 Transitions subject to the requirement of Article 10.48.2.1(c) where (3 FcI= 4,400- IF, b = compression flange width t = compression flange thickness Sx,= section modulus with respect to tension flange (in.3) Sx,= section modulus with respect to compression flange (in.3) % = flange-stress reduction factor determined from the provisions of Article 10.48.4.1, with fbsubstituted for the term M,/Sxcwhen Equation (10-103b) applies The maximum strength of sections with geometric properties falling between the limits of Articles 10.48.1 TABLE 10.48.2.1A Limitations for Braced Noncompact Sections Fy(psi) 36,000 50,000 70,000 bit* 23.2 19.7 16.6 - 90,000 100,000 14.7 13.9 Lbd D/t, 556 400 286 222 200 Refer to Articles 10.48.5.1,10.48.6.1,10.49.2, or 10.49.3, as applicable. For unstiffened webs, the limit is 150. * Limits shown are for Fa = Fy.Refer also to Articles 10.48.2 and 10.48.2.1(a). 10.48.3 DIVISION I-DESIGN and 10.48.2 may be computed by straight-line interpolation, except that the web thickness must always satisfy Article 10.48.1.l(b). Arc= area of compression flange (in.'); M, = lateral torsional buckling moment, or yield moment, defined below (Ib-in.); S, = section modulus with respect to compression flange (in.3. Use S, for live load for a composite section; A = 15,400 for all sections where Dc is less than or equal to D/2; = 12,500 for sections where D, is greater than D/2. 10.48.4 Partially Braced Members Members not meeting the lateral bracing requirement of Article 10.48.2.1 (c) shall be braced at discrete locations spaced at a distance, Lb, such that the maximum strength of the section under consideration satisfies the requirements of Article 10.48.4.1. Bracing shall be provided such that lateral deflection of the compression flange is restrained and the entire section is restrained against twisting. 10.48.4.1 If the lateral bracing requirement of Article 10.48.2.1(c) is not satisfied and the ratio of the moment of inertia of the compression flange to the moment of inertia of the member about the vertical axis of the web, I,,J, is within the limits of 0.1 5 I,JI, 5 0 0.9, the maximum strength for the limit state of lateral-torsional buckling shall be computed as Mu= MrRb (10-103a) Rb = 1 for longitudinally stiffened girders if the web slenderness satisfies the following requirement: 319 The moment capacity, M,, cannot exceed the yield moment, My. In addition M, cannot exceed the lateral torsional buckling moment given below: h S -or with longitudinally For sections with stiffened webs 8 r MI = 91 x 106cb For sections with a where for for d 2 < 0.4 < (Lb IL p M, = M y k = 11.64 Dc d, = the distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinalstiffener to the inner surface or the leg of the compression flangecomponent. fb= factored bending stress in the compression flange When both edges of the web are in compression, k shall be taken equal to 7.2. Otherwise, for girders with or without longitudinal stiffeners, Rbshall be calculated as (10-103b) D, = depth of the web in compression (in.). For composite beams and girders, D, shall be calculated in accordance with the provisions specified in Article 10.50(b). t, = thickness of web (in.); for L, 2 Lb > Lp for Lb>Lr (10 - 103d) Lb= unbraced length of the compression flange, in. L, = 9,500rr/fl, in. r' = radius of gyration of compression flange about the vertical axis in the plane of the web, in. I,, = moment of inertia of compression flange about the vertical axis in the plane of the web, in.4 d = depth of girder, in. 320 HIGHWAY BRIDGES J= [(bt3), + (bt3)t+ ~ 3 t ~ ~] where b and t repre- sent the flange width and thickness of the compression &d tension flange, respectively, in.4 Cb= 1.75 + 1.05 (M1/M2)+ o . ~ (M ~ / M I ~2.3)~ where M1 is the smaller and M2 the larger end moment in the unbraced segment of the beam; M,/M2is positive when the moments cause reverse curvature and negative when bent in single curvature. Cb= 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.* The compression flange shall satisfy the requirement of Article 10.48.2.1(a). The web thickness shall meet the requirement given by Equation (10-104) or Equation (10-109), as applicable, subject to the corresponding requirementsof Article 10.49.2 or 10.49.3. For unstiffened webs, the web thickness shall not be less than Dl150. 10.48.4.1 If the web slenderness Dlt, exceeds the upper limit, either the section shall be modified to comply with the limit, or a longitudinal stiffener shall be provided. 10.48.5.2 The maximum bending strength of transversely stiffened girders meeting the requirements of Article 10.48.5.1 shall be computed by Articles 10.48.1, 10.48.2, 10.48.4.1, 10.50, 10.51, or 10.53, as applicable, subject to the requirements of Article 10.48.8.2. 10.48.5.3 The shear capacity of transversely stiffened girders shall be such that where b' is the projecting width of the stiffener. The gross cross-sectional area of intermediate transverse stiffeners shall not be less than 10.48.4.2 Members with axial loads in excess of 0. 15FyAshould be designed as beam-columns as specified in Article 10.54.2. 10.48.5 Transversely Stiffened Girders 10.48.5.1 For girders not meeting the shear requirements of Article 10.48.8.1 (Equation 10-113) transverse stiffeners are required for the web. For girders with transverse stiffeners but without longitudinal stiffeners the thickness of the web shall meet the requirement: subject to the web thickness requirement of Article 10.49.2. For different grades of steel this limit is the yield strength of the stiffener; where F, B = 1.0 for stiffener pairs, 1.8 for single angles, and 2.4 for single plates; and C is computed by Article 10.48.8

the box girders at each support to resist transverse rotation, displacement, and distortion. Intermediate diaphragms or cross-frames are not required for box girder bridges designed in accordance with this specification. 10.51.7 Design of Flange to Web Welds The total effective thickness of the web-flange welds shall not be less than the thickness of the web, except, The number of shear connectors shall be determined in accordance with Article 10.38.5.1.2 and checked for fatigue in accordance with Articles 10.38.5.1.1 and 10.38.5.1.3. 10.52.3 Maximum Spacing The maximum pitch shall not exceed 24 inches except over the interior supports of continuous beams where wider spacing may be used to avoid placing connectors at locations of high stresses in the tension flange. 10.53 HYBRID GIRDERS This section pertains to the design of girders that utilize a lower strength steel in the web than in one or both of the flanges. It applies to composite and noncomposite plate girders and to composite box girders. At any cross section where the bending stress in either flange caused by the maximum design load exceeds the minimum specified yield strength of the web steel, the compression-flange area shall not be less than the tension-flange area. The topflange area shall include the transformed area of any portion of the slab or reinforcing steel that is considered to act compositely with the steel girder. The provisions of Articles 10.48 through 10.52, 10.57.!-,and 10.57.2 shall apply to hybrid beams and gird- 10.53 DIVISION I-DESIGN 329 -- ers except as modified below. In all equations of these articles, Fy shall be taken as the minimum specified yield strength of the steel of the element under consideration with the following exceptions (1) In Articles 10.48.1.l(b), 10.48.4.17 10.48.5.1, 10.48.6.1, 10.49.2, 10.49.3.2(b), and 10.50.1.1.2, use F, of the compression flange. (2) Articles 10.57.1 and 10.57.2 shall apply to the flanges, but not to the web of hybrid girders. The provision specified in Article 10.40.4 shall also apply. Longitudinal web stiffeners preferably shall not be located in yielded portions of the web. 10.53.1 Noncomposite Hybrid Sections 10.53.1.1 Compact Sections The equation of Article 10.48.1 for the maximum strength of compact sections shall be replaced by the expression Mu = F,Z (10-145) where Fyfis the specified minimum yield strength of the flange, and Z is the plastic section modulus. In computing Z, the web thickness shall be multiplied by the ratio of the minimum specified yield strength of the web, F, to the minimum specified yield strength of the flange, F* R = l - p ~ (-' P)'(~- 1 v + PV) (10 - 148) where @ is the distance from the outer fiber of the tension flange to the neutral axis divided by the depth of the steel section. R be taken as 1.0 at sections where the stress in both flanges caused by the maximum desig loads does not exceed the specified minimum yield strength of the web.

10.53.1.3 Partially Braced Members The strength of noncompact hybrid sections of partially braced members not satisfyingthe lateral bracing requirement given by Equation (10-101) shall be calculated as the lesser of Mucalculated from Equation (10-146) or Mucalculated from Equation (10-146a). Mu calculated from Equation (10-146a) is not to exceed Mucalculated from the provisions of Article 10.48.4.1 with Equation (10-103a) replaced by the expression and the yield moment calculated as where the appropriate R is determined from Article 10.53.1.2 above, and Rb is determined by Equation (10103b). 10.53.2 Composite Hybrid Sections 10.53.1.2 Braced Noncompact Sections The equations of Article 10.48.2 for the maximum strength of braced noncompact sections shall be replaced by the expressions For symmetrical sections where The maximum strength of a compact composite section shall be computed as specified iu Article 10.50.1.1.2 or Article 10.50.2.1, as applicable, using the specified minimum yield strength of the element under consideration to compute the plastic moment capacity. The yield moment in Article 10.50.1.1.2 shall be multiplied by R (for unsymmetrical sections) from Article 10.53.1.2, with calculated as specified below for noncompact composite sections. The maximum strength of a noncompact composite section shall be taken as the maximum strength computed from Article 10.50.1.2 or Article 10.50.2.2, as applicable, times R (for unsymmetrical sections) from Article 10.53.1.2, in which $ is the distance from the outer fiber of the tension flange to the neutral axis of the transformed section divided by the depth of the steel section.

10.53.3 Shear For unsymmetrical sections Equation (10-114) of Article 10.48.8.1 for the shear capacity of transversely stiffened girders shall be replaced by the expression 330 HIGHWAY BRIDGES 10.53.3 10.54.1.2 Effective Length The provisions of Article 10.48.8.2, and the equation for A in Article 10.48.'5.3 are not applicable to hybrid girders. The effective length factor K shall be determined as follows 10.54 COMPRESSION MEMBERS (a) For members having lateral support in both directions at its ends 10.54.1 Axial Loading K 10.54.1.1 Maximum Capacity The maximum strength of concentrically loaded columns shall be computed as where A, is the gross effective area of the column cross section and F, is determined by one of the following two formulas*: = 0.75 for riveted, bolted, or welded end connec- tions; K = 0.875 for pinned ends. (b) For members having ends not fully supported laterally by diagonal bracing or an attachment to an adjacent structure, the effective length factor shall be determined by a rational procedure.** 10.54.2 Combined Axial Load and Bending 10.54.2.1 Maximum Capacity The combined maximum axial force P and the maximum bending moment M acting on a beam-column subjected to eccentric loading shall satisfy the following equations: where: where K = effective length factor in the plane of buckling; LC= length of the member between points of support in inches; r = radius of gyration in the plane of buckling in inches; F, = yield stress of the steel in pounds per square inch; E = 29,000,000 pounds per square inch; F, = buckling stress in pounds per square inch. *Singly symmetric and unsymmetric compression members, such as angles or tees, and doubly symmetric compression members, such as cruciform or built-up members with very thin walls, may also require consideration of flexural-torsional and torsional buckling, Refer to the Manual of Steel Construction, Ninth Edition, 1989, American Institute of Steel Construction. F, = buckling stress as determined by the equations of Article 10.54.1.1; Mu= maximum strength as determined by Articles 10.48.1, 10.48.2, or 10.48.4; Fe = EX' 2 ( ) = the Euler Buckling stress in the plane of bending; (10-157) = equivalent moment factor, as defined below; C = F,Z, the full plastic moment of the section; M, Z = plastic section modulus; KL = effective slenderness ratio in the plane of r bending. **B. G . Johnston, Guide to Stability Design Criteria for Metal Structures, John Wiley and Sons, Inc., New York, 1976. 10.54.2.2 DIVISION I-DESIGN 10.54.2.2 Equivalent Moment Factor C 331 The b'lt, ratio for the stiffeners shall be If the ends of the beam-column are restrained from sidesway in the plane of bending by diagonal bracing or attachment to an adjacent laterally braced structure, then the value of equivalent moment factor, C, may be computed by the formula b' -= 2,200 b' maximum - = 12 (10 - 164) ts 10.55.3 Flange Plates where a is the ratio of the numerically smaller to the larger end moment. The ratio a is positive when the two end moments act in an opposing sense (i.e., one acts clockwise and the other acts counterclockwise) and negative when they act in the s h e sense. In all cases, factor C may be taken conservatively as unity. See Article 3.2 for load factors and combinations. Use Service Load Design Method for factored loads and the formulas changed as follows: 10.55.1 Moment Amplification and Allowable Stresses 1 1.18T 1 -- m ." .

tf- b' = 27 2oo . for overhang widths, 'f ~ m a x i m u m b ' l t f = 1 2 (10-166) 10.56 SPLICES, CONNECTIONS, AND DETAILS 10.55 SOLID RIB ARCHES A, = b ' -=57 7W for width between webs (10 - 165) (10 - 159) 10.56.1 Connectors 10.56.1.1 General Connectors and connections shall be proportioned so that their design resistance, 4R, (maximum strength multiplied by a resistance factor) as given in this Article, as applicable, shall be at least equal to the effects of service loads multiplied by their respective load factors as specified in Article 3.22. AF, 10.56.1.2 Welds and Fb = Fy (10-160) 10.55.2 Web Plates No longitudinal stiffener The ultimate strength of the weld metal in groove and fillet welds shall be equal to or greater than that of the base metal, except that the designer may use electrode classifications with strengths less than the base metal when detailing fillet welds for quenched and tempered steels. However, the welding procedure and weld metal shall be selected to ensure sound welds.

The effective weld area shall be taken as defined in ANSVAASHTO/AWS D1.5 Bridge Welding Code, Article 2.3. (10 -161) 10.56.1.3 Bolts and Rivets One longitudinal stiffener 10,150 Dlt, =- a 10.56.1.3.1 In proportioning fasteners, the cross sectional area based upon nominal diameter shall be used. (10 - 162) 10.56.1.3.2 The design force, +R, in kips, for AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490) high-strength bolts subject to applied axial tension or shear is given by Two longitudinal stiffeners 13,500 Dlt, =- a (10 - 163) 4R = +FAb (10-166a) -- HIGHWAY BRIDGES - - - TABLE 10.56A Design Strength of Connectors Strength (+F) 5 p e of Fastener Groove Welda Fillet weldb l.OOFy 0.45 F,, Low-Carbon Steel Bolts ASTM A 307 Tension Shear on Bolt with Threads in Shear Planed 30 ksi 18 ksi Power-Driven Rivets ASTM A 502 Shear-Grade 1 Shear-Grade 2 25 ksi 30 ksi High-Strength Bolts AASHTO M 164 (ASTM A 325) Applied Static TensionC Shear on Bolt with Threads in Shear lane'.^.' 68 ksi AASHTO M 253 (ASTM A 490) Applied Static Tension Shear on Bolt with Threads in Shear Planedse 85 ksi 35 ksi 43 ksi "F, = yield point of connected material. bF, = minimum strength of the welding rod metal. 'The tensile smngth of M 164 (A 325) bolts decreases for diameters greater than 1 inch.

The design values listed are for bolts up to 1-inch in diameter. The design values shall be multiplied by 0.875 for diameters greater than 1 inch. dTabulated values shall be reduced by 20 percent in bearing-type connections whose length between extreme fasteners in each of the spliced parts measured parallel to the line of axial force exceeds 50 inches. For flange splices, the 50-inch length is to be measured between the extreme bolts on only one side of the connection. 'If material thickness or joint details preclude threads in the shear plane, multiply values by 1.25.

where +F = design strength per bolt area as given in Table 10.56A for appropriate kind of load, ksi; Ab = area of bolt corresponding to nominal diameter, sq in. The design bearing force, +R, on the connected material in standard, oversized, short-slotted holes loaded in any direction, or long-slotted holes parallel to the applied bearing force shall be taken as The design bearing force, +R, on the connected material in long-slotted holes perpendicular to the applied bearing force shall be taken as +R = 0.75LctF, 5 1.5dtFu (10-166c) The design bearing force for the connection is equal to the sum of the design bearing forces for the individual bolts in the connection.

In the foregoing +R Fu design bearing force, kips. specified minimum tensile strength of the connected material, ksi. LC = clear distance between the holes or between the hole and the edge of the material in the direction of the applied bearing force, in. d = nominal diameter of bolt, in. t = thickness of connected material, in. = = 10.56.1.3.3 High-strength bolts preferably shall be used for fasteners subject to tension or combined shear and tension. For combined tension and shear, bolts and rivets shall be proportioned so that the tensile stress does not exceed 10.56.1.3.3 DIVISION I-DESIGN for fJF, 1 0.33 for F,' = F, f,/F, > 0.33 F:= F~~ 333 10.56.3 Rigid Connections (10 - 167) (lo - 167a) where f, = computed rivet or bolt stress in shear, ksi; F, = design shear strength of rivet or bolt from Table 10.56A, ksi; F, = design tensile strength of rivet or bolt from Table 10.56A, ksi; F,' = reduced design tensile strength of rivet or bolt due to the applied shear stress, ksi. 10.56.1.4 Slip-Critical Joints Slip-critical joints shall be designed to prevent slip at the overload in accordance with Arhcle 10.57.3, but as a minimum the bolts shall be capable of developing the minimum strength requirements in shear and bearing of Article 10.56.1.3 under the maximum design loads. Potential slip of joints should be investigated at intermediate load stages especially those joints located in composite regions. 10.56.2 Bolts Subjected to Prying Action by Connected Parts Bolts required to support applied load by means of direct tension shall be proportioned for the sum of the external load and tension resulting from prying action produced by deformation of the connected parts. The total tension should not exceed the values given in Table 10.56A.

The tension due to prying actions shall be computed as (10 - 168) where Q = prying tension per bolt (taken as zero when negative); T = direct tension per bolt due to external load; a = distance from center of bolt to edge of plate; b = distance from center of bolt to toe of fillet of connected part; t = thickness of thinnest part connected in inches. 10.56.3.1 All rigid frame connections, the rigidity of which is essential to the continuity assumed as the basis of design, shall be capable of resisting the moments, shears, and axial loads to which they are subjected by maximum loads. 10.56.3.2 The beam web shall equal or exceed the thickness given by t w L & (~) F,dbdC (10-169) where M, = column moment; db = beam depth; d, = column depth. When the thicknessof& k l e s h n that given by the above formula, the web shall be strengthened by diagonal stiffeners or by a reinforcing plate in contact with the web over the connection area. At joints where the flanges of one member are rigidly framed into one flange of another member, the thickness of the web, t, supporting the latter flange and the thickness of the latter flange, &, shall be checked by the formulas below. Stiffeners are required on the web of the second member opposite the compression flange of the first member when A, t, < --t, +5k (10 - 170) and opposite the tension flange o the first member when f tc (10 - 171) where t, = thickness of web to be stiffened; k = distance from outer face of flange to toe of web fillet of member to be stiffened; tb = thickness of flange delivering concentrated force; & = thickness of flange of member to be stiffened; Af = area of flange delivering concentrated load. 10.57 OVERLOAD For AASHTO H or HS loadings, the overload is defined as D + 5(L+I)/3, except for beams and girders designed 334 HIGHWAY BRIDGES 10.57 -- TABLE 10.57A Design Slip Resistance for Slip-Critical Connections (Slip Resistance per Unit of Bolt Area, +Fs = +Tbp,ksi) Hole Type and Direction of Load Application Any Direction Oversize and Short Slot Standard Transverse Parallel Long Slots Long Slots AASHTO AASHTO AASHTO AASHTO AASHTO AASHTO MI64 M253 MI64 M253 MI64 M253 MI64 (ASTM (ASTM (ASTM (ASTM (ASTM (ASTM (ASTM A 490) A 325)' A 325)' A 490) A 325)' Contact Surface of Bolted Parts A 490) A 325)' Class A (Slip Coefficient 0.33) Clean mill scale and blastcleaned surfaces with Class A coatingsb Class 3 (Slip Coefficient 0.50) Blast-cleaned surfaces and blast-cleaned surfaces with Class B coatingsb Class C (Slip Coefficient 0.33) Hot-dip galvanized surfaces roughened by wire brushing after galvanizing AASHTO M 253 (ASTM A 490) 21 26 18 22 15 18 13 16 32 40 27 34 22 28 19 24 21 26 18 22 15 18 13 16 'The tensile strength of M 164 (A 325) bolts decreases for diameters greater than 1 inch.

The design values listed are for bolts up to 1inch diameter. The design values shall be multiplied by 0.875 for diameters greater than 1 inch. bCoatings classified as Class A or Class B include those coatings which provide a mean slip coefficient not less than 0.33 or 0.50, respectively, as determined by Testing Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints. See Article 10.32.3.2.3. for the Group IA load combination specified in Article 3.5.1 for which overload is defined as D + 2.2(L+I) with (L+I) assumed to occupy a single lane without concurrent loading in any other lane. For beams and girders designed for an overload vehicle selected by the operating agency in accordance with the Group IB load combination, the overload is defined as D + (LSI).

If moment redistribution is permitted under the provisions of Article 10.48.1.3, the limitations specified in Articles 10.57.1 and 10.57.2 shall apply to the modified moments, but not to the original moments. Web bend-buckling shall be checked for the overload according to Equation (10-173). For composite sections, D, shall be calculated in accordance with Article 10.50(b). Sections that do not satisfy Equation (10-173) shall be modified to comply with the requirement. 10.57.1 Noncomposite Sections At noncomposite sections, the maximum overload flange stress shall not exceed 0.8FY. 10.57.2 Composite Sections At composite sections, the maximum overload flange stress shall not exceed 0.95Fy. In computing dead load stresses, the presence or absence of temporary supports during the construction shall be considered. For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 10.50.2.3, the overload flange stresses caused by loads acting on the appropriate composite section may be computed assuming the concrete deck to be fully effective for both positive and negative moment. For this case, the resulting stresses shall be combined with the stresses due to loads acting on the noncomposite section to calculate D, for checking web bend buckling. 10.57.3 Slip-Critical Joints 10.57.3.1 In addition to the requirements of Articles 10.56.1.3.1 and 10.56.1.3.2 for fasteners, the force caused by D + 5(L + I)/3 on a slip-critical joint shall not exceed the design slip force (+R,) given by where +F, = +Tbp,design slip resistance per unit of bolt area given in Table 10.57A, ksi; Ab = area corresponding to the nominal body area of the bolt, sq in.; 10.57.3.1 DIVISION I-DESIGN 335 -- Nb N, Tb p = number of bolts in the joint; = number of slip planes; = specified tension in the bolt; = slip coefficient; = 0.33 for clean mill scale and Class A coatings - - F, = 120 ksi for M 164 (A 325) bolts up to 1-inch diameter; = 105 ksi for M 164 (A 325) bolts over 1-inch diameter; = 150 ksi for M 253 (A 490) bolts.

= 0.50 for blast-cleaned surfaces and Class B coatings; = 0.33 for hot-dip galvanized and roughened + surfaces; = 1.0 for standard holes; = 0.85 for oversized and short slotted holes; = 0.70 for long slotted holes loaded transversely; = 0.60 for long slotted holes loaded longitudinally. Class A, B, or C surface conditions of the bolted parts as defined in Table 10.57Ashall be used in joints designated as slip-critical except as permitted in Article 10.57.3.2. 10.57.3.2 Subject to the approval of the Engineer, coatings providing a slip coefficient less than 0.33 may be used provided the mean slip coefficient is established by test in accordance with the requirements of Article 10.57.3.3, and the slip resistance per unit area established. The slip resistance per unit area shall be taken as equal to the slip resistance per unit area from Table 10.57A for Class A coatings as appropriate for the hole type and bolt type times the slip coefficient determined by test divided by 0.33. 10.57.3.3 Paint, used on the faying surfaces of connections specified to be slip critical, shall be qualified by test in accordance with "Test Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints" as adopted by the Research Council on Structural Connections. See Appendix A of Allowable Stress Design SpeciJication for Structural Joints Using ASTM A 325 or A 490 Bolts, published by the Research Council on Structural Connections. 10.573.4 For combined shear and tension in slip critical joints where applied forces reduce the total clamping force on the friction plane, the design slip force shall not exceed the value +%' obtained from the following equation: 10.58 FATIGUE 10.58.1 General The analysis of the probability of fatigue of steel members or connections under service loads and the allowable range of stress for fatigue shall conform to Article 10.3, except that the limitation imposed by the basic criteria given in Article 10.3.1 shall not apply.

For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 10.50.2.3, the range of stress may be computed using the composite section assuming the concrete deck to be fully effective for both positive and negative moment. 10.58.2 Composite Construction 10.58.2.1 Slab Reinforcement When composite action is provided in the negative moment region, the range of stress in slab reinforcement shall be limited to 20,000 psi. 10.58.2.2 Shear Connectors The shear connectors shall be designed for fatigue in accordance with Article 10.38.5.1. 10.58.3 Hybrid Beams and Girders Hybrid girders shall be designed for fatigue in accordance with Article 10.3. 10.59 DEFLECTION The control of deflection of steel or of composite steel and concrete structures shall conform to the provision of Article 10.6. where f, = computed tensile stress in the bolt due to ap- plied loads including any stress due to prying action, ksi; +R,= design slip force specified in Equation (10- 172a), kips; 10.60 ORTHOTROPIC SUPERSTRUCTURES A rational analysis based on the Strength Design Method, in accordance with the specifications, will be considered as compliance with the specifications. 336 HIGHWAY BRIDGES 10.61 10.61 CONSTRUCTIBILITY The moment and shear capacity of a steel beam or girder shall meet the requirements specified below to control local buckling of the web and compression flange, and to prevent lateral torsional buckling of the cross section under the noncomposite dead load prior to hardening of the deck slab. The casting or placing sequence of the concrete deck specified in the plans shall be consideredin determining the applied moments and shears. Aload factor of y = 1.3 shall be used in calculating the applied moments and shears. 10.61.1 Web Bend Buckling The maximum factored noncomposite dead load compressive bending stress in the web shall not exceed the value given below: where $ = the distance from the centerline of a plate longi- tudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or the leg of the compression flange component. For members with or without a longitudinal stiffener, k shall be taken equal to 7.2 when both edges of the web are in compression. The web thickness requirements specified in Articles 10.48.5.1, 10.48.6.1, 10.49.2, and 10.49.3.2(b) shall not be applied to the constructibility load case. 10.61.2 Web Shear Buckling The sum of the factored noncomposite and composite dead-load shears shall not exceed the shear buckling capacity of the web specified in Article 10.48.8.1 (Equation 10-113). where F, = specified minimum yield strength of the web D, = depth of the web of the steel beam or girder in compression D = web depth t, = thickness of web k = ~ (D / D ,)for ~ members without a longitudinal stiffener a = 1.3 for members without a longitudinal stiffener a = 1.0 for members with a longitudinal stiffener Sections without longitudinal stiffeners that do not satisfy Equation (10-173) shall either be modified to comply with the requirement or a longitudinal stiffener shall be added to the web at a location on the web that satisfies both Equation (10-173) and all strength requirements, which may or may not correspond to the optimum location of the longitudinal stiffener specified in Article 10.49.3.2(a).For longitudinally stiffened girders, the buckling coefficient, k, is calculated as d for -20.4 Dc k=5.17 10.61.3 Lateral-Torsional Buckling of the Cross Section The maximum factored non-composite dead-load moment shall not exceed the value of Mu calculated for the steel beam or girder using the equations specified in Article 10.48.4.1, nor My 10.61.4 Compression Flange Local Buckling The ratio of the top compression flange width to thickness in positive-moment regions shall not exceed the value determined by the formula where fdt is the top-flange compressive stress due tp the factored noncomposite dead load divided by the factor Rb specified in Article 10.48.4.1, but not to exceed F,. Section 11 ALUMINUM DESIGN 11.1 GENERAL The purpose of this section is to provide a location for indexing aluminum design, material fabrication, and construction specifications. 11.2 BRIDGES The Specijications for Aluminum Structures, Fifth Edition, December 1986, published by the Aluminum Association, Inc., as it applies to "Bridge and Similar Type Structures," are intended to serve as a standard or guide for the preparationof plans and specifications and as a reference for designers, fabricators, and erectors of aluminum bridge and railing structures and their aluminum structural components. Welding shall conform to Section 10 of the current AWS D1.2 Structural Welding CodeAluminum, and workmanship requirements for Class I1 structures. 11.3 SOIL-METAL PLATE INTERACTION SYSTEMS The design of aluminum soil-metal plate interaction systems shall be in accordance with Section 12. Fabrication and installation shall be in accordance with Section 23Division II. 11.4 STRUCTURAL SUPPORTS FOR MGHWAY SIGNS, LUMINAIRES, AND TRAFFIC SIGNALS The AASJ3TO Standard Specijications for Structural Supports for Highway Signs, Luminaires and Trafic Signals shall be used for the design and preparation of plans and specifications, fabrication, and erection of aluminum sign supports, luminaires, and traffic signals. Welding shall conform to Section 10 of the current AWS D1.2 Structural Welding Code-Aluminum, and workmanship requirements for Class I structures. Special consideration may be given to certain support structures, which may be designed and fabricated according to the provisions of Article 11.2, Bridges. 11.5 BRIDGE RAILING The design of aluminum bridge railing shall be governed by Article 2.7; the fabrication and erection shall conform to Section 6 of the Specijications for Aluminum Structures, Fifth Edition, 1986; and the welding shall conform to Section 10 of the current AWS D1.2 Structural Welding Code-Aluminum, and workmanship requirements for Class II Structures.

The AASHTO Roadside Design Guide should be consulted for guidance on the safety considerations in the design of bridge rail. Section 12 SOIL-CORRUGATED METAL STRUCTURE INTERACTION SYSTEMS 12.1 GENERAL 12.1.1 Scope The specifications of this Section are intended for the structuraldesign of corrugated metal structures. It must be recognized that a buried flexible structure is a composite structure made up of the metal ring and the soil envelope, and that both materials play a vital part in the structural design of flexible metal structures. Only Article 12.7 is applicable to structural plate box culverts. 12.1.2 Notations A = required wall area (Article 12.2.1) A = area of pipe wall (Article 12.3.1) AL = total axle load on single axle or tandem axles (Articles 12.8.4.3.2 and 12.8.4.4) C, = number of axles coefficient (Article 12.8.4.3.2) C2 = number of wheels per axle coefficient (Article 12.8.4.3.2) Gee = live load adjustment coefficient (Article 12.8.4.3.2) D = straight leg of haunch (Article 12.8.2) Em = modulus of elasticity of metal (Articles 12.2.2 and 12.3.2) Em = modulus of elasticity of pipe material (Articles 12.2.4 and 12.3.4) FF = flexibility factor (Articles 12.2.4 and 12.3.4) fa = allowable stress-specified minimum yield point divided by safety factor (Article 12.2.1) = critical buckling stress (Articles 12.2.2 and 12.3.2) f, f, = specified minimum tensile strength (Articles 12.2.2 and 12.3.2) = specified minimum yield point (Article 12.3.1) f, H = height of cover above crown (Article 12.8.4.4) 1 = moment of inertia, per unit length, of cross section of the pipe wall (Articles 12.2.4 and 12.3.4) k = soil stiffness factor (Articles 12.2.2 and 12.3.2) MIC = dead load factored moment (Article 12.8.4.3.1) MEe= live load factored moment (Article 12.8.4.3.2) M, = crown plastic moment capacity (Article 12.8.4.3.3) MPh =haunch plastic moment capacity (Article 12.8.4.3.3) P = design load (Article 12.1.4) P = proportion of total moment carried by the crown. Limits for P are given in Table 12.7.4D (Article 12.8.4.3.3) r = radius of gyration of corrugation (Articles 12.2.2 and 12.3.2) = radius of crown (Table 12.8.2A) r, rh = radius of haunch (Table 12.8.2A) R = rise of box culvert (Articles 12.7.2 and 12.8.4.4) Rh = haunch moment reduction factor (Article 12.8.4.3.3) S = diameter of span (Articles 12.1.4, 12.2.2, 12.8.2, and 12.8.4.4) s = pipe diameter or span (Articles 12.2.4,12.3.2, and 12.3.4) SF = safety factor (Article 12.2.3) SS = required seam strength (Articles 12.2.3 and 12.3.3) T = thrust (Article 12.1.4) TL = thrust, load factor (Articles 12.3.1 and 12.3.3) T, = thrust, service load (Articles 12.2.1 and 12.2.3) t = length of stiffening rib on leg (Article 12.8.2) V = reaction acting in leg direction (Article 12.8.4.4) A = haunch radius included angle (Table 12.8.2A) y = unit weight of backfill (Articles 12.8.4.3.2 and 12.8.4.4) #I = capacity modification factor (Articles 12.3.1, 12.3.3, 12.5.3.1, 12.6.1.3, and 12.8.4.2) 12.1.3 Loads Design load, P, shall be the pressure acting on the structure. For earth pressures, see Article 3.20. For live load, see Articles 3.4 to 3.7,3.11,3.12, and 6.4, except that the HIGHWAY BRIDGES words "When the depth of fill is 2 feet or more" in Article 6.4.1 need not be considered. For loading combinations, see Article 3.22. 12.1.4 Design 12.1.4.1 The thrust in the wall shall be checked by three criteria. Each considers the mutual function of the metal wall and the soil envelope surrounding it. The criteria are: (a) Wall area; (b) Buckling stress; (c) Seam strength (structures with longitudinal seams). (3) The density of the embankment material above the pipe must be determined. See Article 6.2. (b) Dimensions of soil envelope. The general recommended criteria for lateral limits of the culvert soil envelope are as follows: (1) Trench installations-2-feet minimum each side of culvert. This recommended limit should be modified as necessary to account for variables such as poor in situ soils. (2) Embankment installations-one diameter or span each side of culvert. (3) The minimum upper limit of the soil envelope is 1 foot above the culvert. 12.1.4.2 The thrust in the wall is: 12.1.6.2 Pipe Arch Design where: P = design load, in pounds per square foot; S = diameter or span, in feet; T = thrust, in pounds per foot. 12.1.4.3 Handling and installation strength shall be sufficient to withstand impact forces when shipping and placing the pipe.

12.15 Materials The materials shall conform to the AASHTO specifications referenced herein. 12.1.6 Soil Design 12.1.6.1 Soil Parameters The performance of a flexible culvert is dependent on soil structure interaction and soil stiffness. The following must be considered: (a) Soils: (1) The type and anticipated behavior of the foundation soil must be considered; i.e., stability for bedding and settlement under load. (2) The type, compacted density, and strength properties of the soil envelope immediately adjacent to the pipe must be established. Good side fill is obtained from a granular material with little or no plasticity and free of organic material, i.e., AASHTO classification groups A-1, A-2, and A-3, compacted to a minimum 90% of standard density based on AASHTO Specification T 99 (ASTM D 698). The design of the comer backfill shall account for comer pressure which shall be considered to be approximately equal to thrust divided by the radius of the pipe arch comer. The soil envelope around the comers of pipe arches shall be capable of supporting this pressure.

12.1.6.3 Arch Design 12.1.6.3.1 Special design considerations may be applicable; a buried flexible structure may raise two important considerations. The first is that it is undesirable to make the metal arch relatively unyielding or fixed compared with the adjacent sidefill. The use of massive footings or piles to prevent any settlement of the arch is generally not recommended. Where poor materials are encountered, consideration should be given to removing some or all of this poor material and replacing it with acceptable material. The footing should be designed to provide uniform longitudinal settlement, of acceptable magnitude from a functional aspect. Providing for the arch to settle will protect it from possible drag down forces caused by the consolidation of the adjacent sidefill.

The second consideration is bearing pressure of soils under footings. Recognition must be given to the effect of depth of the base of footing and the direction of the footing reaction from the arch. Footing reactions for the metal arch are considered to act tangential to the metal plate at its point of connection to the footing. The value of the reaction is the thrust in the metal arch plate at the footing.

12.1.6.3.2 Invert slabs and other appropriate measures shall be provided to anticipate scour. 12.1.7 DIVISION I-DESIGN Extra metal thickness, or coatings, may be required for resistance to corrosion and abrasion. For highly abrasive conditions, a special design may be required. 12.1.8 Minimum Spacing When multiple lines of pipes or pipe arches greater than 48 inches in diameter or span are used, they shall be spaced so that the sides of the pipe shall be no closer than one-half diameter or 3 feet, whichever is less, to permit adequate compaction of backfill material. For diameters up to and including 48 inches, the minimum clear spacing shall not be less than 2 feet. 12.1.9 End Treatment Protection of end slopes may require special consideration where backwater conditions may occur, or where erosion and uplift could be a problem. Culvert ends constitute a major run-off-the-road hazard if not properly designed. Safety treatment, such as structurally adequate grating that conforms to the embankment slope, extension of culvert length beyond the point of hazard, or provision of guardrail, are among the alternatives to be considered. End walls on skewed alignment require a special design. 12.1.10 Construction and Installation The construction and installation shall conform to Section 23-Division 11. 12.2 SERVICE LOAD DESIGN Service Load Design is a working stress method, as traditionally used for culvert design. 12.2.1 Wall Area 341 Corrugations with the required wall area, A, shall be checked for possible buckling. If the allowable buckling stress, fcJSF, is less than fa, the required area must be recalculated using fcJSFin lieu of f,. Formulae for buckling are: where: f, specified minimum tensile strength in pounds per square inch; f, = critical buckling stress in pounds per square inch; k = soil stiffness factor = 0.22; S = diameter or span in inches; r = radius of gyration of corrugation in inches; E,= modulus of elasticity of metal in pounds per square inch. = 12.2.3 Seam Strength For pipe fabricated with longitudinal seams (riveted, spot-welded, bolted), the seam strength shall be sufficient to develop the thrust in the pipe wall. The required seam strength shall be where: SS = required seam strength in pounds per foot; T, = thrust in pipe wall in pounds per foot; SF = safety factor. 12.2.4 Handling and Installation Strength Handling and installation rigidity is measured by a flexibility factor, FF,determined by the formula: where: A = required wall area in square inches per foot; T, = thrust, service load in pounds per foot; fa = allowable stress-specified minimum yield point, pounds per square inch, divided by safety factor, f,/SF. where: FF= flexibility factor in inches per pound; s = pipe diameter or maximum span in inches; Em= modulus of elasticity of the pipe material in pounds per square inch; 342 HIGHWAY BRIDGES I = moment of inertia per unit length of cross section of the pipe wall in inches to the 4th power per inch. 12.3 LOAD FACTOR DESIGN Load Factor Design is an alternative method of design based on ultimate strength principles. 12.2.4 where: SS = required seam strength in pounds per foot; TL = thrust multiplied by applicable factor, in pounds + per linear foot; = capacity modification factor.

12.3.4 Handling and Installation Strength 12.3.1 Wall Area Handling rigidity is measured by a flexibility factor, FF, determined by the formula: where: where: A = area of pipe wall in square inches per foot; TL = thrust, load factor in pounds per foot; f, = specified minimum yield point in pounds per square inch; = capacity modification factor. + 12.3.2 Buckling If f,, is less than f,, A must be recalculated using f,, in lieu of f,: then f,, = f, - f' (k s ~ r) ~(12-8) 48E, FF = flexibility factor in inches per pound; s = pipe diameter or maximum span in inches; Em= modulus of elasticity of the pipe material in pounds per square inch; I = moment of inertia per snit length of cross section of the pipe wall in inches to the 4th power per inch. 12.4 CORRUGATED METAL PIPE 12.4.1 General 12.4.1.1 Corrugated metal pipe and pipe-arches may be of riveted, welded, or lock seam fabrication with annular or helical corrugations. The specifications are: Aluminum where: f, specified minimum metal strength in pounds per square inch; f,, = critical buckling stress in pounds per square inch; k = soil stiffness factor = 0.22; s = pipe diameter or span in inches; r = radius of gyration of corrugation in inches; Em= modulus of elasticity of metal in pounds per square inch. = Steel AASHTO M 36, M 190, M 245 AASHTO M 190, M 196 12.4.1.2 Service Load Design-safety factor, SF Seam strength Wall area Buckling = 3.0 = 2.0 = 2.0 12.4.1.3 Load Factor Design--capacity modificatio~factor, I$ 12.3.3 Seam Strength For Helical pipe with lock seam or fully welded seam: For pipe fabricated with longitudinal seams (riveted, spot-welded, bolted), the seam strength shall be sufficient to develop the thrust in the pipe wall. The required seam strength shall be: Wall area and buckling + = 1.0 For Annular pipe with spot welded, riveted or bolted seam: Wall area and buckling Seam strength + = 1.0 + = 0.67 12.4.1.4 Flexibility Factor (a) For steel conduits, FF should generally not exceed the following values: %-in. and Yz-in. depth corrugation, FF = 4.3 X 10-2 1-in. depth corrugation, FF = 3.3 X (b) For aluminum conduits, FF should generally not exceed the following values: 74-in. and %in. depth corrugations, for 0.060 in. material thickness FF = 3.1 X for 0.075 in. material thickness FF = 6.1 X for all other material thicknesses FF = 9.2 X I-in. depth corrugation, FF = 6 X 12.4.1.5 Minimum Cover The minimum cover for design loads shall be Span18 but not less than 12 inches. (The minimum cover shall be measured from the top of a rigid pavement or the bottom of a flexible pavement.) For construction requirements, see Article 26.6-Division 11. 12.6.3.2 Aluminum Conduits & 9" x 2%" Cormgations r I x loJ Thickness (in.) (sq in.lft) (in.) (in.4/in.) 0.100 0.125 0.150 0.175 0.200 0.225 0.250 1.404 1.750 2.100 2.449 2.799 3.149 3.501 0.8438 0.8444 0.8449 0.8454 0.8460 0.8468 0.8473 83.065 103.991 124.883 145.895 166.959 188.179 209.434 348 HIGHWAY BRIDGES 1 2 . m h e m i c a l and Mechanical Properties 12.6.4.1 Aluminum Structural Plate Pipe, PipeArch, and Arch Material Requirements-AASHTO M 219, Alloy 5052 Mechanical Properties for Design Thickness (in.) Minimum Tensile Strength (Psi) Minimum Yield Point @si) Mod. of Elast. (psi) 0.100 to 0.175 0.176 to 0.250 35,000 34,000 24,000 24,000 10 x lo6 10 x lo6 12.6.4.2 Steel Structural Plate Pipe, Pipe-Arch, and Arch Material RequirementsAASHTO M 167 Mechanical Properties for Design Minimum Tensile Strength (psi) 45,000 Minimum Yield Point (psi) 33,000 12.6.4 construction and installation shall conform to Section 26-Division 11. 12.7.2 Structure Design 12.7.2.1 General Long-span structures shall be designed in accordance with Articles 12.1 and 12.6, and 12.2 or 12.3 except that the requirements for buckling and flexibility factor shall not apply. The span in the formulae for thrust shall be replaced by twice the top arc radius. Long-span - - structures shall include acceptable special features. Minimum requirements are detailed in Table 12.7.2A. TABLE 12.7.2A M i u m Requirements for Long-Span Structures with Acceptable Special Features I. TOP ARC MINIMUM THICKNESS Too Radius (ftftl Mod.of Elast. (psi) 29 x lo6 6" x 2" Cormgated Steel Plates 0.109 in. 0.138 in. 0.168 in. 0.218 in. 0.249 in. 11. MINIMUM COVER IN FEET TOP RADIUS (Fl1 12.6.5 Structural Plate Arches The design of structural plate arches should be based on ratios of a rise to span of 0.3 minimum. Steel Thicknessa ininches .I09 15 15-17 17-20 20-23 23-25 2.5 12.7 LONG-SPAN STRUCTURAL PLATE STRUCTURES 12.7.1 General Long-span structural plate structures are short-span bridges defined as follows: III. GEOMETRIC LIMITS A. Maximum Plate

shallow covers and extreme shapes of box culverts require special design procedures. Requirements of Articles 12.1 through 12.7 are not applicable to box culvert designs unless included in Article 12.8 by specific reference. 12.8.1.1 Scope Article 12.8 presents structural capacity requirements for box culverts based on the load factor method. Standard 12.7.5.3.2 shapes, soil requirements, and permissible product details for box culverts in compliance with this specification are defined. 12.8.2 Structural Standards The design criteria presented in subsequent articles are applicable only to structures in compliance with the standards described in Article 12.8. 12.8.2.1 Structural plate box culverts shall be bolted. The box culvert materials specifications are Aluminum AASHTO M 219 Steel AASHTO M 167 12.8.2.2 Reinforcing ribs shall be an aluminum or steel structural section curved to fit the structural plates. Ribs shall be bolted to the plates so as to develop the plastic moment capacity required. Spacing between ribs shall not exceed 2 feet on the crown and 4.5 feet on the haunch. Rib splices shall develop the plastic moment capacity required at the location of the splice. 12.8.2.3 Plastic moment capacities of ribbed sections may be computed using minimum yield strength values for both rib and corrugated shell. Such computed values may be used for design only after they have been confirmed by representative flexural test data. (Reference Article 10.48.1). 12.8.3 Structure Backfill 12.8.3.1 Structure backfill material shall conform to the requirements of Article 12.7.2.4, compacted to a minimum 95% of standard density based on AASHTO T 99 or 90% of standard density based on AASHTO T 180. 12.8.3.2 Specified structure backfill material shall be 3 feet wide, minimum, at the footing and shall extend upward to the road base elevation. TABLE 12.8.2A Geometric Requirements for Box Culverts I. Span, (S), may vary from 8 ft-9 in. to 25 ft-5 in. II. Rise,(R), may vary from 2 ft-6 in. to 10 ft-6 in. III. Radius of crown, (r,) = 24 ft-9% in. maximum IV. Radius of haunch, (rh) = 2 ft-6 in. minimum V. A may vary from 50' to 70" VI. Length of leg, @), measured to the bottom of the plate, may vary from 0.4 ft to 5.9 ft. VII. Minimum length of rib on leg, (t), is either 19 in.; the length of leg, (D), minus 3 in.
or to within 3 in. of the top of a concrete footing, whichever is less.
12.8.4 DIVISION I-DESIGN 355 factor (P). P represents the proportion of the total moment that can be canied by the crown of the box culvert and varies with the relative moment capacities of the crown and haunch components. Limits for P are given in Table 12.8.4B. Crown 12.8.4.3.1 The sum of the factored crown and haunch dead load moments are FIGURE 1282A Standard Terminology of Structural Plate Box Culvert Shapes 12.8.4 Design 12.8.4.1 Analytical Basis for Design Structural requirements for box culverts have been developed from finite element analyses covering the range of structures allowed by Article 12.8.2. 12.8.4.1.1 Structural requirements are based on analyses using two dimensional live loads equivalent to HS 20, 4-wheel, single-axle vehicles. Dead load of soil equals 120 pounds per cubic foot. Coefficients to adjust for other load conditions are contained in Article 12.8.4.3.2. (S3[0.0053 - 0.00024(S - 12)) Mde = y X (12-12) 0.053 (H - 1.4)SZ] X (Dead load, load factor) + where MdE= The sum of the factored crown and haunch dead load moments (kip-fttft) S = Box culvert span in feet. y = Soil density (lbs/ft3) H = Height of cover from the box culvert rise to top of pavement (ft) 12.8.4.3.2 The sum of the factored crown and haunch live load moments are Mcc = (CetK1SIK2) X (Live load, load factor) 12.8.4.1.2 Backfill required in Article 12.8.3 is dense granular material. The analyses that provide the basis for this specification were based on conservative soil properties of low plasticity clay (CL) compacted to 90% of standard AASHTO T 99. 12.8.4.2 Load Factor Method The combined gamma and beta factors to be applied are Dead load, load factor = 1.5 Live load, load factor = 2.0 The capacity modification factor + is 1.00. 12.8.4.3 Plastic Moment Requirements Analyses covering the range of box culvert shapes described in Article 12.8.2 have shown moment requirements govern the design in all cases. Effects of thrust were found to be negligible when combined with moment. Metal box culverts act similar to rigid frames, distributing moment between the crown and haunch on the basis of their relative stiffness. Within limits, increasing the stiffness of one component of the box (either crown or haunch) reduces the portion of the total moment carried by the other. Article 12.8 provides for this moment distribution within the allowable limits of the moment proportioning (12-13) where Met = The sum of the factored crown and haunch live load moments (kip-ftlft) Cee = Live load adjustment coefficient for axle loads, tandem axles, and axles with other than 4 wheels: AL = Total axle load on single axle or tandem axles in kips; C, = Adjustment coefficient for number of axles; I CI = 1.O, for single axle; CI = (0.5 + SISO), for tandem axles, (C, I1.0); S = Box culvert span in feet; C1 = Adjustment coefficient for number of wheels per axle.
(Values for C2 are given in Table 12.8.4A.) H = Height of cover from the box culvert rise to top of pavement (ft.) K, =- 0.08 for 8 IS < 20 (H/S)02 ' (12-15) 356 12.8.4.3.2 HIGHWAY BRDGES K2= 0.54H2 - 0.4H + 5.05, for 1.4 5 H < 3.0 (12-17) K2 = 1.90H + 3, for 3.0 5 H 5 5.0 2 4 8 Cover Depth, ft 1.4 2.0 3.0 1.18 1.00 0.63 1.21 1.24 1.m 0.82 1.00 0.70 Cover Depth, ft 1.4 2 3 4 to 5 0.66 0.74 0.87 1.MI (12-18) TABLE 12.8.4A C,, Adjustment Coefficient Values for Number of Wheels Per Axle wheels Per Axle TABLE 12.8.4C R,,, Haunch Moment Reduction Values 5.0 1.02 l.OO 0.93 If Equation (12-19) indicates a higher P factor than permitted by the ranges of Table 12.8.4B, the actual crown is over designed, which is acceptable. However, in this case only the maximum value of P allowed by the table shall be used to calculate the required haunch moment capacity from Equation (12-20). 12.8.4.4 12.8.4.3.3 Crown plastic moment capacity (Mpc +), and haunch plastic moment capacity (Mph +), must be equal to or greater than the proportioned sum of load adjusted dead and live load moments.
Footing Reactions The reaction at the box culvert footing may be computed using the following equation V = y(HS/2,000 + S2/40,000) + AU[8 + 2(H + R)] (1 2-21) where V = Reaction in kips per foot acting in the direction y = Backfill unit weight in pounds per cubic foot; = Height of cover over the crown in feet; = Span of box culvert in feet; of the box culvert straight side; where H P = Proportion of total moment canied by the crown. Limits for P are given in Table 12.8.4D; % =Haunch moment reduction factor from Table 12.8.4E. S AL = Axle load in kips; R = Rise of box culvert in feet. 12.8.5 12.8.4.3.4 Article 12.8 can be used to check the adequacy of manufactured products for compliance with the requirements of this specification. Using the actual crown moment capacity provided by the box culvert under consideration and the loading requirements of the application, Equation (12-19) is solved for the factor P. This factor should fall within the allowable range of Table 12.8.4B. Knowing the factor P, Equation (12-20) is then solved for required haunch moment capacity, which should be less than or equal to the actual haunch moment capacity provided. TABLE 12.8.4B P, Crown Moment ProportioningValues Span ft Allowable Range of P Less Than 10 10-15 15-20 20-26 0.55 to 0.70 0.50 to 0.70 0.45 to 0.70 0.45 to 0.60 Manufacturing and Installation 12.85.1 Manufactureand assembly of structural plates shall be in accordance with Division 11, Articles 23.3.1.4, 26.3.2, 26.3.3, 26.3.4, and 26.4.1. Reinforcing ribs shall be attached as shown by the manufacturer. Bolts connecting plates, plates to ribs and rib splices shall be torqued to 150-foot pounds.
12.8.5.2 Sidefill and overfill per Article 12.8.3 shall be placed in uniform layers not exceeding 8 inches in compacted thickness at near optimum moisture with equipment and methods which do not damage or distort the box culvert. 12.8.53 Following completion of roadway paving, crown deflection due to live load may be checked.
After a minimum of 10 loading cycles with the design live load, the change in rise loaded with the design live load relative to the rise unloaded, should not exceed '/XO of the box culvert span. Section 13 WOOD STRUCTURES 13.1 GENERAL AND NOTATIONS 13.1.1 General The following information on wood design is generally based on the National Design Specification for Wood Construction (NDSB), 1991 Edition. See the 1991 Edition of the NDSB for additional information. 13.1.2 Net Section In determining the capacity of wood members, the net section of the member shall be used. Unless otherwise noted, the net section shall be determined by deducting from the gross section, the projected area of all material removed by boring, grooving, dapping, notching or other means. 13.1.3 Impact In calculating live load stresses in wood, impact shall be neglected unless otherwise noted. See Article 3.8.1. 13.1.4 Notations a = coefficient based on support conditions for ta- b c = width of bending member (Article 13.6.4.3) = coefficient based on sawn lumber, round timber pered columns (Article 13.7.3.4.2) piles, glued laminated timber or structural composite lumber (Article 13.7.3.3.5) = load duration factor (Article 13.5.5.2) CD = bending size factor for sawn lumber, strucCF tural composite lumber, and for glued laminated timber with loads applied parallel to the wide face of the laminations (Article 13.6.4.2) = compression size factor for sawn lumber (footCF notes to Table 13.5.1A) = tension size factor for sawn lumber (footnotes CF to Table 13.5.1A) and structural composite lumber (footnotes to Tables 13.5.4A and 13.5.4B) = sheer stress factor (footnotes to Table 13.5.1A) CH CL = beam stability factor (Article 13.6.4.4) CM = wet service factor (Article 13.5.5.1) Cp = column stability factor (Article 13.7.3.3) Cv = volume factor for glued laminated timber with loads applied perpendicular to the wide face of the laminations (Article 13.6.4.3) = bearing area factor (Article 13.6.6.3) Cb = form factor (Article 13.6.4.5) Cf = flat use factor for sawn lumber (footnotes to Cf, Table 13.5.1A) = repetitive member factor for sawn lumber (footC, notes to Table 13.5.1A) d = depth of member (Article 13.6.4.2.2) d, = maximum column face dimension (Article 13.7.3.4.2) = minimum column face dimension (Article d 13.7.3.4.2) = representative dimension for a tapered column d,, face (Article 13.7.3.4.2) = tabulated modulus of elasticity (Article 13.6.3) E E' = allowable modulus of elasticity (Article 13.6.3) = tabulated unit stress in bending (Article 13.6.4.1) F, = allowable unit stress in bending (Article 13.6.4.1) FA = adjusted tabulated bending stress for beam staF$ bility (Article 13.6.4.4.5) Fc = tabulated unit stress in compression parallel to grain (Article 13.7.3.2) = allowable unit stress in compression parallel to FL grain (Article 13.7.3.2) = adjusted tabulated stress in compression parFz allel to grain for column stability (Article 13.7.3.3.5) = actual unit stress in compression parallel to grain fc (Article 13.7.3.1) Fc, = tabulated unit stress in compression perpendicular to grain (Article 13.6.6.2) = allowable unit stress in compression perpendicFL ular to grain (Article 13.6.6.2) = tabulated unit stress in bearing parallel to grain F, (Article 13.7.4.1) = allowable unit stress in bearing parallel to grain FL (Article 13.7.4.1) HIGHWAY BRIDGES 358 = tabulated unit stress in tension parallel to grain (Article 13.8.1) = allowable unit stress in tension parallel to grain (Article 13.8.1) = tabulated unit stress in shear parallel to grain (Article 13.6.5.3) = allowable unit stress in shear parallel to grain (Article 13.6.5.3) = actual unit stress in shear parallel to grain (Article 13.6.5.2) = allowable unit stress for bearing on an inclined surface (Article 13.6.7) = column effective length factor (Article 13.7.3.3.3) = material factor for beam stability (Article 13.6.4.4.5) = material factor for column stability fArticle 13.7.3.3.5) = length of bending member between points of zero moment (Article 13.6.4.3.1) = actual column length between points of lateral support (Article 13.7.3.3.3) = length of bearing (Article 13.6.6.3) = effective bending member length (Article 13.6.4.4.3) = effective column length (Article 13.7.3.3.3) = unsupported bending member length (Article 13.6.4.4.3) = parameter for the specific material determined in accordance with the requirements of ASTM D 5456 (Tables 13.5.4A and 13.5.4B) = bending member slenderness ratio (Article 13.6.4.4.4) = vertical shear (Article 13.6.5.2) = maximum vertical shear at 3d or Ll4 due to wheel loads distributed laterally as specified for moment (Article 13.6.5.2) = distributed live load vertical shear (Article 13.6.5.2) = maximum vertical shear at 3d or L14 due to undistributed wheel loads (Article 13.6.5.2) = species variable for computing the volume factor (Article 13.6.4.3.1) = angle between the direction of load and the direction of grain (Article 13.6.7) 13.2 MATERIALS 13.1.4 13.2.1.2 Dimensions 13.2.1.2. I Structural calculations for sawn lumber shall be based on the net dimensions of the member for the anticipated use conditions. These net dimensions depend on the type of surfacing, whether dressed, roughsawn or full-sawn. 13.2.1.2.2 For dressed lumber, the net dry dimensions given in Table 13.2.1A shall be used for design, regardless of the moisture content at the time of manufacture or in use. 13.2.1.2.3 Where the design is based on rough, fullsawn or special sizes, the applicable moisture content and dimensions used in design shall be noted in the plans and specifications. TABLE 13.2.1A Net Dry Dimensions for Dressed Lumber Nominal Thickness Dry Thickness Nominal Width -Dry Width -- Dimension Lumber (inches): 2 I-v2 2 2-v2 2-112 3 3 3-Y2 3-112 4 4 4-v2 2 1-10 3 2-10 4 3-10 5 4-1/2 6 5-10 8 7-114 9-114 10 12 11-1/4 13-ll4 14 16 15-114 Beams and Stringers and Posts and 'Ilmbers (inches): 5 and 1 0less 5 and 1/2 less than greater than greater nominal nominal 13.2.2 Glued Laminated Timber 13.2.2.1 General Glued laminated timber shall comply with the requirements of AASHTO M 168 and shall be manufactured -using wet-use adhesives. 13.2.1 Sawn Lumber 13.2.2.2 Dimensions 13.2.1.1 General Sawn lumber shall comply with the requirements of AASHTO M 168. 13.2.2.2.1 Structural calculations for glued laminated timber shall be based on the net finished dimensions. 13.2.2.2.2 DIVISION I-DESIGN 13.2.2.2.2 For Western Species and Southern Pine, the standard net finished widths shall be as given in Table 13.2.2A. Other, nonstandard finished widths may be used subject to design requirements. TABLE 13.2.2A Standard Net Finished Widths of Glued Laminated Timber Manufactured from Western Species or Southern Pine Nominal Width (in.) 4 6 8 10 12 14 16 Wesiern Species Net Finished Width (in.) Southern Pine Net Finished Width (in.) 3 5 6314 8-112 10-112 12 14 13.2.3 Structural Composite Lumber 13.2.3.1 General Structural composite lumber, including laminated veneer lumber and parallel strand lumber, shall comply with the requirements of ASTM D 5456 and shall be manufactured using wet-use adhesives which comply with requirements of ASTM D 2559. 13.2.3.2 Laminated Veneer Lumber Laminated veneer lumber shall consist of a composite of wood veneer sheet elements with wood fibers oriented primarily along the length of the member. Veneer thickness shall not exceed 0.25 inches. 13.2.3.3 Parallel Strand Lumber Parallel strand lumber shall consist of wood strand elements with wood fibers oriented primarily along the length of the member. The least dimension at the strands shall not exceed 0.25 inches and the average length shall be a minimum of 150 times the least dimension. 13.2.3.4 Dimensions Structural calculations for structural composite lumber shall be based on the net finished dimensions. 13.2.4 Piles Wood piles shall comply with the requirements of AASHTO M 168. 359 13.3 PRESERVATIVETREATMENT 13.3.1 Requirement for Treatment All wood used for structural purposes in exposed permanent applications shall be pressure impregnated with wood preservative in accordance with the requirements of AASHTO M 133. 13.3.2 Treatment Chemicals All structural members that are not subject to direct pedestrian contact shall preferably be treated with oil-type preservatives. Members that are subject to direct pedestrian contact, such as rails and footpaths, shall be treated with waterborne preservatives or oilborne preservativesin light petroleum solvent. Direct pedestrian contact is considered to be contact which may be made while the pedestrian is situated anywhere in the access route provided for pedestrian traffic. 13.3.3 Field Treating Insofar as is practicable,all wood members shall be designed to be cut, drilled, and otherwise fabricated prior to pressure treatment with wood preservatives. When cutting, boring, or other fabrication is necessary after preservative treatment, exposed, untreated wood shall be specified to be field treated in accordance with the requirements of AASHTO M 133. 13.3.4 Fire Retardant Treatments Fire-retardant chemicals shall not be used unless it is demonstrated that they are compatible with the preservative treatment. When fire retardants are used, design values shall be reduced by the strength and stiffness reduction factors specified by the fire retardant chemical manufacturer. 13.4 DEFLECTION 13.4.1 The term "deflection" as used herein shall be the deflection computed in accordance with the assumptions made for loading when computing stress in the members. 13.4.2 Flexural members of bridge structures shall be designed to have adequate stiffness to limit deflections or any deformations that may adversely affect the strength or serviceability of the structure. 360 HIGHWAY BRIDGES 13.4.3 13.4.3 Members having simple or continuous spans preferably should be designed so that the deflection due to service live load does not exceed 11500 of the span. are graded to Beam and Stringer grade requirements, the tabulated unit bending stress for the applicable Beam and Stringer grades may be used. 13.4.4 For timber deck structures with timber girders or stringers of equal stiffness, and cross-bracing or diaphragms sufficient in depth and strength to ensure lateral distribution of loads, the deflection may be computed by considering all girders or stringers as acting together and having equal deflection. When the cross-bracing or diaphragms are not sufficient to laterally distribute loads, deflection shall be distributed as specified for moment. 13.5.2.2.4 Beam and Stringer grades are normally graded for use as a single, simple span. When used as a continuous beam, the grading provisions customarily applied to the middle third of the simple span length shall be applied to the middle two-thirds of the length for two-span beams, and to the entire length for beams continuous over three or more spans. 13.4.5 For concrete decks on wood girders or stringers, the deflection shall be assumed to be resisted by all beams or stringers equally. 13.5 DESIGN VALUES 13.5.1 General Stress and modulus of elasticity values used for design, referred to as allowable design values, shall be the tabulated values modified by all applicable adjustments required by this Section. The actual stress due to loading shall not exceed the allowable stress. 13.5.2 Tabulated Values for Sawn Lumber 13.5.2.1 Tabulated values for sawn lumber are given in Table 13.5.1A for visually graded lumber and Table 13.5.1Bfor mechanically graded lumber. Values for bearing parallel to grain are given in Table 13.5.2A. These values are taken from the 1991 Edition of the represent a partial listing of available species and grades. Refer to the 1991 Edition of the a more complete listing. 13.5.2.2 Stress Grades in Flexure 13.5.2.2.1 The tabulated unit bending stress for Dimension (2 to 4 inches thick) and Post and Timber grades applies to material with the load applied either to the narrow or wide face. 13.5.3 Tabulated Values for Glued Laminated Timber 13.5.3.1 Tabulated values for glued laminated timber of softwood species are given in Tables 13.5.3A and 13.5.3B. Values for bearing parallel to grain are given in Table 13.5.2A. These values are taken from the 1993 Edition of the American Institute of Timber Construction, AITC 117-93 Design, "Standard Specifications for Structural Glued Laminated Timber of Softwood Species." Refer to AITC 117-93 Design for a more complete listing. 13.5.3.2 Tabulated values for hardwood species shall be as given in the 1985 Edition of American Institute of Timber Construction, AITC 119, "Standard Specifications for Hardwood Glued Laminated Timber." 13.5.3.3 Species other than those specifically included or referenced in this Section may be used, provided that tabulated values are established for each species in accordance with AASHTO M 168. 13.5.4 Tabulated Values for Structural Composite Lumber 13.5.4.1 Representative tabulated design values for structural composite lumber are given in Table 13.5.4A for laminated veneer lumber and Table 13.5.4B for parallel strand lumber. 13.5.5 Adjustments to Tabulated Design Values 13.5.5.1 Wet Service Factor, Cw The tabulated unit bending stress for Decking grades applies only when the load is applied to the wide face. 13.5.2.2.2 13.5.2.2.3 The tabulated unit bending stress for Beam and Stringer grades applies only when the load is applied to the narrow face. When Post and Timber sizes 13.5.5.1.1 Tabulated values for sawn lumber assume that the material is installed and used under continuously dry conditions where the moisture content of the wood does not exceed 19%. When the moisture content at installation or in service is expected to exceed 19%, tabulated values shall be reduced by the wet service fac- TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers Design Values in Pounds per Square Inch (psi) Species and Commercial Grade Size Classification Compression Perpendicular to Grain Compression Parallel to Grain Fv Fc, Fc 1000 775 675 575 95 95 95 95 625 625 625 625 1700 1500 1450 1300 1,900,000 1,800,000 WWPA 1,700,000 1,600,000 WCLIB 1100 950 775 675 425 85 85 730 625 730 625 625 1300 1100 1100 925 1,700,000 1,600,000 1,700,000 1,600,000 1,300,000 WCLIB Shear Parallel to Grain Fb Tension Parallel to Grain Ft 1450 1150 lo00 875 1900 1600 1550 1350 875 Bending Modulus of Elasticity E Grading Rules Agency DOUGLAS FIR-LARCH Select Structural No. 1 & Btr No. 1 No. 2 Dense Select Structural Select Structural Dense No. 1 No. 1 No. 2 Dense Select Structural Select Structural Dense No. 1 No. 1 No. 2 Dense Select Structural Select Structural Dense No. 1 No. 1 Dense No. 2 No. 2 Dense Select Structural Select Structural Dense No. 1 No. 1 Dense No. 2 No. 2 T-4" thick 2" & wider Beams and Stringers 85 85 85 600 Posts and Timbers 1,700,000 1,600,000 1,700,000 1,600,000 1,400,000 1,300,000 WWPA Beams and Stringers Posts and Timbers 1750 1500 1400 1200 700 1150 1000 950 825 550 475 85 85 85 85 85 730 625 730 625 730 625 1350 1150 1200 1000 550 475 1,700,000 1,600,000 1,700,000 1760%000 1,400,OC'O 1,300,000 1250 775 575 575 350 275 70 70 70 335 335 335 1200 1000 825 1,2009000 NELMA 1,100,000 1,100,000 NSLB 800 EASTERN SOFTWOODS Select Structural No. 1 No. 2 2 " 4 thick 2" & wider TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued) Design Values in Pounds per Square Inch (psi) Species and Commercial Grade Size Classification Bending Tension Parallel to Grain Shear Parallel to Grain Compression Perpendicular to Grain Compression Parallel to Grain Fb Ft Fv FcI Fc 1400 1060 950 850 900 700 600 500 75 75 75 75 405 405 405 405 1500 1350 1300 1250 1,~,000 1,500,000 1,500,000 1,300,000 WCLIB Modulus of Elasticity E Grading Rules Agency HEM-FIR Select Structural No. 1 & Btr No. 1 No. 2 2"-4" thick 2" & wider Select Structural No. 1 No. 2 Beams and Stringers 1300 1050 675 750 525 350 70 70 70 405 405 405 925 750 500 1,300,000 1,300,000 1,100,000 WCLIB Select Structural No. 1 No. 2 Posts and Timbers 1200 975 575 800 650 375 70 70 70 405 405 405 975 SO 575 1,300,000 1,300,000 1,100,000 select structural No. 1 No. 2 Beams and Stringers 1250 1050 675 725 525 325 70 70 70 405 405 405 925 775 475 1,300,000 l,3omo 1,100,Oo WWPA Select Structural No. 1 No. 2 Posts and Timbers 1200 950 525 800 650 350 70 70 70 405 405 405 975 850 375 1,300,000 1,300,000 1,100,000 MIXED SOUTHERN PINE Select Structural No. 1 No. 2 2"4"thick 2"4"wide 2050 1450 1300 1200 875 775 100 100 90 565 565 565 565 1800 1650 1650 1,600,000 1,500,000 l,~,OOo Select Structural No. 1 No. 2 2"-4" thick 5"-6"wide 1850 1300 1150 1100 750 675 90 90 90 565 565 565 1700 1550 1550 1,600,000 1,500,000 1,~,000 SpIB 1750 1200 1050 lo00 700 625 90 90 565 565 565 565 1600 1450 1450 1,600,000 1,500,000 1,400,000 1500 1050 925 875 600 550 565 565 565 160P 90 1450 1450 1,600,000 1,500,000 1,400,000 Select Structural No. 1 No. 2 2"-4"thick Select Structural No. 1 No. 2 2 " 4 thick 8"wide 10" wide 90 90 90 TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued) Design Values in Pounds per Square Inch (psi) - Species and Commercial Grade Select Structural No. 1 No. 2 Size Classification 2"-4"thick 1 2 wide Bending Fb 1400 975 875 - Tension Parallel to Grain Ft Shear Parallel to Grain Compression Perpendicular to Grain Compression Parallel to Grain Fv F.1 Fc Modulus of Elasticity E 825 575 525 90 90 90 565 565 565 1550 1400 1400 1,600,000 1,500,000 1,400,000 Grading Rules Agency MMED SOUTHERN PINE (Dry or Wet Service Conditions) Select Structural No. 1 No. 2 5" x 5" & larger 1500 1350 850 1000 900 550 110 110 95 375 375 375 900 800 525 1400 1000 975 800 575 575 110 110 110 885 885 885 1150 925 725 1,300,000 1,300,000 SPIB w"A000 NORTHERN RED OAK Select Structural No. 1 No. 2 2"-4" thick 2" & wider 1,400,000 1,400,000 1,300,000 Select Structural No. 1 No. 2 Beams and Stringers 1600 1350 875 950 675 425 105 105 105 885 885 885 950 800 500 Select Structural No. 1 No. 2 Posts and Timbers 1500 1200 700 1000 800 475 105 105 105 885 885 885 1000 875 400 1,300,000 1,300,000 1,000,000 1300 925 900 750 550 525 105 105 105 615 615 615 1100 900 700 1,700,000 1,600,000 1,500,000 1,300,000 1,300,000 1,000,000 NELMA RED MAPLE Select Structural No. 1 No. 2 2"-4"thick 2" & wider Select Structural No. 1 No. 2 Beams and Stringers 1500 1250 800 875 625 400 100 100 100 615 615 615 900 750 475 1,500,000 1,500,000 1,200,000 NELMA Select Structural No. 1 No. 2 Posts and Timbers 1400 1150 650 925 750 425 100 100 100 615 615 615 950 825 375 1,500,000 1,500,000 1,200,000 TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued) Design Values in Pounds per Square Inch (psi) Species and Commercial Grade Size Classification Bending Tension Parallel to Grain Shear Parallel to Grain Compression perpendicular to Grain Compression piallel to Grain Modulus of Elasticity Fb Ft Fv I?c, Fc E 1150 825 800 675 500 475 85 85 85 820 820 820 1000 825 625 1,400,000 1,300,000 1,200,000 Grading Rules Agency RED OAK Select Structural No. 1 No. 2 2"4"thick 2" & wider Select Structural No. 1 No. 2 Beams and Stringers 1350 1150 725 800 550 375 80 80 80 820 820 820 825 700 450 1,200,000 1,200,000 1,000,000 NELMA Select Structural No. 1 No. 2 Posts and Tllbers 1250 1000 575 850 675 400 80 80 80 820 820 820 875 775 350 1,200,000 1,200,000 1.o00,~ SOUTHERN PINE Select Structural No. 1 No. 2 2"-4" thick 2"-4"wide 2850 1850 1500 1600 1050 825 100 100 90 565 565 565 2100 1850 1650 1,800,600 SPIB 1,700,000 1 , ~ , ~ Select Structural No. 1 No. 2 2"4"thick 5"-6"wide 2550 1650 1250 1400 900 725 90 90 90 565 565 565 2000 1750 1600 1,800,000 1,700,000 1,600,000 Select Structural No. 1 No. 2. 2300 2"-4"thick 8" wide 1500 1200 1300 825 650 90 90 90 565 565 565 1900 1650 1550 1,800,000 1,700,000 1,600,000 2"-4" thick 10" wide 2050 1300 1050 1100 725 575 90 90 90 565 565 565 1850 1600 1500 1,800,000 1,700,000 1,600,000 1900 1250 975 1050 675 550 90 90 90 565 565 565 1800 1600 1450 1,800,000 1,700,000 1,600,000 Select Structural No. 1 No. 2 Select Structural No. 1 No. 2 2"4"thick 12" wide TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued) Design Values in Pounds per Sauar Inch (psi) Species and Commercial Grade Size Classification Bending Fb Tension Parallel to Grain Ft Shear Parallel to Grain Compression Perpendicular to Grain Compression Parallel to Grain Fv FC, Fc Modulus of Elasticity E Grading Rules Agency SOUTHERN PINE (Dry or Wet Service Conditions) 5" x 5" & larger 1750 1500 1350 850 1200 1000 900 550 110 110 110 100 440 375 375 375 1100 950 825 525 1,600,000 1,500,000 1,500,000 SPIB 1,200,000 Select Structural No. 1Mo. 2 2 " 4 ' thick 2" & wider 1250 875 675 425 70 70 425 425 1400 1100 1,500,000 1,400,000 Select Structural No. 1 No. 2 Beams and Stringers 1100 900 600 650 450 300 65 65 65 425 425 425 775 625 425 1,300,000 1,300,000 1,000,000 Select Structural No. 1 No. 2 Posts and Timbers 1050 850 500 700 550 325 65 65 65 425 425 425 800 700 500 1,300,000 1,300,000 1,oOo7000 1300 850 750 575 400 325 70 70 70 335 335 335 1200 1050 975 1,300,000 1,200,000 1,100,000 NELMA Dense Select Structural Select Structural No. 1 No. 2 SPRUCE-PINE-FIR NLGA SPRUCE-PINE-FIR (SOUTH) Select Structural No. 1 No. 2 2"4" thick 2" & wider Select Structural No. 1 No. 2 Beams and Stringers 1050 900 575 625 450 300 65 65 65 335 335 335 675 575 350 1,200,000 1,200,000 WCLIB 1,000,000 NSLB Select Structural No. 1 No. 2 Posts and Timbers 1000 800 350 675 550 225 65 65 65 335 335 335 700 625 225 1,200,000 WWPA 1,200,000 1,000,000 1000 725 700 575 425 400 75 75 75 420 420 420 900 725 575 1,500,000 1,400,000 NLSB 1,300,000 YELLOW POPLAR Select Structural No. 1 No. 2 Y-4" thick 2" & wider TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued) 1. Design values are taken from the 1991 Edition of the NDS' and are for a 10-year load duration and dry service conditions.
Refer to the 1991 NDS' for additional species and grades and for a summary of grading rules agencies and commercial species classfications. 2. Wet Service Factor, CM.When dimension lumber, 2"to 4"thick is used where moisture content will exceed 19%, design values shall be multiplied by the following wet service factors: WET SERVICE FACTORS, CM 0.85* 1.0 0.97 0.67 0.8'. 0.9 when (Fb)(Cp)s 1,150 psi, CM= 1.0 **when F, s 750 psi, CM= 1.0 When timbers 5" by 5" and larger are used where moisture content will exceed 19%, design values shall be multiplied by the following wet service factors (for Southern Pine and Mixed Southern Pine, use tabulated values without further adjustment): WET SERVICE FACTORS, CM Fb Ft Fcl Fv Fe E 3. Size Factor, &.For all species other than Southern Pine and Mixed Southern Pine, tabulated bending, tension, and compression parallel to grain design values for dimension lumber 2" to 4" thick shall be multiplied by the following size factors: SIZE FACTORS, Cp Grades Width Select Structural, No. 1 & Btr. No. 1, No. 2, No. 3 2",3" & 4" 5" 6" 8" 10" 12" 14" & wider Thickness 2" & 3" 4" 1.5 1.4 1.3 1.2 1.1 1 .o 0.9 1.5 1.4 1.3 1.3 1.2 1.1 1.0 1.5 1.4 1.3 1.2 1.1 1.0 0.9 1.15 1.1 1.1 1.05 1.0 1.0 0.9 TABLE 13.5.1A Tabulated Design Values for Visually Graded Lumber and Timbers (Continued) For Southern Pine and Mixed Southern Pine dimensionlumber, 2" to 4" thick, appropriate sue adjustment factors have been incorporated in tabulated values, with the following exceptions: For dimension lumber 4" thick, 8"and wider, tabulated bending design values shall be multiplied by the sue factor, CF= 1.1. For dimension lumber wider than 12", tabulated bending, tension, and compression parallel to grain design values for 12" wide lumber shall be multiplied by the size factor, Cp= 0.9. 4. Flat Use Factor, G.Bending design values are based on edgewise use (load applied to narrow face). When dimension lumber 2" to 4" thick is used flatwise (load applied to wide face), the bending design value shall be multiplied by the following flat use factors: FLAT USE FACTORS. Cn, Thickness Width 2" & 3" 4" 2" & 3" 4" 5" 6" 8" 10" & wider 1.0 1.1 1.1 1.15 1.15 1.2 1.0 1.05 1.05 1.05 1.1 ...
c.
5. Repetitive Member Factor, Bending design values for dimension lumber 2" to 4" thick shall be multiplied by the repetitive member factor C,= 1.15, when such members are used as stringers, decking or similar members which are in contact or are spaced not more than 24" on centers, are not less than 3 in number and are joined by load distributing elements adequate to support the design load. 6. Shear Stress Factor, CH.Tabulated shear design values parallel to grain,Fv, have been reduced to allow for the occurrence of splits, checks, and shakes and may be multiplied by the shear stress factors given below when the length of split, or size of check or shake is known and no increase in them is anticipated. When the shear stress factor is applied to Southern Pine or Mixed Southern Pine, a tabulated design value of FV= 90 lblin.' shall be used for all grades. Shear stress factors shall be linearly interpolated. SHEAR STRESS FACTORS, CH Length of split on wide face of 2" (nominal) lumber CH no split..; .....2 . 112 x wide face ..... 1.67 314 x wide face ..... .1.50 1 x wide face .....1.33 1-112 x wide face or more .....1.00 Length of split on wide face of 3" (nominal) and thicker lumber Sue of shake* in 2" (nominal) and thicker lumber CH no split .....
2 . 112 x narrow face 1.67 314 x narrow face... ..... 1.5 1. x narrow face.. .....1 . 3 1-Y2 x narrow face or more 1.00 .... CH no shake.. ..... 2.00 l/6 X narrow face.....1.67 114 X narrow face.. .....1.50 l/3 X narrow face.. ..... 1.33 Y2 x narrow face or more ..... 1.00 *Shake is measured at the end between lines enclosing the shake and perpendicular to the loaded face.
368 HIGHWAY BRIDGES 13.5.5.1.1 TABLE 13.5.1B Tabulated Design Values for Mechanically Graded Dimension Lumber Design Values in Pounds per Square Inch (psi) Species and Commercial Grade Size Classification Tension Compression Modulus Parallel Parallel of Bending to Grain to Grain Elasticity Fb Ft F, E Grading Rules Agency MACHINE STRESS RATED (MSR) LUMBER 1200 1350 1450 1500 1500 1650 1650 1800 1950 1950 2100 2250 2250 2400 2400 2550 2700 2850 3000 3150 3300 WCLIB, WWPA NLGA, SPIB, WCLIB, WWPA SPIB NLGA, SPIB, WWPA SPIB NLGA, SPIB, WCLIB, WWPA SPIB, WWPA NLGA, SPIB, WWPA NLGA, SPIB, WCLIB, WWPA SPIB, WCLIB, WWPA NLGA, WCLIB, WWPA SPIB NLGA, SPIB, WCLIB, WWPA SPIB NLGA, SPIB, WCLIB, WWPA NLGA, SPIB, WCLIB, WWPA SPIB NLGA, SPIB, WWPA NLGA, SPIB, WCLIB, WWPA SPIB NLGA, SPIB, WWPA SPIB NLGA, SPIB, WCLIB, WWPA NLGA, SPIB, WWPA SPIB NLGA, SPIB, WCLIB, WWPA NLGA, SPIB, WCLIB, WWPA SPIB NLGA, SPIB SPIB 900 1200 1350 1500 1800 NLGA, WCLIB NLGA, WCLIB NLGA WCLIB NLGA, WCLIB 900 2" & less in thickness 6" & wider 1. Design values are taken from the 1991 Edition of the NDS' and are for a 10-year load duration and dry service conditions. Refer to the 1991 NDS' for additional grades and for a summary of grading rules agencies. 2. Design values for shear parallel to grain and compressionperpendicular to grain shall be as specified in Table 13.5.1Afor No. 2 visually graded dimension lumber of the appropriate species. 3. Use of the wet service factor, shear stress factor, repetitive member factor, and flat use factor shall be as specified in Table 13.5.1A for visually graded dimension lumber. tors, CM,given in footnotes to Tables 13.5.1A and 13.5.1B. 13.5.5.1.2 Tabulated values for glued laminated timber and structural composite lumber assume that the material is used under continuously dry conditions where the moisture content in service does not exceed 16%. When the moisture content in service is expected to exceed 16%, tabulated values shall be reduced by the wet service factors, CM,given in the footnotes to Tables 13.5.3A and 13.5.3B for glued laminated timber and Tables 13.5.4A and 13.5.4B for structural composite lumber. 13.5.5.1.3 The moisture content of wood used in exposed bridge applications will normally exceed 19% and tabulated values shall be reduced by the wet service factor unless an analysis of regional, geographic, and climatological conditions that affect moisture content indicate that the in-service moisture content will not exceed 19% for sawn lumber and 16% for glued laminated timber and structural composite lumber over the life of the structure. 13.5.5.2 DIVISION I-DESIGN 369 TABLE 13.5.2A Tabulated Design

service conditions or extraordinary hazardous conditions, the use of lower design values than those obtained above may be necessary. Refer to the 1991 Edition of the NDSD. 13.7.3.4 Tapered Columns 13.7.3.4.1 For rectangular columns tapered at one or both ends, the cross-sectional area shall be based on the representative dimension of each tapered face. The representative dimension, d, of each tapered face shall be based on the support condition coefficient given in Table 13.7.1A. 13.7.3.4.2 For support conditions given in Table 13.7.1A, the representative dimension, LP, of each tapered face shall be as given by the following equation: d, = d, + (d, - d,,) [I)(: : : a - 0.15 1 - - 13.7.3.4.3 For support conditions other than those in Table 13.7.1A, the representative dimension of each tapered face shall be as given by the following equation: 13.7.3.4.4 For any tapered column, the actual stress in compression parallel to grain, f,, shall not exceed the allowable stress determined by Equation (13-14), assuming the column stability factor Cp = 1.O. 13.7.3.5 Round Columns The design of a round column shall be based on the design of a square column of the same cross-sectional area with the same degree of taper.
13.7.4 Bearing Parallel to Grain 13.7.4.1 The actual stress in bearing parallel to grain shall be based on the net area and shall not exceed the tabulated stress for bearing parallel to grain adjusted by the applicable adjustment factor given in the following equation: where: Fi = allowable unit stress in bearing parallel to grain in psi; F, = tabulated unit stress in bearing parallel to grain from Table 13.5.2A, in psi; CD= load duration factor from Article 13.5.5.2. 13.7.4.2 When the bearing load is at an angle to the grain, the allowable bearing stress shall be determined by Equation (13-14), using the design values for end-grain bearing parallel to grain and design values in compression perpendicular to grain.
13.7.4.3 When bearing parallel to grain exceeds 75% of the allowable value determined by Equation (13-21), bearing shall be on a metal plate or on other durable, rigid, homogeneous material of adequate strength and stiffness to distribute applied loads over the entire bearing area. 13.8 TENSION MEMBERS 13.8.1 Tension Parallel to Grain where: d, 13.7.3.4.2 = representative dimension for a tapered column face, in inches; d- = minimum column face dimension, in inches; dm,, = maximum column face dimension, in inches; a = coefficient based on support conditions. The allowable unit stress in tension parallel to grain hall be the tabulated value adjusted by the applicable adjustment factors given in the following equation: : F = FtCMCDCF (13-22) 13.8.1 DIVISION I-DESIGN where: F: = allowable unit stress in tension parallel to grain in psi; F, = tabulated unit stress in tension parallel to grain in psi; CM = wet service factor from Article 13.5.5.1; CD = load duration factor from Article 13.5.5.2; CF = tension size factor for sawn lumber from footnotes to Table 13.5.1A and for structural composite lumber from footnotes to Tables 13.5.4A and 13.5.4B. 13.8.2 Tension Perpendicular to Grain 383 13.9.2.2 All steel components, timber connectors, and castings, other than malleable iron, shall be galvanized in accordance with AASHTO M 111. 13.9.2.3 Alternative corrosion protection coatings, such as epoxies, may be used when the demonstrated performance of the coating is sufficient to provide adequate protection for the intended exposure conduction. 13.9.2.4 Heat-treated alloy components and fastenings shall be protected by an approved alternative protective treatment that does not adversely affect the mechanical properties of the material. Designs which induce tension perpendicular to the grain of wood members should not be used.
When tension perpendicular to grain cannot be avoided, mechanical'reinforcement sufficient to resist all such forces should be used. Refer to the 1991 Edition of the additional information. 13.9.3 Fasteners 13.9 MECHANICAL CONNECTIONS 13.9.3.2 When determining fastener design values, wood shall be assumed to be used under wet-use or exposed to weather conditions.
13.9.1 General 13.9.1.1 Except as otherwise required by this specification, mechanical connections and their installation shall conform to the requirements of the NDS@,1991 Edition. 13.9.1.2 Components at mechanical connections, including the wood members, connecting elements, and fasteners, shall be proportioned so that the design strength equals or exceeds the required strength for the loads acting on the structure. The strength of the connected wood components shall be evaluated considering the net section, eccentricity, shear, tension perpendicular to grain and other factors that may reduce component strength. 13.9.2 Corrosion Protection 13.93.1 Except as permitted by this section, all steel hardware for wood structures shall be galvanized in accordance with AASHTO M 232 or cadmium plated in accordance with AASHTO M 299. 13.9.3.1 Fastener design values shall be adjusted by the applicable adjustment factors for the intended use condition. 13.9.3.3 Glulam rivets shall not be used in permanent structures. 13.9.4 Washers 13.9.4.1 Washers shall be provided under bolt and lag screw heads and under nuts that are in contact with wood. Washers may be omitted under heads of special timber: bolts or dome-head bolts when the size and strength of the head is sufficient to develop connection strength without excessive wood crushing. 13.9.4.2 Washers shall be of sufficient size and strength to prevent excessive wood crushing when the fastener is tightened. For bolts or rods loaded in tension, washers shall be of sufficient size and strength to develop the tensile strength of the connection without excessive bending or exceeding wood strength in compression perpendicular to grain. Section 14 BEARINGS 14.1 SCOPE This section contains requirements for the design and selection of structural bearings. The selection and layout of the bearings shall be consistent with the proper functioning of the bridge, and shall allow for deformations due to temperature and other time dependent causes. The loads induced in the bearings and structural members depend on the stiffnesses of the individual elements and the tolerances achieved during fabrication and erection. These influences shall be taken into account when calculating design loads for the elements. Units used in this section shall be taken as KIP, IN, RAD, O F and Shore Hardness, unless noted. 14.2 DEFINITIONS Bearing-A structural device that transmits loads while facilitating translation and/or rotation. Bronze Bearing-A bearing in which displacements or rotations take place by the slip of a bronze surface against a mating surface. pad made from Cotton Duck Reinforced Pad (CDP)-A closely spaced layers of elastomer and cotton duck, bonded together during vulcanization. Disc Bearing-A bearing which accommodates rotation by deformation of a single elastomeric disc, molded from a urethane compound. It may contain a device for partially confining the disc against lateral expansion.
Double Cylindrical Bearing-A bearing made from two cylindrical bearings placed on top of each other with their axes at right angles to each other, in order to provide rotation about any horizontal axis. Fiberglass Reinforced Pad (FGP)-A pad made from discrete layers of elastomer and woven fiberglass, bonded together during vulcanization. Fixed Bearing-A bearing which prevents differential longitudinal translation of abutting structure elements.
It may or may not provide for differential lateral translation or rotation. Knuckle Bearing-A bearing in which a concave metal surface rocks on a convex metal surface to provide rotation capability about any horizontal axis. Longitudinal-The direction associated with the axis of the main structural trusses or girders in the bridge. Metal Rocker or Roller Bearing-A bearing which carries vertical load by direct contact between two metal surfaces and which accommodates movement by rolling of one surface with respect to the other. Movable Bearing-A bearing that facilitates differential horizontal translation of abutting structural elements in a longitudinal and/or lateral direction. It may or may not provide for rotation. A pad made exclusively of Plain Elastomeric Pad (PEP)elastomer. Pot Bearing-A bearing which carries vertical load by compression on an elastomeric disc confined in a steel cylinder and which accommodates rotations by deformations of the disc. PTFE Sliding Bearing-A bearing which carries vertical load by contact stresses between a PTFEsheet or woven fabric and its mating surface, and which permits movements by sliding of the PTFE over the mating surface. Rotation about the Longitudinal Axis-Rotation about an axis parallel to the longitudinal axis of the bridge. Rotation about the Transverse Axis-Rotation about an axis parallel to the transverse axis of the bridge. RMS-Root mean square. Sliding Bearing-A bearing which accommodates movement by slip of one surface over another. Steel Reinforced Elastomeric Bearing-A bearing made from alternate laminates of steel and elastomer, bonded together during vulcanization.
Translation-Horizontal movement of the bridge in the longitudinal or transverse direction. Transverse-The horizontal direction normal to the longitudinal axis of the bridge.
14.3 NOTATIONS A = Plan area of elastomeric bearing (in2) B . = length of pad if rotation is about its transverse axis, or width of pad if rotation is about its longitudinal axis (in) 386 c D Dd D, Dl Dz dj E E, E, e ,?I F, G H, hti h, h, h, I L M, n PD Pn. PL P, R R,, S HIGHWAY BRIDGES = Design clearance between piston and pot wall (in) = Diameter of the projection of the loaded surface of the bearing in the horizontal plane (in) = Diameter of disc element (in) = Internal pot diameter in pot bearing (in) = Diameter of curved surface of rocker or roller unit (in) = Diameter of curved surface of mating unit (DP= w for a flat plate) (in) = Diameter of the jh hole in an elastomeric bearing = Young's modulus (ksi) = Effective modulus in compression of elastomeric bearing (ksi) = Young's modulus for steel (ksi) = Eccentricity of loading on a bearing (in) = Allowable fatigue stress range for over 2,000,000 cycles (ksi) = Yield strength of the least strong steel at the contact surface (ksi) = Shear modulus of the elastomer (ksi) = Maximum horizontal load on the bearing or restraint considering all appropriate load combinations (kip) = Thickness of i" elastomeric layer in elastomeric bearing (in) = Thickness of thickest elastomeric layer in elastomeric bearing (in) = Total elastomer thickness in an elastomeric bearing (in) = Thickness of steel laminate in steel-laminated elastomeric bearing (in) = Moment of inertia (in4) = Length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis) (in) = Maximum bending moment (K-in) = Number of interior layers of elastomer = Compressive load due to dead load (kip) = Compressive load due to live plus dead load (kip) = Compressive load due to live load (kip) = Maximum compressive load considering all appropriate load combinations (kip) = Radius of a curved sliding surface (in) = Radial distance from center of bearing to object, such as an anchor bolt, for which clearance must be provided (in) = Shape factor of one layer of an elastomeric bearing Plan Area Area of Perimeter Free to Bulge - 14.3 LW for rectangular bearings without 2 h m (I,+ W) holes -- D for circular bearings without holes 4hm ,,t W = Pot wall thickness (in) w p = Height of piston rim in pot bearing (in) = Effective angle of friction angle in PTFE bear- A. = Maximum service horizontal displacement of the A, = Maximum shear deformation of the elastomer 6 = Instantaneous compressive deflection of bearing 6, = Maximum compressive deflection of bearing (in) E = Instantaneous compressive strain of a plain elas- ei = Instantaneous compressive strain in i~ elastomer 0 = Component of maximum service rotation in di- = Width of the bearing in the transverse direction (in) ings = tan-' (Hm/PD) bridge deck (in) (in) (in) tomeric pad layer of a laminated elastomeric bearing rection of interest on an elastomeric bearing under load for Article 14.6.5.3 = Maximum rotation due to dead load (rad) OD = Maximum rotation due to live load (rad) OL em, = Maximum rotation considering all appropriate load and deformation combinations about transverse axis (rad) 0,, = Maximum rotation considering all appropriate load and deformation combinations about longitudinal axis (rad) 0, = Maximum design rotation considering all appropriate load and deformation combinations including live and dead load, bridge movements, and construction tolerances (rad) p = Coefficient of friction a~ = Average compressive stress due to dead load (ksi) aL = Average compressive stress due to live load (ksi) an. = Average compressive stress due to total dead plus live load (ksi) a, = Maximum average compressive stress (ksi) 14.4 MOVEMENTS AND LOADS Bearings shall be designed to resist loads and accommodate movements. No damage due to joint or bearing movement shall be permitted under any appropriate load and movement combination. 14.4 DIVISION I-DESIGN Translational and rotational movements of the bridge shall be considered in the design of bearings. The sequence of construction shall be considered and all critical combinations of load and movement shall be considered in the design. Rotations about two horizontal axes and the vertical axis shall be considered. The movements shall include those caused by the loads, deformations and displacements caused by creep, shrinkage and thermal effects, and inaccuracies in installation. In all cases, both instantaneous and long-term effects shall be considered, but the influence of impact need not be included. The most adverse combination of movements shall be used for design.
All design requirements shall be tabulated in a rational form such as shown in Figure 14.4. 387 ings shall have lateral strength adequate to resist all applied loads and restrain unwanted translation. Combinations of different types of fixed or moveable bearings should not be used at the same expansion joint, bent or pier unless the effects of differing deflection and rotational characteristics on the bearings and structure are accounted for in the design. 14.5.1 Load and Movement Capabilities The movements and loads to be used in the design of the bearing shall be clearly defined on the contract drawings.
14.5.2 Characteristics 14.4.1 Design Requirements The minimum thermal movements shall be computed from the extreme temperature defined in Article 3.16 of Division I and the estimated setting temperature. Design loads shall be based on the load combinations and load factors specified in Section 3 of DivisionI. The design rotation, Om, for bearings such as elastomeric pads or steel reinforced elastomeric bearings which do not achieve hard contact between metal components shall be taken as the sum of: -the dead and live load rotations. -an allowance for uncertainties, which is normally taken as less than 0.005 rad. The design rotation, Om, for bearings such as pot bearings, disc bearings and curved sliding surfaces which may develop hard contact between metal components shall be taken as the sum of: -the greater of either the rotations due to all applicable factored loads or the rotation at the service limit state. -the maximum rotation caused by fabrication and installation tolerances, which shall be taken as 0.01 rad unless an approved quality control plan justifies a smaller value. -an allowance for uncertainties, which shall be taken as 0.01 rad unless an approved quality control plan justifies a smaller value. 14.5 GENERAL REQUIREMENTSFOR BEARINGS Bearings may be fixed or movable as required for the bridge design. Movable bearings may include guides to control the direction of translation.
Fixed and guided bear- The bearing chosen for a particular application must have appropriate load and movement capabilities. Those listed in Table 14.5.2-1 may be used as a guide. Figure 14.5.2-1 may be used as a guide in defining the different bearing systems. The following terminology shall apply to Table 14.5.2-1: S U L R = Suitable = Unsuitable = Suitable for limited applications = May be suitable but requires special considera- tions or additional elements such as sliders or guideways. Long. = Longitudinal axis Trans. = Transverse axis Vert. = Vertical axis 14.5.3 Forces in the Structure Caused by Restraint of Movement at the Bearing Horizontal forces and moments induced in the bridge by restraint of movement at the bearing shall be taken into account in the design of the bridge and the bearings. They shall be determined using the calculated movements and the bearing characteristics given in Article 14.6. 14.5.3.1 Horizontal Force Horizontal forces may be induced by sliding friction, rolling friction or deformation of a flexible element in the bearing. The force used for design shall be the largest one applicable. Sliding friction force shall be computed as 388 14.5.3.1 HIGHWAY BRIDGES - Bridge Name or Ref. Bearing Identification Mark Number of bearings required Seating Material Upper Suriace Lower Surface Allowable contact pressure (KSI) Average - Edge Load Vertical D e s ~ g nload effects (KIP) -Transverse Longitudinal Translation Irreversible Transverse Longitudinal Reversible Rotauon (RAD) Irreversible Reversible Maximum Bearing dimensions (IN) Upper surface Lower surface Overall height Tolerable movement of bearing under transient loads (IN) FIGURE 14.4 Transverse Longitudinal Transverse Longitudinal Transverse Longitudinal Transverse Transverse Longitudinal max. perm. min. 14.5.3.1 DIVISION I-DESIGN 389 Table 14.5.2-1 Bearing Suitability Movement Q p e of Bearing Long Trans Rotation about bridge axis indicated Trans Long Vert Resistance to Loads Vert Long Plain Elastomeric Pad Fiberglass Reinforced Pad Cotton Duck Reinforced Pad Steel-reinforced Elastomeric Bearing Plane Sliding Bearing Curved Sliding Spherical Bearing Curved Sliding Cylindrical Bearing Disc Bearing Double Cylindrical Bearing Pot Bearing Rocker Bearing Knuckle Bearing Single Roller Bearing Multiple Roller Bearing Lor frlotlan 91 ldlna Sur4aoa - Cyllndrloal b a r l n g Sphertoal b a r lng mum Pot Be a r t nu COKR LRYER El a s t o m o r i c Boar i n g FIGURE 14.5.2-1 'Qpical Bearing Components Trans HIGHWAY BRIDGES 390 where: H, = maximum horizontal load (kip) p = coefficient of friction P, = maximum compressive load (kip) The force required to deform an elastomeric element shall be computed as: 14.5.3.1 14.6 SPECIAL DESIGN PROVISIONS FOR BEARINGS The stress increases permitted for certain load combinations by Table 3.22.1A of this specification shall not apply in the design of bearings. 14.6.1 Metal Rocker and Roller Bearings where: G = shear modulus of the elastomer (ksi) A = plan area of elastomeric element or bearing (in2) A, = maximum shear deformation of the elastomer (in) h, = total elastomer thickness (in) Rolling forces shall be determined by test. 14.5.3.2 Bending Moment The bridge substructure and superstructure shall be designed for the largest moment, M,, which can be transferred by the bearing. For curved sliding bearings without a companion flat sliding surface, M, shall be estimated by: and for curved sliding bearings with a companion flat sliding surface, M, shall be estimated by: where: M, = maximum bending moment (K-in) R = radius of curved sliding surface (in) For unconfined elastomeric bearings and pads, M, shall be estimated by: where: I = moment of inertia of plan shape of bearing (in4) 8, = maximum design rotation (rad) Ec = effective modulus of elastomeric bearing in compression (ksi) The load deflection curve of an elastomeric bearing is nonlinear, so Ec is load-dependent. However, an acceptable constant approximation is: 14.6.1.1 General Design Considerations The rotation axis of the bearing shall be aligned with the axis about which the largest rotations of the supported member occur. Provision shall be made to ensure that the bearing alignment does not change during the life of the bridge. Multiple roller bearings shall be connected by gearing to ensure that individual rollers remain parallel to each other and at their original spacing. Metal rocker and roller bearings shall be detailed so that they can be easily inspected and maintained. 14.6.1.2 Materials Rocker and roller bearings shall be made of stainless steel conforming to ASTM A 240, or of structural steel conforming to AASHTO M 169 (ASTM A 108), M 102 (ASTM A 668), or M 270 (ASTM A 709) Grades 36, 50 or 50W. Material properties of M 169 (ASTM A 108), M 102 (ASTM A 668), and M 270 (ASTM A 709) steel are given in Tables 10.2A and 10.2B. 14.6.1.3 Geometric Requirements The dimensions of the bearing shall be chosen taking into account both the contact stresses and the movement of the contact point due to rolling. Each individual curved contact surface shall have a constant radius. Bearings with more than one curved surface shall be symmetric about a line joining the centers of their two curved surfaces. Bearings shall be designed to be stable. If the bearing has two separate cylindrical faces, each of which rolls on a flat plate, stability may be achieved by making the distance between the two contact lines no greater than the sum of the radii of the two cylindrical surfaces. 14.6.1.4 Contact Stresses The maximum compressive load, P,, shall satisfy: where: G = shear modulus of elastomer (ksi) S = shape factor" for cylindrical surfaces: 14.6.1.4 DMSION I-DESIGN for spherical surfaces: Pm 5 40(1-D,/D2)F; E: (14.6.1.4-2) where: Dl = the diameter of rocker or roller surface (in), and Dz = the diameter of the mating surface (in). Dz shall be taken as: positive if the curvatures have the same sign infinite if the mating surface is flat F, = specified minimum yield strength of the least strong steel at the contact surface (ksi) E, = Young's modulus for steel (ksi) W = Width of the bearing (in) 14.6.2 PTFE Sliding Surfaces PTFE,polytetrafluorethylene, may be used in sliding surfaces of bridge bearings to accommodate translation or rotation. All PTFE surfaces other than guides shall satisfy the requirements of this section. Curved PTFE surfaces shall also satisfy Article 14.6.3. 14.6.2.1 PTFE Surface The PTFE surface shall be made from pure virgin PTFE resin satisfying the requirements of ASTM D 4894 or D 4895. It shall be fabricated as unfilled sheet, filled sheet or fabric woven from PTFE and other fibers. Unfilled sheets shall be made from PTFE resin alone. Filled sheets shall be made from PTFE resin uniformly blended with glass fibers or other chemically inert filler. The maximum filler content shall be 15%. Sheet PTFE may contain dimples to act as reservoirs for lubricant.
Their diameter shall not exceed 0.32411 at the surface of the PTFE and their depth shall be not less than .08-inch and not more than half the thickness of the PTFE. The reservoirs shall be uniformly distributed over the surface area and shall cover more than 20% but less than 30% of it. Lubricant shall be silicone grease which satisfies military specification MIL-S-8660. Woven fiber PTFE shall be made from pure PTFE fibers. Reinforced woven fiber PTFE shall be made by interweaving high strength fibers, such as glass, with the PTFE in such a way that the reinforcing fibers do not appear on the sliding face of the finished fabric. 14.6.2.2 Mating Surface The PTFE shall be used in conjunction with a mating swface. Flat mating surfaces shall be stainless steel and curved mating surfaces shall be stainlesssteel or anodizedaluminium. Flat surfaces shall be a minimum #8 mirror finish 304 stainless steel and shall conform to ASTM A 167lA 264. 391 C w e d metallic surfaces shall not exceed 16 micro in RMS. Other surface finishes may be employed if the coefficient of friction is substantiated by test results. The mating surface shall be large enough to cover the PTFE at all times. 14.6.2.3 Minimum Thickness Requirements 14.6.2.3.1 PTFE For all applications, the thickness of the PTFE shall be at least 1/16 inch after compression. Recessed sheet PTFE shall be at least 3/16 inch thick when the maximum dimension of the PTFE is less than or equal to 24 inches, and 1/4 inch when the maximum dimension of the PTFE is greater than 24 inches. Woven fabric PTFE which is mechanically interlocked over a metallic substrate shall have a minimum thickness of 1/16 inch and a maximum thickness of 1/8 inch over the highest point of the substrate. 14.6.2.3.2 Stainless Steel Mating Su$aces The thickness of the stainless steel mating surface shall be at least 1/16 inch when the maximum dimension of the surface is less than or equal to 12 inches and l/s inch when the maximum dimension is larger than 12 inches. Backing plate requirements are specified in Article 14.6.2.6.2. 14.6.2.4 Contact Pressure The maximum contact stress, a,, between the PTFE and the mating surface shall be determined with the maximum compressive load, P,, using the nominal area.
The average contact stress shall be computed by dividing the load by the projection of the contact area onto a plane perpendicular to the direction of the load. The contact stress at the edge shall be computed by taking into account the maximum moment, M,, transferred by the bearing assuming a linear distribution of stress across the PTFE. Stresses shall not exceed those given in Table 14.6.2.4-1. Permissible stresses for intermediate filler contents shall be obtained by linear interpolation within Table 14.6.2.4-1. 14.6.2.5 Coefficient of Friction The design coefficient of friction of the PTFE sliding surface shall be determined from Table 14.6.2.5- 1. Intermediate values may be determined by interpolation.
The coefficient of friction shall be determined by using the stress level associated with the maximum compressive load, P,. Lesser values of the coefficient of friction may be used if verified by tests. Where friction is required to resist applied loads, the design coefficient of friction under dynamic loading may be taken as not more than 10% of the value listed in Table 14.6.2.5-1 for the bearing stress and PTFE type. 392 HIGHWAY BRIDGES 14.6.2.5 TABLE 14.6.2.4-1 Limits on Contact Stress for PTFE Ave. Contact Stress (KSI) Material Dead Load Edge Contact Stress (KSI) AU Loads Dead Load All Loads Unconfined P T E Unfilled sheets Filled sheets-These figures are for maximum filler content Confined sheet P ' Woven PTFE over a metallic substrate Reinforced woven PTFE over a metallic substrate TABLE 14.6.2.5-1 Design Coefficients of Friction Coefficient of Friction Type of PTFE Pressure (psi) 500 lo00 2000 >3000 Temperature ("I?) Dimpled Lubricated 68 -13 -49 Unfilled or Dimpled Unlubricated 68 -13 -49 Filled 68 -13 -49 Woven 68 -13 -49 The coefficients of friction in Table 14.6.2.5-1 are based on a#8 mirror finish mating surface. Coefficients of friction for rougher surface finishes must be established by test results in accordance with Division 11, Section 18. 14.6.2.6 Attachment mating surface so that interface corrosion cannot occur. The attachment shall be capable of resisting the maximum friction force which can be developed by the bearing under service loads. The welds used for the attachment shall be clear of the contact and sliding area of the PTFE surface. 14.6.2.6.1 PTFE 14.63 Bearings with Curved Sliding Surfaces Sheet PTFE confined in a recess in a rigid metal backirig plate for one half its thickness may be bonded or unbonded. Sheet PTFE which is not confined shall be bonded by an approved method to a metal surface or an elastomeric layer with a Shore A durometer hardness of at least 90. Woven PTFE on a metallic substrate shall be attached to the metallic substrate by mechanical interlocking which can resist a shear force no less than 0.10 times the applied compressive force. 14.6.2.6.2 Mating Su$ace The mating surface for flat sliding shall be attached to a backing plate by welding in such a way that it remains flat and in full contact with its backing plate throughout its service life. The weld shall be detailed to form an effective moisture seal around the entire perimeter of the Bearings with curved sliding surfaces shall consist of two metal parts with matching curved surfaces and a low friction sliding interface. The curved surfaces shall be either cylindrical or spherical. The material properties, characteristics, and frictional properties of the sliding interface shall satisfy the requirements of either Article 14.6.2 or Article 14.6.7. 14.6.3.1 Geometric Requirements The radius of the curved surface shall be large enough to assure that the maximum average bearing stress, om,on the horizontal projected area of the bearing at the maximum load, P,, shall satisfy the average stress requirements of Article 14.6.2.4 or Article 14.6.7.3. The maximum average bearing stress shall be taken as 14.6.3.1 a, DIVISION I-DESIGN 393 and For cylindrical bearings and: For spherical bearings H, L = maximum horizontal load. projected length of the sliding surface perpendicular to the rotation axis. PD = compressive load due to permanent loads. = radius of the curved sliding surface. R w = length of the cylindrical surface. p = angle between the vertical and applied loads. = maximum design rotation angle. See Article 0, 14.4.1. urn = maximum average contact stress permitted on the PTFE by Table 14.6.2.4-1.
Y = subtended semi-angle of the curved surface. where D = diameter of the projection of the loaded surface of the bearing in the horizontal plane (in) W = length of the cylinder (in) The two surfaces of a sliding interface shall have equal radii. = 14.6.3.2 Resistance to Lateral Load 14.6.4 Pot Bearings In bearings which are required to resist horizontal loads, either an external restraint system shall be provided, or for a cylindrical sliding surface the horizontal load shall be limited to H, 5 2RW urn sin(Y - P - 0,) sin f3 (14.6.3.2-1) and for a spherical surface the horizontal load shall satisfy H, 5 xR2 urn sin2(Y - P - 0,) sin P (14.6.3.2-2) Where 14.6.4.1 General Where pot bearings are provided with a PTFE slider to provide for both rotation and horizontal movement, such sliding surfaces and any guidance systems shall be designed in accordance with the appropriateArticles 14.6.2 and 14.6.9. The rotational elements of pot bearing shall satisfy the requirements of this section. They shall consist of at least a pot, a piston, an elastomeric disc, and sealing rings. For the purpose of establishing the forces and deformations imposed on a pot bearing, the axis of rotation shall be taken as lying in the horizontal plane at midheight of the elastomeric disc.
SURFACE AREA AVAIUBLE TO CARRY COMPRESSION EQUAL LENGTHS - 7 L FIGURE 14.6.3.2-1 I 394 HIGHWAY BRIDGES The minimum vertical load on a pot bearing should not be less than 20% of the vertical design load. 14.6.4.2 Materials The elastomeric disc shall be made from a compound based on virgin natural rubber or virgin neoprene. Its nominal hardness shall lie between 50 and 60 on the Shore 'A' scale. The pot and piston shall be made from structural steel conforming to AASHTO M 270 (ASTM A 709) Grades 36, 50 or 50W, or from stainless steel conforming to ASTM A 240. The finish of surfaces in contact with the elastomeric pad shall be smoother than 63 micro-in rms. Sealing rings satisfying Articles 14.6.4.5.1 and 14.6.4.5.2 shall be made from brass conforming to ASTM B 36 (half hard) for rings of rectangular cross-section, and Federal Specification QQB626, Composition 2, for rings of circular cross-section. 14.6.4.3 Geometric Requirements The depth of the elastomeric disc, h,, shall satisfy where D, = internal diameter of the pot (in) 8, = maximum design rotation specified in Article 14.4.1 (rad) The dimensions of the components shall satisfy the following requirements under the least favorable combination of maximum displacements and rotations: the pot shall be deep enough to permit the seal and piston rim to remain in full contact with the vertical face of the pot wall. contact or binding between metal components will not prevent further displacement or rotation. 14.6.4.1 under compressive load and simultmeously applied cyclic rotations. The seals shall also be adequate to prevent escape of elastomer under compressive load and simultaneously applied static rotation. Brass rings satisfying the requirements of either Article 14.6.4.5.1 or 14.6.4.5.2 may be used to satisfy the above requirements.
The Engineer may approve other sealing systems on the basis of experimental evidence. 14.6.4.5.1 Rings with rectangular cross-sections Three rings shall be used. Each ring shall be circular in plan, but shall be cut at one point around its circumference. The faces of the cut shall be on a plane at 45' to the vertical and to the tangent of the circumference. The rings shall be oriented so that the cuts on each of the three rings are equally spaced around the circumference of the pot. The width of each ring shall be equal to or greater than the larger of 0.02 D, or 1/4 inch, but it shall not exceed 3/4 inch. The depth of each shall be equal to or greater than 0.2 times the width. 14.6.4.5.2 Rings with circular cross-sections One circular closed ring shall be used with an outside diameter of D,. It shall have a cross-sectional diameter not less than the larger of 0.0175 D, or 5/16 inch. 14.6.4.6 Pot The pot shall consist at least of a wall and base. All components shall be designed to act as a single structural unit. The minimum thickness of the base shall exceed 0.06 D, and 3/4 inch when bearing directly against concrete or grout, and shall exceed 0.04 D, and 1/2 inch when bearing directly on steel girders or load distribution plates. The pot walls shall be thick enough to resist all the forces induced in them. In lieu of a more precise analysis, this requirement may be satisfied for unguided sliding pot bearings by using a minimum wall thickness such that 14.6.4.4 Elastomeric Disc The maximum average stress on the elastomer shall not exceed 3.5 ksi. To facilitate rotation, the top and bottom surfaces of the elastomer shall be treated with a lubricant which is not detrimental to the elastomer, or thin PTFE discs may be used on the top and bottom of the elastomer. 14.6.4.5 Sealing Rings A seal shall be used between the pot and the piston. The seals shall be adequate to prevent escape of elastomer and tw 2 3/4" where t, = pot wall thickness (in) a, = maximum average compressive stress (ksi) F, = yield strength of the steel (ksi) 14.6.4,7 Piston The piston shall have the same plan shape as the inside of the pot. Its thickness shall be adequate to resist the 14.6.4.7 DIVISION I-DESIGN loads imposed on it, but shall not be less than 6.0% of the inside diameter of the pot, D,, except at the rim. The diameter of the piston rim shall be the inside diameter of the pot less a clearance, c. The clearance, c, shall be as small as possible in order to prevent escape of the elastomer, but not less than 0.02 inch.
If the surface of the piston rim is cylindrical, the clearance shall satisfy where D, = internal diameter of pot (in) w = height of piston rim (in) 0, = design rotation specified in Article 14.4.1 (rad) 14.6.4.8 Lateral Loads Pot bearings which are subjected to lateral loads shall be proportioned so that the thickness, t, of the pot wall and the pot base shall satisfy For pot bearings which transfer lateral load through the piston w 2 I/N" and where w is the rim thickness of the piston which is in contact with the pot wall. 14.6.5 Steel Reinforced Elastomeric BearingsMethod B 14.6.5.1 General Steel reinforced elastomeric bearings shall consist of alternate layers of steel reinforcement and elastomer, bonded together. Tapered elastomer layers shall not be used. All internal layers of elastomer shall be of the same thickness. The top and bottom cover layers shall be no thicker than 70% of the internal layers. In addition to any internal reinforcement, bearings may have external steel load plates bonded to the upper or lower elastomer layers or both. 14.6.5.2 395 Material Properties The elastomer shall have a shear modulus between 0.08 and 0.175 ksi and a nominal hardness between 50 and 60 on the Shore A scale. The shear modulus of the elastomer at 73OF shall be used as the basis for design. If the elastomer is specified explicitly by its shear modulus, then that value shall be used in design and the other properties shall be obtained from Table 14.6.5.2-1. If the material is specified by its hardness, the shear modulus shall be taken as the least favorable value from the range for that hardness given in Table 14.6.5.2-1. Intermediate values shall in all cases be obtained by interpolation. For the purposes of bearing design, all bridge sites shall be classified as being in temperature Zones A, B, C, D or E. Characteristics for each zone are given in Table 14.6.5.2-2. In the absence of more precise information, Figure 14.6.5.2-1 may be used as a guide in selecting the zone required for a given region. Bearings shall be made from AASHTO low temperature grades of elastomer as defined in Section 18 of Division II. The minimum grade of elastomer required for each low temperature zone is specified in Table 14.6.5.2-2. Any of the three design options listed below may be used: specify the elastomer with the minimum low temperature grade indicated in Table 14.6.5.2-2 and determine the shear force transmitted by the bearing as specified in Article 14.5.3.1. specify the elastomer with the minimum low temperature grade for use when special force provisions are incorporated in the design and provide a low friction sliding surface, in which case the special force provision is that the bridge components shall be designed to withstand twice the design shear force specified in Article 14.5.3.1, or specify the elastomer with the minimum low temperature grade for use when special force provisions are incorporated in the design, but do not provide a low friction sliding surface, in which case the components of the bridge shall be designed to resist four times the design shear force as specified in Article 14.5.3.1. Table 14.4.5.2-1 Elastomer Properties At Different Hardnesses Hardness (Shore 'A') Shear modulus at 73"F(psi) deflection 25 yrs Instantaneousdeflection * 50 60 70 95-130 130-200 200-300 25% 35% 45% 396 HIGHWAY BRIDGES 14.6.5.3 Table 14.6.5.2-2 Low Temperature Zones and Elastomer Grades C D E 0 -30 14 3 -45 NIA 4 all others 7 2 0 0 2 3 A B 50 year low temperature ("F) Max.no. of days below 32°F 0 -20 3 Low temp. elastomer grade without special provisions Low temp. elastomer grade with special provisions Low Temperature Zone - NIA 5 5 FIGURE 14.65.2-1 Map of Low Temperature Zones Design Requirements 14.6.5.3 14.6.5.3.2 Compressive Stress In any bearing layer, the average compressive stress (ksi) shall satisfy the following: 14.6.5.3.1 Scope Bearings designed by the provisions of this section shall be subsequently tested in accordance with the requirements for steel reinforced elastomeric bearings of Article 18.7 of Division I1 of this Specification. Steel reinforced elastomeric bearings may also be designed under the provisions of Article 14.6.6. * for bearings subject to shear deformation ,u 5 1.6 ksi UT, 5 1.66 GS a, 0.66 GS (14.6.5.3.2-1) 1600 Shape factor 1400 = 12 1400 - e 60 durometer 1200 - reinforced 0 1 2 3 4 5 Compressive strain (%) 6 g 7 Compressive strain (%) FIGURE 14.6533-1 Load Deflection Behavior of Elastomeric Bearings 14.6.5.3.2 DIVISION I-DESIGN for bearings fixed against shear deformation uTL5 1.75 ksi , U 5 2.00 GS 397 h, = total elastomeric thickness (in) A = maximum service shear deformation of the elastomer (in) (14.6.5.3.2-2) 14.6.5.3.5 Combined Compression and Rotation where UL = average compressive stress due to the live load (ksi) on. = Average compressive stress due to total dead plus live load (ksi) G = shear modulus of elastomer (ksi) S = shape factor of the thickest layer of the bearing 14.6.5.3.3 Compressive Defiection Rotations shall be taken as the maximum possible difference in slope between the top and bottom surfaces of the bearing, They shall include the effects of initial lackof-parallelism and subsequent girder end rotation due to imposed loads and movements. Bearings shall be designed so that uplift does not occur under any combination of loads and corresponding rotation.
All rectangular bearings shall satisfy 2 Deflections due to total load and to live load alone shall , o 2 l.OGS(?)($) (14.6.5.3.5-1) be considered separately.
A maximum relative deflection of % inch across a joint is preferred. Instantaneous deflection shall be calculated as ~ O ~ ~ O W S : A bearing subject to shear deformation shall also satisfy Equation (14.6.5.3.5-2); those fixed against S = Xeihri (14'6'5'3'3-1) shear deformation shall also satisfy Equation (14.6.5.3.5-3).
where: ei = instantaneous compressive strain in the i" elastomer laver of a laminated elastomeric bearing h" = thickness of i~ elastomeric layer in elastomeric bearing (in) Values for ei shall be determined from test results or by rational analysis. The effects of creep of the elastomer shall be added to the instantaneous deflection when considering long-term deflections. They should be computed from information relevant to the elastomeric compound used. In the absence of material-specific data, the values given in Article 14.6.5.2 shall be used. In the absence of information specific to the particular bearing to be used, Figure 14.6.5.3.3-1 may be used. 14.6.5.3.4 Shear The horizontal movement of the bridge superstructure, A,, shall be taken as the maximum possible displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects computed in accordance with this Specification. The maximum shear deformation of the bearing, A,, shall be taken as A,, modified to account for the pier flexibility and construction procedures. If a low friction sliding surface is installed, A, need not be taken larger than the deformation corresponding to first slip. The bearing shall be designed so that ,o L 2.250GS1- 0 . 1 6 7 (+) (~ 1) where B = length of pad if rotation is about its transverse axis, or width of pad if rotation is about its longitudinal axis (in) G = shear modulus of elastomer (ksi) h* = thickness of the i" layer of elastomer (in) n = number of interior layers of elastomer, where interior layers are defined as those layers which are bonded on each face. Exterior layers are defined as those layers which are bonded only on one face. When the thickness of an exterior layer of elastomer is more than one-half the thickness of an interior layer, the parameter, n, may be increased by one-half for each such exterior layer. S = shape factor of the thickest layer of the bearing 8, = component of maximum service rotation in direction of interest (rad) an. = average compressive stress due to the total dead plus live load (ksi) All circular bearings shall satisfy where (11.6.1.3.5-1) 398 HIGHWAY BRIDGES A circular bearing subject to shear deformation shall also satisfy Equation (14.6.5.3.5-5); those fixed

Effective Area (in.?i.) 0.067 0.076 0.085 0.105 0.114 0.132 0.150 0.155 0.193 0.230 Moment of Inertia (i~~.~lin.) 0.042 0.049 0.055 0.070 0.075 0.087 0.120 0.101 0.123 0.143 Steel tunnel liner plates shall be of heavier gage or thickness or protected by coatings or other means when required for resistance to abrasion or corrosion. 15.7 BOLTS 15.7.1 Bolts and nuts used with lapped seams shall be not less than 5/s inch in diameter. The bolts shall conform to the specifications of ASTM A 449 for plate thickness equal to or greater than 0.209 inches and A 307 for plate thickness less than 0.209 inches. The nut shall conform to ASTM A 307, Grade A. 15.7.2 Circumferential seam bolts shall be A 307 or better for all plate thicknesses. 15.7.3 Bolts and nuts used with four flanged plates shall be not less than % inch in diameter for plate thicknesses to and including 0.179 inches and not less than 5 / 8 inch in diameter for plates of greater thickness.The bolts and nuts shall be quick acting coarse thread and shall conform to ASTM A 307, Grade A. 15.8 SAFETY FACTORS Longitudinal test seam strength Pipe wall buckling =3 =2 Section 16 SOIL-REINFORCED CONCRETE STRUCTURE INTERACTION SYSTEMS 16.1 GENERAL 16.1.1 Scope Specifications in this Section govern the design of buried reinforced concrete structures. A buried reinforced concrete element becomes part of a composite system comprising the reinforced concrete section and the soil envelope, both of which contribute to the structural behavior of the system. = width of section which resists M, N, V-Usually b = 12 inches (Article 16.4.6) = out-to-out horizontal span of pipe or box, ft (Ar- ticles 16.4.4, 16.4.5, 16.6.4, and 16.7.4.) = horizontal width of trench at top of pipe or box, ft (Articles 16.4.4, 16.6.4, and 16.7.4.) = bedding factor (Article 16.4.5) = earth load bedding factor = live load bedding factor = crack control coefficient for effect of cover and spacing of reinforcement (Article 16.4.6) 16.1.2 Notations = effective tension area of concrete surrounding the flexural tension reinforcement and hav-ing the same centroid as that reinforcement, divided by the number of bars or wires, sq in.; when the flexural reinforcementconsists of several bar sizes or wire the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest bar or wire used (Articles 16.6.4 and 16.7.4) = total active lateral pressure acting on pipe, lbslft (Article 16.4.5 and Figure 16.4C) = tension reinforcement area on width b, inS2/ft (Articles 16.4.6.6,16.6.4.7,16.7.4.7, and 16.8.5.7) = area of total inner cage reinforcement required in length b, in2/ft (Article 16.4.6.6) = area of total outer cage reinforcement required in length b, in2/ft (Article 16.4.6.6) = stirrup reinforcement area to resist radial tension forces on width b, ir~.~/ft in each line of stirrups at circumferential spacing s (Article 16.4.6) = required area of stirrups for shear reinforcement, in.2 (Article 16.4.6.6.6.2) = steel area required for an individual circumferential wire for flexure at a splice or at the point of maximum moment for quadrant mat reinforcement, in2 (Article 16.4.7) = out-to-out vertical rise of pipe, ft (Figure 16.4C) = load coefficient for embankment installations (Article 16.4.5) = load coefficient for trench installations (Article 16.4.4) = constant corresponding to the shape of pipe (Ar- ticle 16.4.5J = parameter which is a function of the distribution of the vertical load and the vertical reaction (Article 16.4.5) = crack control coefficient for type of reinforcement (Article 16.4.6) = distance from compression face to centroid of tension reinforcement, in. (Articles 16.4.6.6, 16.6.4.7, 16.7.4.7, and 16.8.5.7) = thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto (Articles 16.6.4.7, 16.7.4.7, and 16.8.5.7) = D-laad of pipe, three-edge bearing test load expressed in pounds per linear foot per foot of span to produce a 0.01-inch crack (Article 16.4.5) = inside diameter of pipe, in. = service load stress in reinforcing steel for crack control (Articles 16.6.4.7, 16.7.4.7, and 16.8.5.7) = maximum allowable strength of stirrup material, l b ~ l i n(Article .~ 16.4.6.6.6) = specified yield strength of reinforcement, l b ~ / i n . ~ (Article 16.4.6) 408 HIGHWAY BRIDGES factor for effect of curvature on diagonal tension (shear) strength in curved components (Article 16.4.6.6.5) F,, = factor for adjusting crack control relative to average maximum crack width of 0.01 inch when F, = 1.0 (Article 16.4.6.6.4) Fd = factor for crack depth effect resulting in increase in diagonal tension (shear) strength with decreasing d (Article 16.4.6.6.5) Fe = soil-structure interaction factor (Articles 16.4.4, 16.6.4, and 16.7.4) Fe, = soil structure interaction factor for embankment installations (Articles 16.4.4, 16.6.4, and 16.7.4) Fe2 = soil-structure interaction factor for trench installations (Articles 16.4.4, 16.6.4, and 16.7.4) = factor for process and local materials that afF, fect the radial tension strength of pipe (Article 16.4.6) = factor for pipe size effect on radial tension F, strength (Article 16.4.6.6.3.1) = factor for process and local materials that affect F, the shear strength of pipe (Article 16.4.6.6.5) = coefficient for effect of thrust on shear strength FN (Article 16.4.6.6.5) f,' = design compressive strength of concrete, 1bs/in2 (Articles 16.4.6, 16.6.2, and 16.7.2) h = overall thickness of member (wall thickness), in. (Articles 16.4.6.6,16.6.4.7,16.7.4.7, and 16.8.5.7) H = height of fill above top of pipe or box, ft (Articles 16.4.4, 16.4.5, 16.6.4, and 16.7.4) HAF = horizontal arching factor (Figure 16.4A) i = coefficient for effect of axial force at service load stress, f, (Articles 16.4.6.6.4, 16.6.4.7, 16.7.4.7, and 16.8.5.7) = coefficient for moment arm at service load stress, j f, (Articles 16.4.6.6.4, 16.6.4.7, 16.7.4.7, and 16.8.5.7) K = ratio of the active unit lateral soil pressure to unit vertical soil pressure-Rankine's coefficient of active earth pressure (Figures 16.4B-D) Ld = development length of reinforcing wire or bar, in (Article 16.4.7) M, = factored moment acting on length b as modified for effects of compressive or tensile thrust, inlbs/ft (Article 16.4.6.6.5) M, = moment acting on cross section of width, b, service load conditions, in-lbs/ft (Taken as absolute value in design equations, always +) (Articles 16.4.6.6.4, 16.6.4.7, 16.7.4.7, and 16.8.5.7) Mu = factored moment acting on cross section of width b, in.-lbs/ft (Article 16.4.6.6.6.1) n = number of layers of reinforcement in a cage-1 or 2 (Article 16.4.6.6.4) Fc = 16.1.2 = axial thrust acting on cross section of width b, service load condition (+ when compressive, - when tensile), lbs/ft (Articles 16.4.6.6.4,16.6.4.7, 16.7.4.7, and 16.8.5.7) = factored axial thrust acting on cross section of width b, lbs/ft (Article 16.4.6) = projection ratio (Article 16.4.5.2.1) = negative projection ratio (Figure 16.4Aand Table 16.4A) = PL denotes the prism load (weight of the column of earth) over the pipe's outside diameter, lbs/ft (Figure 16.4.A) = ratio of the total lateral pressure to the total vertical load (Article 16.4.5) = radius of the inside reinforcement, in. (Article 16.4.6.6.3.1) = settlement ratio (Article 16.4.5.2.1) = spacing of reinforcement wire or bar, in. (Article 16.4.6.6.6) = spacing of circumferential reinforcement,in. (Article 16.4.6.6.4) = internal horizontal span of pipe, in. (Articles 16.4.6.6 and 16.4.5.1) = clear cover over reinforcement, in. (Article 16.4.6.6.4) = basic shear strength of critical section, lbslft where M,/(V,d) = 3.0 (Article 16.4.6.6.5) = nominal shear strength provided by width b of concrete cross section, lbs/ft (Article 16.4.6.6.6) = factored shear force acting on cross section of width b, lbs/ft (Article 16.4.6.6.5) = factored shear force at critical section, lbs/ft where MnU/(Vud) = 3.0 (Article 16.4.6.6.5) VAF = vertical arching factor (Article 16.4.4.2.1.1) W = unit weight of soil, lbs/ft3 (Article 16.4.4) we = total earth load on pipe or box, lbs/ft (Articles 16.4.4, 16.4.5, 16.6.4, and 16.7.4) = fluid load in the pipe as determined according to Article 16.4.4.2.2, lbslft = total live load on pipe or box, lbs/ft (Articles 16.4.4 and 16.4.5) = total load, earth and live, on pipe or box, lbs/ft (Articles 16.4.4 and 16.4.5) = parameter which is a function of the area of the vertical projection of the pipe over which lateral pressure is effective (Article 16.4.5) = coefficient of internal friction of the soil (Figure 16.4B) = coefficient of friction between backfill and trench walls (Figure 16.4B) = central angle of pipe subtended by assumed distribution of external reactive force (Figure 16.4F) 16.1.2 p $ $ DIVISION I-DESIGN = ratio of reinforcement area to concrete area (Ar- ticle 16.4.6) = strength reduction factor for flexure (Article 16.4.6.6.1) = strength reduction factor for radial tension (Article 16.4.6.6.3.1) = strength reduction factors for shear (Article 16.4.6.6.5) 16.1.3 Loads Design loads shall be determined by the forces acting on the structure. For earth loads, see Article 3.20. For live loads see Articles 3.4 through 3.8 and Articles 3.11 and 3.12.

For loading combinations see Article 3.22. 16.1.4 Design Design may be based on working stress or ultimate strength principles. The design criteria shall include structural aspects (e.g. flexure, thrust, shear), handling and installation, and crack control. Footing design for cast-in-place boxes and arches shall be in conformity with Article 4.4. 409 16.2 SERVICE LOAD DESIGN 16.2.1 For soil-reinforced concrete structure interaction systems designed with reference to service loads and allowable stresses, the service load stresses shall not exceed the values shown in Article 8.15 except as modified herein. 16.2.2 For precast reinforced concrete circular pipe, elliptical pipe, and arch pipe, the results of three edgebearing tests made in accordance with AASHTO materials specifications may be used in lieu of service load design. 16.3 LOAD FACTOR DESIGN 16.3.1 Soil-reinforced concrete structure interaction systems shall be designed to have design strengths of all sections at least equal to the required strengths calculated for the factored loads as stipulated in Article 3.22, except as modified herein. 16.3.2 For precast reinforced concrete circular pipe, elliptical pipe, and arch pipe, the results of three edge-bearing tests made in accordance with AASHTO materials specifications may be used in lieu of load factor design. 16.1.5 Materials The materials shall conform to the AASHTO materials specifications referenced herein.

16.4 REINFORCED CONCRETE PIPE 16.4.1 Application 16.1.6 Soil Structural performance is dependent on soil structure interaction. The type and anticipated behavior of the material beneath the structure, adjacent to the structure, and over the structure must be considered. 16.1.7 Abrasive or Corrosive Conditions Where abrasive or corrosive conditions exist, suitable protective measures shall be considered. 16.1.8 End Structures Structures may require special consideration where erosion may occur. Skewed alignment may require special end wall designs. 16.1.9 Construction and Installation The construction and installation shall conform to Section 27, Division II. This Specification is intended for use in design for precast reinforced concrete circular pipe, elliptical pipe, and arch pipe. Standard dimensions are shown in AASHTO material specifications M 170, M 206, M 207, and M 242. Design wall thicknesses other than the standard wall dimensions may be used, provided the design complies with all applicable requirements of Section 16. 16.4.2 Materials 16.4.2.1 Concrete Concrete shall conform to Article 8.2 except that evaluation off,' may be based on cores. 16.4.2.2 Reinforcement Reinforcement shall meet the requirements of Articles 8.3.1 through 8.3.3 only, and shall conform to one of the following AASHTO material specifications M 31, M 32, M 55, M 221, or M 255. For smooth wire and smooth 410 HIGHWAY BRIDGES 16.4.2.2 TABLE 16,4A Standard Embankment Installation Soils and Minimum Compaction Requirements Haunch and Installation Type Bedding Thickness Outer Bedding Lower Side Type 1 Type 2 (See Note 3.) Type 3 (See Note 3.) m e4 Bc/24"(600 mm) minimum, not less than 3" (75 mm). If rock foundation, use Bc/12" (300 mm) minimum, not less than 6" (150 mm).

BC/24"(600 mm) minimum, not less than 3" (75 mm). If rock foundation, use Bc/12" (300 mm) minimum, not less than 6" (150 mm). BC/24"(600 mm) minimum, not less than 3" (75 mm).

If rock foundation, use Bc/12" (300 rnm) minimum, not less than 6" (150 mm). No bedding required, except if rock foundation, use B,/12" (300 mm) minimum, not less than 6" (150 mm). 95% SW 90% SW, 95% ML, or 90% SW or 95% ML 85% SW, 90% ML, or 95% CL 85% SW, 90% ML,or 95% CL 85% SW, 90% ML, or 95% CL No compaction required, except if CL, use 85% CL No compaction required, except if CL, use 85% CL NOTES: 1. Compaction and soil symbols 4.e. "95% S W refer to SW soil material with a minimum standard proctor compaction of 95%. See Table 16.4C for equivalent modified proctor values. 2. Soil in the outer bedding, haunch, and lower side zones, except within B,13 from the pipe springline, shall be compacted to at least the same compaction as the majority of soil in the overfill zone. 3. Only Type 2 and 3 installations are available for horizontal elliptical, vertical elliptical and arch pipe.

4.

SUBTRENCHES 4.1 A subtrench is defined as a trench with its top below finished grade by more than 0.1H or, for roadways, its top is at an elevation lower than 1' (0.3 m) below the bottom of the pavement base material. 4.2 The minimum width of a subtrench shall be 1.33 B,, or wider if required for adequate space to attain the specified compaction in the haunch and bedding zones. 4.3 For subtrenches with walls of natural soil, any portion of the lower side zone in the subtrench wall shall be at least as firm as an equivalent soil placed to the compaction requirements specified for the lower side zone and as firm as the majority of soil in the overfill zone, or shall be removed and replaced with soil compacted to the specified level. welded wire fabric, a yield stress of 65,000 psi and for deformed welded wire fabric, a yield stress of 70,000 psi may be used. 16.4.2.3 Concrete Cover for Reinforcement The minimum concrete cover for the reinforcement in precast concrete pipe shall be 1 inch in pipe having a wall thickness of 2%inches or greater and 3/4 inch in pipe having a wall thickness of less than 2%inches. 16.4.3 Installations 16.4.3.1 Standard Installations Standard Embankment Installations are presented in Figure 16.4B and Standard Trench Installations are presented in Figure 16.4C; these figures define soil areas and critical dimensions. Generic soil types, minimum compaction requirements, and minimum bedding thicknesses are listed in Table 16.4A for four Standard Embankment Installation Vpes and in Table 16.4B for four Standard Trench Installation Qpes. 16.4.3.2 Soils The AASHTO Soil Classifications and the USCS Soil Classifications equivalent to the generic soil types in the Standard Installations are presented in Table 16.4C. 16.4.4 Design 16.4.4.1 General Requirements Design shall conform to applicable sections of these specifications except as provided otherwise in this article. For design loads, see Article 16.1.3; for standard installation, see Article 16.4.3.1; and for bedding conditions, see Section 27, Division 11-Construction and the Soil-Structure Interaction Modifications that follow. Live loads, WL, shall be included as part of the total load, WT,and shall be distributed through the earth cover as specified in Article 6.4, except that the 2-foot minimum in the first paragraph of Article 6.4 does not apply. Other methods for determining total load and pressure distribution may be used, if they are based on successful design 16.4.4.1 DIVISION I-DESIGN 411 TABLE 16.4B Standard Trench Installation Soils and Minimum Compaction Requirements Installation Type Type 1 5 p e2 (See Note 3.) Type 3 (See Note 3.) 5 p e4 Bedding Thickness B,/24" (600 mm) minimum, not less than 3" (75 mm). If rock foundation, use Bc/12" (300 mm) minimum, not less than 6" (150 mm). Bc/24"(600 mm) minimum, not less than 3" (75 mm). If rock foundation, use Bc/12" (300 mm) minimum, not less than 6" (150 mm). B,/24" (600 mm) minimum, not less than 3" (75 mm). If rock foundation, use B,/12" (300 mm) minimum, not less than 6" (150 mm). No bedding required, except if rock foundation, use Bc/12" (300 mm)minimum, not less than 6" (150 mm). Haunch and Outer Bedding 95% SW 90% SW or 95% ML 85% SW, 90% ML, or 95% CL No compaction required, except if CL, use 85% CL Lower Side 90% SW, 95% ML, 100% CL, or natural soils of equal firmness 85% SW, 90% ML, 95% CL, or natural soils of equal firmness 85% SW, 90% ML 95% CL, or natural soils of equal h e s s NOTES: Compaction and soil symbols 4.e. "95% SW"-refers to SW soil material with minimum standard Proctor compaction of 95%. 1.

See Table 16.4C for equivalent modified Proctor values. 2.

The trench top elevation shall be no lower than 0.1H below finished grade or, for roadways, its top shall be no lower than an elevation of 1 ' (0.3 m) below the bottom of the pavement base material. Only 'Qpe 2 and 3 installations are available for horizontal elliptical, vertical elliptical and arch pipe. 3. Soil in bedding and haunch zones shall be compacted to at least the same compaction as specified for the majority of soil in the backfill zone. 4. The trench width shall be wider than shown if required for adequate space to attain the specified compaction in the haunch and bedding 5. zones. For trench walls that are within 10 degrees of vertical, the compaction or firmness of the soil in the trench walls and lower side zone need 6. not be considered. For trench walls with greater than 10-degree slopes that consist of embankment, the lower side shall be compacted to at least the same 7. compaction as specified for the soil in the backfill zone.

practice or tests that reflect the appropriate design Conditions. 16.4.4.2 Loads 16.4.4.2.1 Earth Loads and Pressure Distribution m e effects of soil-structure interaction shall be taken into account and shall be based on the design earth cover, sidefill compaction, and bedding characteristics of the pipe-soil installations. 16.4.4.2.1.1 Standard Znstallations For the Standard Installations given in Article 16.4.3.1, the earth load, WE,may be determined by multiplying the prism load (weight of the column of earth) over the pipes butside diameter by the soil-structure interaction factor, F,, for the specified installation type. WE = FewBCH (16-') Standard Installations for both embankments and trenches shall be designed for positive projection, embankment loading conditions where F, = V m given, in Figure 16.4Afor each me of Standard Installation. For Standard Installations the earth pressure distribution shall be the Heger pressure distribution shown in Figure 16.4A for each type of Standard Installation.

The unit weight of soil used to calculate earth load shall be the estimated unit weight for the soils specified for the pipe-soil installation and shall not be less than 110I~S/CU ft. 16.4.4.2.1.2 Nonstandard Installations When nonstandard installationsare used, the earth load and pressure distribution shall be determined by an appropriate soil-structure interaction analysis. 16.4.4.2.2 Pipe Fluid Weight The weight of fluid, Wf,in the pipe shall be considered in design based on a fluid weight of 62.4 lbs/ft3, unless otherwise specified.

For Standard Installations, the fluid weight shall be supported by vertical earth pressure that is assumed to have the same distribution over the lower part of the pipe as given in Figure 16.4A for earth load. HIGHWAY BlUDGES 414 16.4.5.1 STANDARD EMBANKMENT INSTALLATIONS Overfill - SW. ML. or CL I ----- r----- Haunch - See Table 17.4A uncompocted bedding mater~aland corn~ac~lan each side, same rcguaements as haunch ~ ~ ~ ~ d ~ fort '*a i ~ ~ / FIGURE 16.4B STANDARD TRENCH INSTALLATIONS Excovalion line as Haunch - See Table 17.48 Lower Side m o t r # ond compoctii .ock rid., some uncomposted bedding r- FoM&tion requianmls or ham& FIGURE 16.4C exc'Ptfor T~ - See Tabla 17.48. 16.4.5.1 DIVISION I-DESIGN WT = W E + WL; WT = total load on the pipe as determined according to Article 16.4.4; WE = earth load on the pipe as determined according to Article 16.4.4.2.2; WL = live load on the pipe as determined according to Article 16.4.4. 16.4.5.1.1 Ultimate D-load The required D-load at which the pipe develops its ultimate strength in a three-edge-bearing test is the design D-load (at 0.01-inch crack) multiplied by a strength factor that is specified in AASHTO materials specifications M 170 or M 242 (ASTM C 76 or C 655) for circular pipe, M 206 (ASTM C 506) for arch pipe and M 207 (ASTM C 507) for elliptical pipe. q = 415 the ratio of the total lateral pressure to the total vertical fill load. 16.4.5.2.3 Live Load Bedding Factor The bedding factors for live load, WL,for both circular pipe and arch and elliptical pipe are given in Table 16.5E If Bfeis less than BnL, use Bfeinstead of BnL for the live load bedding factor. Design values for CA,CN,and x are found in Table 16.4D. The value of q is determined by the following equations: Arch and Horizontal Elliptical Pipe Vertical Elliptical Pipe 16.4.5.2 Bedding Factor The bedding factor, Bf, is the ratio of the supporting strength of buried pipe to the strength of the pipe determined in the three-edge-bearing test. The supporting strength of buried pipe depends on the type of Standard Installation. See Figures 16.4B and 16.4Cfor circular pipe and Figures 16.4D and 16.4E for other arch and elliptical shapes. The Tables 16.4A and 16.4B apply to circular, arch and elliptical shapes. 16.4.5.2.1 Earth Load Bedding Factor for Circular Pipe Earth load bedding factors, Bf,. for circular pipe are presented in Table 16.4E. 16.4.5.2.2 Earth Load Bedding Factor for Arch and Elliptical Pipe The bedding factor for installations of arch and elliptical pipe, Figures 16.4D and 16.4E, is Values for CAand CNare listed in Table 16.4D. CA= a constant corresponding to the shape of the pipe; CN= a parameter which is a function of the distribution of the vertical load and vertical reaction; x = a parameter which is a function of the area of the vertical projection of the pipe over which lateral pressure is effective; where P = projection ratio, ratio of the vertical distance between the outside top of the pipe and the ground or bedding surface to the outside vertical height of the pipe. 16.4.5.2.4 Intermediate Trench Widths For intermediate trench widths, the bedding factor may be estimated by interpolation between the narrow trench and transition width bedding factors. 16.4.6 Direct Design Method for Precast Reinforced Concrete Circular Pipe 16.4.6.1 Application This Specification is intended for use in direct design of precast reinforced concrete circular pipe, and is based on design of pipe wall for effects of loads and pressure distribution for installed conditions.

Standard dimensions are shown in AASHTO M 170. Design wall thicknesses other than the standard wall dimension may be used provided the design complies with all applicable requirements of Section 16. 16.4.6.2 General Design shall conform to applicable sections of these specifications, except as provided otherwise in this article. 416 HIGHWAY BRIDGES Overfill - SW. ML Overfill - 16.4.6.2 SW. ML. - Hounch See Toble 16.48 Lower Side See Toble 16.48 - ee Toble 16.48 " \Middle moteriols and commction eoch side. some requirements as hounch Bedding uncompocted bedding --, -, Foundotion HORIZONTAL ELLIPTCAL PIPE Overfill loosely ploced uncompocted bedding Outer Bedding moterials and cammclion eoch side, some requirements as haunch LEGEND: Bc = Outside Diameter / H = Bockfill cover above lop pipe / LEGEND: Bc = Outside Diometer H = Bockfill cover above top of pipe VERTICAL ELLIPTICAL PIPE - SW. ML. or - &fh able 16.40 &wc"efid. c able 16.40 Spring Bedding -Outer See Table 16.484 Bedding moteriols and compaction eoch side, some requirements or hounch - k d d l e Bedding loosely placed uncompocted bedding /- FoundOtian LEGEND: Bc = Outside Diameter H = Backfill cover above top of pipe ARCH PIPE FIGURE 16.4D Trench Beddings, Miscellaneous Shapes The total load on the pipe shall be determined according to Article 16.4.4 and Table 3.22.1A. The pressure distribution on the pipe from applied loads and bedding reaction shall be determined from a soil-structure analysis or shall be a rational approximation. Acceptable pressure distribution diagrams are the Heger Pressure Distribution (see Figure 16.4A) for use with the Standard Installations: the OlanderModified Olander Radial Pressure Distribution (see Figure 16.4F); or the ParisManual Uniform Pressure Distribution (see Figure 16.4F). For use with the Heger Pressure Distribution, four Types of Standard Embankment Installations, soil types, and compaction requirements are depicted in Figures 16.4B and 16.4E and Tables 16.4Aand 16.4B. Table 16.4C relates to the Standard Installation designated soils to the AASHTO and Unified Soil Classification System categories. For other bedding conditions, see Section 27, Division II--Constmction. Other methods for determining total load and pressure distribution may be used, if based on successful de- 16.4.6.2 DIVISION I-DESIGN 417 See Toble 16.4A See Table 16.4A Lower Side See Toble 16.4A - - Lower Side See Toble 16.4A Bedding - See T - Bc = Outside Diameter Backfill cover above H top of pipe H = Bockfill cover obowe top of pipe HORIZONTAL ELLIPTICAL PIPE Overfill - VERTK=AL ELLIFTICAL PIPE SW. ML, Or CL Bc/6[Min.) .. Bc 7 . . Bc(Min.) - Spring - - Hounch See loble 16.4A Lower Side See Table 16.4A 3I I \Middle Bedding See fable 16.4A outer .&ding I w s e ~ YPlaced uncompacted bedding materiols and Be/3 comoaclion each --'! side. same Foundation LEGEND: requirements Bc 3 Outside Diameter 0% hounch H = Bocktill cover above l o p of pipe Bedding ARCH PlPE FIGURE 16AE Embankment Beddings, Miscellaneous Shapes sign practice or tests that reflect the appropriate design condition. 16.4.6.3 Strength-Reduction Factors Strength-reduction factors for load factor design of plant made reinforced concrete pipe may be taken as 1.0 for flexure and 0.9 for shear and radial tension. For Q p e 1 installations, the strength-reduction factor shall be 0.9 for flexure and 0.82 for shear and radial tension. 16.4.6.4 Process and Material Factors Process and material factors, F, for radial tension and F, for shear strength for load factor design of plant made reinforced concrete pipe are conservatively taken as 1.0. Higher values may be used if substantiated by appropriate test data approved by the Engineer. 16.4.6.5 Orientation Angle When quadrant mats, stirrups and/or elliptical cages are used, the pipe installation requires a specific orienta- 420 HIGHWAY BRIDGES 16.4.6.6.4 FIGURE 16.4F Suggested Design Pressure Distribution Around a Buried Concrete Pipe for Analysis by Direct Design ESSDnlK F u m ff rn5 1)rPIKUTIrn w mw m C C NEGATIVE PROJECTINO Emwwal FIGURE 16.4G Essential Features of Types of Installation 16.4.6.6.4 421 DIVISION I-DESIGN TOP OF EMBANKMENT n II II I I I I I I I I I I I I I I t I I 1 1 I I I I I I I I I I I I I I

provisions for bridges to minimize their susceptibility to damage from earthquakes. The design earthquake motions and forces specified in these provisions are based on a low probability of their being exceeded during the normal life expectancy of a bridge.' Bridges and their components that are designed to resist these forces and that are constructed in accordance with the design details contained in the provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking. The principles used for the development of the provisions are: 1. Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage. 2.
Realistic seismic ground motion intensities and forces are used in the design procedures. 3. Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair. A basic premise in developing these seismic design guidelines was that they are applicable to all parts of the United States. The seismic hazard varies from very small to high across the country.
Therefore, for purposes of design, four Seismic Performance Categories (SPC) are defined on the basis of an AccelerationCoefficient (A) for the site, determined from the map provided, and the Importance Classification (IC).
Different degrees of complexity and sophistication of seismic analysis and design are specified for each of the four Seismic Performance Categories.
An essential bridge must be designed to function during and after an earthquake. In areas with an Acceleration 'The probability of the elastic design force levels not being exceeded in 50 years is the range of 80 to 95%. However, the design earthquake force level by itself does not determine risk; the risk is also affected by the design rules and analysis procedures used in connection with the design ground motion. Coefficient greater than 0.29 essential bridges must meet additional requirements. A bridge is designated essential on the basis of SociaVSurvivaland SecurityJDefense classifications presented in the Commentary.
1.2 BACKGROUND The 1971 San Fernando earthquake was a major turning point in the development of seismic design criteria for bridges in the United States.
Prior to 1971, the American Association of State Highway and Transportation Officials (AASHTO) specifications for the seismic design of bridges were based in part on the lateral forces requirements for buildings developed by the Structural Engineers Association of California. In 1973, the California Department of Transportation (CalTrans) introduced new seismic design criteria for bridges, which included the relationship of the site to active faults, the seismic response of the soils at the site and the dynamic response characteristics of the bridge. In 1975, AASHTO adopted Interim Specifications which were a slightly modified version of the 1973 CalTrans provisions, and made them applicable to all regions of the United States. In addition to these code changes, the 1971 San Fernando earthquake stimulated research activity on seismic problems related to bridges.
In the light of these research findings, the Federal Highway Administration awarded a contract in 1978 to the Applied Technology Council (ATC) to: Evaluate current criteria used for seismic design of highway bridges. Review recent seismic research findings for design applicability and use in new specifications. Develop new and improved seismic design guidelines for highway bridges applicable to all regions of the United States. Evaluate the impact of these guidelines and modify them as appropriate. The guidelines from this ATC project (known as ATC-6) were first adopted by AASHTO as a set of Guide 440 HIGHWAY BRIDGES Specifications in 1983. They were later adopted as seismic provisions within the Standard Specifications in 1990. After damaging earthquakes occurred in California (1989), Costa Rica (1991) and the Philippines (1991), AASHTO requested the Transportation Research Board to review these criteria and prepare revised specifications as appropriate. Funded through the National Cooperative Highway Research Program under NCHRP Project 207/45, the National Center for Earthquake Engineering Research (NCEER) prepared this current set of seismic design provisions. They closely follow the previous criteria but remove ambiguities and technical errors, correct technical omissions and introduce new material which is based in part on recent field experience and partly on new research findings.
In addition, a new format is introduced so as to assist the application of these specifications to bridges in different seismic zones. 1.3 BASIC CONCEPTS The development of these specifications was predicated on the following basic concepts. Hazard to life to be minimized. Bridges may suffer damage but have low probability of collapse due to earthquake motions. Function of essential bridges to be maintained. Ground motions used in design should have low probability of being exceeded during normal lifetime of bridge. Provisions to be applicable to all of the United States. Ingenuity of design not to be restricted. 1.4 PROJECT ORGANIZATION The ATC-6 project was advised by a Project Engineering Panel comprising the following members: Mr. James Cooper, Federal Highway Administration; Mr. Gerard Fox, HNTB, New York; Mr. James H.
Gates, California Department of Transportation; Mr. Veldo Goins, Oklahoma Department of Transportation; Dr. William Hall, University of Illinois, Urbana; Mr. Edward Hourigan, New York Department of Transportation; Mr. Robert Jarvis, Idaho Department of Transportation; Mr. Robert Kealey, Modjeski and Masters, Harrisburg; Mr. James Libby, Libby Engineers, San Diego; Dr. Geoffrey Martin, Fugro Inc., Long Beach; Mr. Joseph Nicoletti, URS Blume, San Francisco; Dr. Joseph Pen- 1.2 zien, University of California, Berkeley; Dr. Walter Podolny, Federal Highway Administration; and Dr. Robert Scanlan, Princeton University, New Jersey. The ATC project manager and technical director were Mr. Roland Sharpe and Dr. Ronald Mayes, respectively. In a similar manner, the NCHRP project was also guided by a Project Panel. The members were: Mr. James D. Cooper, Federal Highway Adrninistration; Mr. James H. Gates, California Department of Transportation; Mr. Veldo Goins, Oklahoma Department of Transportation; Mr. Ayaz Malik, New York Department of Transportation; Mr. Charles Ruth, Washington Department of Transportation; and Mr. Edward Wasserman, Tennessee Department of Transportation. Liaison members were Dr. John Kulicki, Modjeski and Masters (NCHRP 12-33 Liaison) and Dr. Walter Podolny (Federal Highway Administration Liaison). The principal investigator for NCEER was Dr. Ian Buckle; subcontractors included Computech Engineering Services, Berkeley, CA, and Imbsen and Associates, Sacramento, CA. NCHRP Project Officers were Mr. Ian Friedland and Mr. Scott Sabol. The work was conducted in several stages: Review of 1992 Standard Specifications (Division I-A); survey of designer experience with the application of Division I-A and evaluation of design philosophy. Review of bridge performance in recent earthquakes. Review of revised CalTrans seismic design criteria (ATC-32 project). Review of seismic criteria in the proposed LRFD Bridge Specification (NCHRP 12-33). Conduct of certain special studies. Development of draft revisions in various formats of increasing complexity. Evaluation of proposed revisions. Modification and preparation of final standards, as appropriate. 1.5 QUALITY ASSURANCE REQ'UIREMENTS There are numerous instances of structural failures which have occurred during earthquakes that are directly traceable to poor quality control during construction. The literature is replete with reports noting that collapse may have been prevented had proper inspection been exer- 1.5 DIVISION LA--SEISMIC DESIGN cised. To provide adequate seismic quality assurance requirements the engineer specifies the quality assurancerequirements, the contractor exercises the control to achieve the desired quality and the owner monitors the construction process through special inspection. It is essential that each party recognizes its responsibilities,understands the procedures and has the capability to carry them out. Because the contractor does the work and exercises quality control it is essential that the inspection be performed by someone approved by the owner and not the contractor's direct employee. In recognition of the fact that responsibility must be coordinated during construction, the Project Engineering 441 Panel (PEP) for the ATC-6 project examined the responsibility of each party in the current AASHTO (Division I) specifications. This PEP found the quality assurance requirements of the Division I specifications adequate to cover seismic as well as other design requirements.Therefore, no special quality assurance requirements are included in Division I-A. 1.6 FLOW CHARTS Flow charts outlining the steps in the seismic design procedures implicit in these specifications are given in Figures 1.6A and 1.6B. HIGHWAY BRIDGES 442 1.6 APPLICABILITY OF STANDARDS Article 3.1 I I PRELIMINARY DESIGN I I DETERMINE ACCELERATION COEFFICIENT Article 3.2 I, DETERMINE IMPORTANCE CLASSIFICATION Article 3.3 I I Article 3.4 I I DETERMINE SITE COEFFICIENT Article 3.5 I I Article 3.7 YES NO - SEISMIC PERFORMANCE CATEGORY A - DETERMINE DESIGN FORCES Article 5.2 I DETERMINE DESIGN FORCES Article 3.11 SEISMIC PERFORMANCE CATEGORIES B, C AND D SEISMIC PERFORMANCE CATEGORIES 6, C AND D See Figure 2 for Sub Flow Chart DETERMINE DESIGN DISPLACEMENTS Article 5.3 FIGURE 1.6A Design Procedure Flow Chart DETERMINE DESIGN DISPLACEMENTS Article 3.10 1.6 DIVISION IA-SEISMIC DESIGN 443 DETERMINE ANALYSIS PROCEDURE Article 4.2 I II DETERMINE COMPONENT ELASTIC FORCES AND DISPLACEMENTS Article 3.8 I I COMBINE LONGITUDINAL AND TRANSVERSE FORCES Article 3.9 DETERMINE DESIGN FORCES Article 6.2 - SPC B Article 7.2 - SPC C and D I I DETERMINE DESIGN DISPLACEMENTS Article 6.3 - SPC B Article 7.3 - SPC C and D I I DESIGN STRUCTURAL COMPONENTS Section 6 SPC B Section 7 SPC C and D - - I DESIGN FOUNDATIONS Section 6 SPC B Section 7 SPC C and D - - I DESIGN ABUTMENTS Article 6.4.3 SPC B Article 7.4.3 - SPC C Article 7.4.5 - SPC D - 1 YES I SEISMIC DESIGN COMPLETE I FIGURE 1.6B Sub Flow Chart for Seismic Performance Categories B, C, and D Section 2 SYMBOLS AND DEFINITIONS 2.1 NOTATIONS The following symbols and definitions apply to these Specifications: Vertical spacing of transverse reinforcement (hoops or stirrups) in rectangular reinforced concrete columns (in. or mm) = Acceleration coefficient determined in Article 3.2 (dimensionless) = Area of reinforced concrete column core (in.2or mm2) = Gross area of reinforced concrete column (in2or mm2) = Area of longitudinal reinforcement in a concrete pile (in2or mm2) = Total cross-sectional area of transverse reinforcement (hoops or stirrups) used in rectangular reinforced concrete columns (in.2or mm2) and defined by Equations (M), (6-7), (7-6), and (7-7) = Total amount of reinforcement normal to a construction joint or mm2) = Loads resulting from buoyancy forces and used in the group load combinations of Equations (6-I), (6-2), (7-I), and (7-2) = Coefficient used in steel design to account for boundary conditions (dimensionless) = Elastic seismic response coefficient defined in Article 3.6.1 (dimensionless) = Elastic seismic response coefficient for mode "m" defined in Article 3.6.2 (dimensionless) = Diameter of a reinforced concrete column (in. or mm) = Loads resulting from dead load and used in the group load combinations of Equations (6-I), (6-2), (7-I), and (7-2) = Loads resulting from earth pressure and used in the group load combinations of Equations (GI), (6-2), (7-I), and (7-2) = Modified foundation seismic forces used in the group load combination of Equations (6-2) and (7-2), and defined in Articles 6.2.2 and 7.2.1 = Modified seismic forces used in the group load combination of Equations (6-1) and (7-I), and defined in Articles 6.2.1 and 7.2.1 = Specified compressive strength of reinforced concrete (psi or MPa) = Yield strength of reinforcement in reinforced concrete members (psi or MPa) = Yield strength of transverse reinforcement (psi or MPa) = Axial stress in steel design that would be permitted if axial force alone existed (psi or MPa) = Buckling stress for load factor steel design (psi or MPa) = Euler buckling stress in the plane of bending (psi or MPa) = Euler buckling stress for service load steel design (psi or MPa) = Yield strength of structural steel (psi or MPa) = Acceleration of gravity (in./sec2 or crn/sec2) = Core dimension of a rectangular reinforced concrete column (in.
or mm) = Height of a column or pier defined in Articles 5.3,6.3, and 7.3 (ft or m) = Importance Classification given in Article 3.3 (dimensionless) = Total lateral stiffness of bridge as defined in Article 4.3 (lblin. or kN/m) = Effective length factor used in steel design and given in Articles 6.5 and 7.5 (dimensionless) = Seismic coefficient used to calculate lateral earth pressures and defined in Articles 6.4.3 and 7.4.3 (dimensionless) = A A, A, As Ash HIGHWAY BRIDGES 2.1 = Length of bridge deck defined in Articles 4.3,5.3,6.3, and 7.3 (ft or m) = Minimum support length for girders specified in Articles 3.10,5.3,6.3, and 7.3 (in. or mrn) = Intensity of the equivalent static seismic loading applied to represent the primary mode of vibration in Articles 4.3 and 4.4 (forcefunit length) Minimum axial load specified in Article 7.2.3 for columns and 7.2.4 for piers (Ib or N) = Assumed uniform loading used to calculate the period in Articles 4.3 and 4.4 (forcefunit length) = Vertical force at a support due to longitudinal horizontal seismic loads (Ib or N) = Response modification factor specified in Article 3.7 (dimensionless) = Site coefficient specified in Article 3.5.1 (dimensionless) = Angle of skew of girder support as defined in Articles 5.3 and 6.3 (degrees) = Loads resulting from stream flow forces and used in the group load combinations of Equations (6-I), (6-2), (7-I), and (7-2) = Seismic Performance Category specified in Article 3.4 (dimensionless) = Fundamental period of the bridge determined in Articles 4.3 and 4.4 (sec.) = Period of the mth mode of vibration of a bridge (sec.) = Nominal shear strength provided by concrete as specified in Article 7.6.2(C) = Limiting shear force across a construction joint (lb or N) = Shear stress (psi or MPa) = Shear force (lb or N) = Static displacement profiles resulting from applied loads po and pe, respectively, and used in Articles 4.3 and 4.4 (in. or mm) = Dead weight of the bridge superstructure and tributary substructure per unit length (forcefunit length) = Total dead weight of bridge superstructure and tributary substructure (lb or kN) = The ratio of horizontal shear reinforcement area to gross concrete area of a vertical section-Article 7.6.3 (dimensionless) = The ratio of vertical shear reinforcement area to the gross concrete area of a horizontal sectionArticle 7.6.3 (dimensionless) = Volumetric ratio of spiral reinforcement for a circular column (dimensionless) = Strength reduction factor (dimensionless) = Coefficient used to calculate the period of the bridge in Article 4.4 (length2) = Coefficient used to calculate the period of the bridge in Article 4.4 (force . length) = Coefficient used to calculate the period of the bridge in Article 4.4 (force length2) = SPC T Tm vc Vj V" vu vs(x), ve(x) . Section 3 GENERAL REQUIREMENTS 3.1 APPLICABILITY OF SPECIFICATIONS These Specificationsare for the design and construction of new bridges to resist the effect of earthquake motions. The provisions apply to bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 500 feet (150 meters). Suspension bridges, cable-stayed bridges, arch type and movable bridges are not covered by these Specifications. Seismic design is usually not required for buried type (culvert) bridges. The provisions contained in these Specifications are minimum requirements. No detailed seismic analysis is required for any single span bridge or for any bridge in Seismic Performance Category A. For single span bridges (Article 3.11) and bridges classified as SPC A (Section 5) the connections must be designed for specified forces and must also meet minimum support length requirements. 3.2 ACCELERATION COEFFICIENT The Acceleration Coefficient (A) to be used in the application of these provisions shall be determined from the contour maps of Figures 3.2A and 3.2B. (Note: An en- FIGURE 3.2A Acceleration Coefficient-Continental United States (An enlarged version of this map, including counties, is given at the end of Division-I-A.) HIGHWAY BRIDGES 448 3.2 ALASKA HAWAII PUERTO RlCO FIGURE 3.2B Acceleration Coefficient-Alaska, Hawaii, and Puerto Rico 3.2 DIVISION IA-SEISMIC DESIGN larged version of Figure 3.2A is given at the end of Division I-A.) Values given in Figures 3.2A and 3.2B are expressed in percent. Numerical values for the coefficient A are obtained by dividing contour values by 100.0. Local maxima (and minima) are given inside the highest (and lowest) contour for a particular region. Linear interpolation shall be used for sites located between contour lines and between a contour line and local maximum (or minimum). The seismic loads represented by the acceleration coefficients in Figures 3.2A and 3.2B have a 10% probability of exceedance in 50 years (which is approximately equivalent to a 15% probability of exceedance in 75 years). This corresponds to a return period of approximately 475 years. Special studies to determine site- and structure-specific acceleration coefficients shall be performed by a qualified professional if any one of the following conditions exist: (a) The site is located close to an active fault. (b) Long duration earthquakes are expected in the region. (c) The importance of the bridge is such that a longer exposure period (and therefore return period) should be considered. The effect of soil conditions at the site are considered in Article 3.5. 3.3 IMPORTANCE CLASSIFICATION An Importance Classification (IC) shall be assigned for all bridges with an Acceleration Coefficient greater than 0.29 for the purpose of determining the Seismic Performance Category (SPC) in Article 3.4 as follows: 1. Essential bridges - IC = I 2. Other bridges - IC = II Bridges shall be classified on the basis of SociallSurvival and SecurityIDefense requirements, guidelines for which are given in the Commentary. 3.4 SEISMIC PERFORMANCE CATEGORIES Each bridge shall be assigned to one of four Seismic PerformanceCategories (SPC), A through D, based on the Acceleration Coefficient (A) and the Importance Classification (IC), as shown in Table 3.4. Minimum analysis and design requirements are governed by the SPC. 449 TABLE 3.4 Seismic Performance Category (SPC) Acceleration Coefficient A Importance Classification (IC) I I1 SOIL PROFILE TYPE I is a profile with either1. Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 feetlseconds (760 meters/seconds),or by other appropriate means of classification); or 2. Stiff soil conditions where the soil depth is less than 200 feet (60 meters) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays. SOIL PROFILE TYPE II is a profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 200 feet (60 meters) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
SOIL PROFILE TYPE 111 is a profile with soft to medium-stiff clays and sands, characterized by 30 feet (9 meters) or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils. SOIL PROFILE TYPE TV is a profile with soft clays or silts greater than 40 feet (12 meters) in depth. These materials may be characterized by a shear wave velocity less than 500 feetlseconds (150 meters/seconds) and might include loose natural deposits or synthetic, nonengineered fill. In locations where the soil properties are not known in sufficient detail to determine the soil profile type with confidence, the Engineer's judgement shall be used to select a site coefficient from Table 3.5.1 that conservatively represents the amplification effects of the site. The soil profile coefficients apply to all foundation types including pile supported and spread footings. A site coefficient need not be explicitly identified if a site-specific seismic response coefficient is developed by a qualified professional (Article 3.6). 3.5 SITE EFFECTS 3.5.1 Site Coefficient The effects of site conditions on bridge response shall be determined from a Site Coefficient (S) based on soil profile types defined as follows: The Site Coefficient (S) approximatesthe effects of the site conditions on the elastic response coefficient or spectrum of Article 3.6 and is given in Table 3.5.1. 450 HIGHWAY BRIDGES TABLE 3.5.1 Site Coefficient (S) .. Soil Profile T h e S I I1 111 IV 1.0 1.2 1.5 2.0 3.6 ELASTIC SEISMIC RESPONSE COEFFICIENT A seismic response coefficient is specified in this Article which defines the earthquake load to be used in the elastic analysis for seismic effects.
These requirements may be superseded by a 5% damped, site-specific, response spectrum developed by a qualified professional. Such a spectrum shall include the effects of both the local seismology and the site soil conditions. 3.6 The value of C,, need not exceed 2.5A. For Type III or Type I ' soils in areas where the coefficient A r 0.30, C,, need not exceed 2.OA. EXCEPTIONS: 1. For Soil Profile Type III or Type IV soils, C,, for modes other than the fundamental mode which have periods less than 0.3 seconds may be determined in accordance with the following formula: C,, = A(0.8 + 4.0Tm) (3-3) 2. For structures in which any T, exceeds 4.0 seconds, the value of C,, for that mode may be determined in accordance with the following formula: 3.6.1 Elastic Seismic Response Coefficient for Single Mode Analysis The elastic seismic response coefficient C, used to determine the design forces is given by the dimensionless formula: 3.7 RESPONSE MODIFICATION FACTORS S = the dimensionless coefficient for the soil profile characteristics of the site as given in Article 3.5, Seismic design forces for individual members and connections of bridges classified as SPC B, C, or D are determined by dividing the elastic forces by the appropriate Response Modification Factor (R) as specified in Article 6.2 or 7.2. The Response Modification Factors for various bridge components are given in Table 3.7. These factors shall only be used when all of the design requirements of Sections 6 and 7 are satisfied. If these requirements are not satisfied, the maximum value of R for substructures and connections shall be 1.0 and 0.8, respectively. T = the period of the bridge as determined in Articles 4.3 and 4.4 or by other acceptable methods. 3.8 DETERMINATION OF ELASTIC FORCES AND DISPLACEMENTS where, A = the Acceleration Coefficient from Article 3.2, The value of C, need not exceed 2.5A. For Soil Profile Q p e I11 or Type IV soils in areas where A r 0.30, C, need not exceed 2.OA. 3.6.2 Elastic Seismic Response Coefficient for Multimodal Analysis The elastic seismic response coefficient for mode "m," C,,, shall be determined in accordance with the following formula: For bridges classified as SPC B, C, or D the elastic forces and displacements shall be determined independently along two perpendicular axes by use of the analysis procedure specified in Aaicle 4.2. The resulting forces shall then be combined as specified in Article 3.9. Qpically, the perpendicular axes are the longitudinal and transverse axes of the bridge but the choice is open to the designer. The longitudinal axis of a curved bridge may be a chord connecting the two abutments. 3.9 COMBINATION OF ORTHOGONAL SEISMIC FORCES where T, = the period of the mth mode of vibration. A combination of orthogonal seismic forces is used to account for the directional uncertainty of earthquake DIVISION IA-SEISMIC DESIGN TABLE 3.7 Response Modifications Factor (R) 1 Substructure R Connections3 R - Wall- type pie?
Reinforced ,concrete pile bents a. Vertical piles only b. One or more batter piles Single columns Steel or composite steel and concrete pile bents a. Vertical piles only b. One or more batter piles Multiple column bent 2 3 2 3 Superstructure to abutment Expansion joints within a span of the superstructure Columns, piers or pile bents to cap beam or superstructure4 Columns or piers to foundations4 - 0.8 0.8 1.0 1.0 'The R-Factor is to be used for both orthogonal axes of the substructure. 'A wall-type pier may be designed as a wlumn in the weak direction of the pier provided all the provisionsfor columns in Article 6.6 or 7.6, as appropriate, are followed. The R-Factorfor a single wlumn may then be used. 'Connections are those mechanical devices which transfer shear and axial forces from one structural component to another. They generally do not include moment connections and thus comprise bearings and shear keys. The R factors in this Table are applied to the elastic forces in the restrained directions only. 'For bridges classified as SPC C or D,it is recommended that the w ~ e c t i o n sbe designed for the maximum forces capable of being developed by plastic hinging of the column or column bent as specified in Article 7.2.5. These forces will often be significantly less than those obtained using an R-Factor of 1. motions and the simultaneous occurrences of earthquake forces in two perpendicular horizontal directions. The elastic seismic forces and moments resulting from analyses in the two perpendicular directions of Article 3.8 shall be combined to form two load cases as follows: LOAD CASE 1: Seismic forces and moments on each of the principal axes of a member shall be obtained by adding 100% of the absolute value of the member elastic seismic forces and moments resulting from the analysis in one of the perpendicular (longitudinal) directions to 30% of the absolute value of the corresponding member elastic seismic forces and moments resulting from the analysis in the second perpendicular direction (transverse). (NOTE: The absolute values are used because a seismic force can be positive or negative.) LOAD CASE 2: Seismic forces and moments on each of the principal axes of a member shall be obtained by adding 100% of the absolute value of the member elastic seismic forces and moments resulting from the analysis in the second perpendicular direction (transverse) to 30% of the absolute value of the corresponding member elastic seismic forces and moments resulting from the analysis in the first perpendicular direction (longitudinal). EXCEPTION: For SPC C and D when foundation andlor column connection forces are determined from plastic hinging of the columns (Article 7.2.2) the resulting forces need not be combined as specified in this section. If a pier is designed as a column per Article 7.2.4 this exception only applies for the weak direction of the pier when forces resulting from plastic hinging are used. The combination specified must be used for the strong direction of the pier. 3.10 MINIMUM SEAT-WIDTH REQUIREMENTS AU bridges, regardless of Seismic Performance Category (SPC) and number of spans, shall satisfy minimum support length requirements at the expansion ends of all girders. These support lengths are defined in Figure 3.10 as dimension N. The minimum value for N is given for SPC A in Article 5.3; for SPC B in Article 6.3; and for SPC C and D in Article 7.3. 3.11 DESIGN REQUIREMENTS FOR SINGLE SPAN BRIDGES The detailed analysis and design requirements of Sections 4,5,6, and 7 are not required for single span bridges. In lieu of rigorous analysis, the connections between the bridge span and the abutments shall be designed to resist the tributary weight at the abutment multiplied by the Acceleration Coefficient and the Site Coefficient for the site. This force must be considered to act in each horizontally restrained direction. The minimum support lengths shall be as specified in Article 3.10. HIGHWAY BRIDGES 452 ABUTMENT 3.12 COLUMN OR PIER HINGE WITHIN A SPAN *EXPANSION JOINT OR END OF BRIDGE DECK FIGURE 3.10 Dimensions for Minimum Support Length Requirements 3.12 REQUIREMENTS FOR TEMPORARY BRIDGES AND STAGED CONSTRUCTION The requirement that an earthquake shall not cause collapse of all or part of a bridge as stated in Article 1.1, applies to temporary bridges which are expected to carry traffic andor pass over routes that carry traffic.
It also applies to those bridges that are constructed in stages and expected to carry tr c andor pass over routes that carry traffic. However, in view of the limited exposure period, the Acceleration Coefficient given in Article 3.2 may be reduced by a factor of not more than 2 in order to calculate the component elastic forces and displacements. Note that Acceleration Coefficients for construction sites that are close to active faults shall be the subject of special study. Further, the Response Modification Factors given in Article 3.7 may be increased by a factor of not more than 1.5 in order to calculate the design forces. This factor shall not be applied to connections as defined in Table 3.7. The minimum seat-width provisions of Article 3.10 shall apply to all temporary bridges and staged construction. Any bridge or partially constructed bridge that is expected to be temporary for more than 5 years shall be designed using the requirements for permanent structures and shall not use the provisions of this article. Section 4 ANALYSIS REQUIREMENTS 4.1 GENERAL is considered to be "not regular." A more rigorous, generally accepted analysis procedure may be used in lieu of the recommended minimum such as the Time History Method (Procedure 4). Curved bridges comprised of multiple simple spans shall be considered to be "not regular" bridges if the subtended angle in plan is greater than 20"; such bridges shall be analyzed by either Procedure 3 or 4. The requirements of this section shall control the selection and method of seismic analysis of bridges. Four analysis procedures are presented. Procedure 1. Uniform Load Method Procedure 2. Single-Mode Spectral Method Procedure 3. Multimode Spectral Method Procedure 4. Time History Method 4.2.1 Special Requirements for Single-Span Bridges and Bridges in SPC A In each method, all fixed column, pier, or abutment supports are assumed to have the same ground motion at the same instant in time. At movable supports, displacements determined from the analysis prescribed in this chapter, which exceed the minimum seat width requirements as specified in Article 6.3 or 7.3, shall be used in design without reduction by the Response Modification Factor (Article 3.7). Notwithstanding the above requirements,detailed seismic analysis is not required for a single-span bridge or for bridges classified as SPC A. 4.2.2 Special Requirements for Curved Bridges A curved continuous-girder bridge may be analyzed as if it were straight provided all.of the following requirements are satisfied: 4.2 SELECTION OF ANALYSIS METHOD (a) the bridge is regular as defined in Table 4.2B except that for a two-span bridge the maximum span length ratio from span-to-span must not exceed 2; (b) the subtended angle in plan is not greater than 30"; and Minimum requirements for the selection of an analysis method for a particular bridge type are given in Table 4.2A. Applicability is determined by the "regularity" of a bridge which is a function of the number of spans and the distribution of weight and stiffness. Regular bridges have less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry and no large changes in these parameters from span-to-span or support-to-support (abutments excluded). They are defined in Table 4.2B. Any bridge not satisfying the requirements of Table 4.2B TABLE 4.2B Regular Bridge Requirements Parameter Maximum subtended 90" 90" 90" 90" 90" angle (curved bridge) 3 2 2 1.5 1.5 Maximum span length ratio from span-to-span - 4 4 3 2 Maximum bentlpier stiffness ratio from span-to-span (excluding abutments) TABLE 4.2A Minimum Analysis Requirements Seismic Performance Category Regular Bridges with 2 Through 6 Spans Not Regular Bridges with 2 or More Spans A ByCyD Not required Use Procedure 1 or 2 Not required Use Procedure 3 Value Note: All ratios expressed in terms of the smaller value. 453 454 HIGHWAY BRIDGES (c) the span lengths of the equivalent straight bridge are equal to the arc lengths of the curved bridge. If these requirements are not satisfied, then curved continuous-girder bridges must be analyzed using the actual curved geometry.
4.2.3 Special Requirements for Critical Bridges More rigorous methods of analysis are required for certain classes of important bridges which are considered to be critical structures (e.g., those that are major structures in size and cost or perform a critical function), andlor for those that are geometrically complex and close to active earthquake faults. Time history methods of analysis are recommended for this purpose, provided care is taken with both the modeling of the structure and the selection of the input time histories of ground acceleration.
Time history methods of analysis are described in Article 4.6. 4.2.2 VS,MAX = maximum value of v,(x) and w(x) = weight per unit length of the dead load of the bridge superstructure and tributary substructure The weight should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings. Other loads such as live loads may be included.
(Generally, the inertia effects of live loads are not included in the analysis; however, the probability of a large live load being on the bridge during an earthquake should be considered when designing bridges with high live-to-dead load ratios which are located in metropolitan areas where traffic congestion is likely to occur.) Step 3. Calculate the period of the bridge, T, using the expression: 4.3 UNIFORM LOAD METHODPROCEDURE 1 The uniform load method, described in the following steps, may be used for both transverse and longitudinal earthquake motions. It is essentially an equivalent static method of analysis which uses a uniform lateral load to approximate the effect of seismic loads. The method is suitable for regular bridges that respond principally in their fundamental mode of vibration. Whereas all displacements and most member forces are calculated with good accuracy, the method is known to overestimate the transverse shears at the abutments by up to 100%. If such conservatism is undesirable then the single mode spectral analysis method (Procedure 2) is recommended. Step I . Calculate the static displacements v,(x) due to an assumed uniform load p, as shown in Figure 4.4A an@ Figure 4.4B. The uniform loading p, is applied over the length of the bridge; it has units of forcelunit length and may be arbitrarily set equal to 1.0. The static displacement v,(x) has units of length. Step 2. Calculate the bridge lateral stiffness, K, and total weight, W, from the following expressions: where g = acceleration of gravity (lengtWtime2) Step 4. Calculate the equivalent static earthquake loading p, from the expression: where C, = the dimensionless elastic seismic response coefficient given by Equation (3-1) p, = equivalent uniform static seismic loading per unit length of bridge applied to represent the primary mode of vibration. Step 5. Calculate the displacements and member forces for use in design either by applying p, to the structure and performing a second static analysis or by scaling the results of Step 1 by the ratio pJp0. 4.4 SINGLE MODE SPECTRAL ANALYSIS METHOD-PROCEDURE 2 The single mode spectral analysis method described in the following steps may be used for both transverse and longitudinal earthquake motions. Examples illustrating its application are given in the Commentary. where L = total length of the bridge Step I . Calculate the static displacements v,(x) due to an assumed uniform loading p, as shown in Figure 4.4A. 4.2.2 DIVISION IA-SEISMIC DESIGN 455 (a) Plan Transverse Loading (b) Elevation Longitudinal Loading FIGURE 4.4A Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading The uniform loading p, is applied over the length of the bridge; it has units of forcelunit length and is arbitrarily set equal to 1. The static displacement v,(x) has units of length. Step 2. Calculate factors a,P, and y: FIGURE 4.4B Bridge Deck Subjected to Equivalent Transverse and Longitudinal Seismic Loading where, C, = the dimensionless elastic seismic response coefficient given by Equation (3-I), pe(x) = the intensity of the equivalent static seismic loading applied to represent the primary mode of vibration (forcelunit length). Step 5. Apply loading pe(x) to the structure as shown in Figure 4.4B and determine the resulting member forces and displacements for design. where w(x) is the weight of the dead load of the bridge superstructure and tributary substructure (forcelunit length). The computed factors, a, P, y, have units of (length2), (force X length), and (force X length2), respectively. The weight should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings.
Other loads such as live loads may be included. (Generally, the inertia effects of live loads are not included in the analysis; however, the probability of a large live load being on the bridge during an earthquake should be considered when designing bridges with high live-to-dead load ratios which are located in metropolitan areas where traffic congestion is likely to occur.) Step 3. Calculate the period of the bridge, T, using the expression: where g = acceleration of gravity (length/time2). Step 4. Calculate the equivalent static earthquake loading pe(x)from the expression: pe (4 = PC Y w(x)vs(x) 4.5 MULTIMODE SPECTRAL ANALYSIS METHOD-PROCEDURE 3 The multimode response spectrum analysis should be performed with a suitable space frame linear dynamic analysis computer program. 4.5.1 General The multimode spectral analysis method applies to bridges with irregular geometry which induces coupling in the three coordinate directions within each mode of vibration. These coupling effects make it difficult to categorize the modes into simple longitudinal or transverse modes of vibration and, in addition, several modes of vibration will in general contribute to the total response of the structure. A computer program with space frame dynamic analysis capabilities should be used to determine coupling effects and multimodal contributions to the final response. Motions applied at the supports in any one of the two horizontal directions will produce forces along both principal axes of the individual members because of the coupling effects. For curved structures, the longitudinal motion shall be directed along a chord connecting the abutments and the transverse motion shall be applied normal to the chord. Forces due to longitudinal and transverse motions shall be combined as specified in Article 3.9. 456 HIGHWAY BRIDGES 4.5.2 Mathematical Model The bridge should be modeled as a three-dimensional space frame with joints and nodes selected to realistically model the stiffness and inertia effects of the structure.
Each joint or node should have six degrees of freedom, three translational and three rotational. The structural mass should be lumped with a minimum of three translational inertia terms. The mass should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings. Other loads such as live loads may be included. (Generally, the inertia effects of live loads are not included in the analysis; however, the probability of a large live load being on the bridge during an earthquake should be considered when designing bridges with high live-to-dead load ratios which are located in metropolitan areas where traffic congestion is likely to occur.) 4.5.2(A) Superstructure The superstructure should, as a minimum, be modeled as a series of space frame members with nodes at such points as the span quarter points in addition to joints at the ends of each span. Discontinuities should be included in the superstructure at the expansion joints and abutments. Care should be taken to distribute properly the lumped mass inertia effects at these locations. The effect of earthquake restrainers at expansion joints may be approximated by superimposing one or more linearly elastic members having the stiffness properties of the engaged restrainer units. 4.5.2(B) Substructure The intermediatecolumns or piers should also be modeled as space frame members.
Generally, for short, stiff columns having lengths less than one-third of either of the adjacent span lengths, intermediate nodes are not necessary. Long, flexible columns should be modeled with intermediate nodes at the third points in addition to the joints at the ends of the columns. The model should consider the eccentricity of the columns with respect to the superstructure. Foundation conditions at the base of the columns and at the abutments may be modeled using . equivalent linear spring coefficients. 4.5.3 Mode Shapes and Periods The required periods and mode shapes of the bridge in the direction under consideration shall be calculated by 4.2.3 established methods for the fixed base condition using the mass and elastic stiffness of the entire seismic resisting system. 4.5.4 Multimode Spectral Analysis The response should, as a minimum, include the effects of a number of modes equivalent to three times the number of spans up to a maximum of 25 modes. 4.5.5 Combination of Mode Forces and Displacements The member forces and displacements can be estimated by combining the respective response quantities (e.g., force, displacement, or relative displacement) from the individual modes by the Complete Quadratic Combination (CQC) method. The member forces and displacements obtained using the CQC method of combining modes is generally adequate for most bridge systems. 4.6 TIME HISTORY METHOD-PROCEDURE 4 Any step-by-step, time history method of dynamic analysis, that has been validated by experiment and/ or comparative performance with similar methods, may be used provided the following requirements are also satisfied: (a) The time histories of input acceleration used to describe the earthquakeloads shall be selected in consultation with the Owner or Owner's representative. Unless otherwise directed, five spectrum-compatible time histories shall be used when site-specifictime histories are not available. The spectrum used to generate these five time histories shall preferably be a site-specific spectrum. In the absence of such a spectrum, the response coefficient given by Equation (3-I), for the appropriate soil type, may be used to generate a spectrum. (b) The sensitivity of the numerical solution to the size of the time step used for the analysis shall be determined. A sensitivity study shall also be carried out to investigate the effects of variations in assumed material properties. (c) If an in-elastic time history method of analysis is used, the R-factors permitted by Article 3.7 shall be taken as 1.0 for all substructures and connections. Section 5 DESIGN REQUIREMENTS FOR BRIDGES IN SEISMIC PERFORMANCE CATEGORY A 5.1 GENERAL N= (203 + 1.67L + 6.66H) Bridges classified as SPC A in accordance with Table 3.4 of Article 3.4 shall conform to all the requirements of this Section. (1 + 0.000125S2)(nun) (5-1B) where, 5.2 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORY A Equation (5-1A) or meters for Equation (5-lB), of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck. For hinges within a span, L shall be the sum of L1and L,the distances to either side of the hinge. For single span bridges L equals the length of the bridge deck. These lengths are shown in Figwe 3.10. S = angle of skew of support in degrees, measured from a line normal to the span.
L If a mechanical device is used to connect the superstructure to the substructureit shall be designed to resist a hor

appropriate R-Factor. Each component of the structure shall be designed to withstand the forces resultingfrom each load combination according to Division I, and the additional requirements of this chapter. Note that Equation (7-1) shall be used in lieu of the Division I, Group W group loading combination and that the y and factors equal 1. For Service Load Design, a 50% increase is permitted in the allowable stresses for structural steel and a 33Y3% increase for reinforced concrete. Modijied Design Forces for Structural Members and Connections Seismic design forces specified in this Article shall apply to: (a) The superstructure, its expansion joints and the connections between the superstructure and the supporting substructure. (b) The supporting substructure down to the base of the columns and piers but not including the footing, pile cap, or piles.
(c) Components connecting the superstructure to the abutment. 7.2.1 (B) Modijied Design Forces for Foundations Seismic design forces for foundations, including footings, pile caps, and piles shall be the elastic seismic forces obtained from Load Case 1 and Load Case 2 of Article 3.9 divided by the Response Modification Factor (R) specified below. These modified seismic forces shall then be combined independently with forces from other loads as specified in the following group loading combination to determine two alternate load combinations for the foundations. Seismic design forces for the above components shall be determined by dividing the elastic seismic forces obtained from Load Case 1 and Load Case 2 of Article 3.9 by the appropriate Response Modification Factor of Arti465 466 Group Load = 1.0@ + B HIGHWAY BRIDGES + SF + E + EQF) (7-2) 7.2.1(B) The forces corresponding to a single column hinging are: where D, B, E, and SF are as defined in Article 7.2.1 and EQF = the elastic seismic force for either Load Case 1 or Load Case 2 of Article 3.9 divided by an RFactor equal to 1.O. Each component of the foundation shall be designed to resist the forces resulting from each load combination according to the requirements of Division I and to the additional requirements of Article 7.2.6. 7.2.2 Forces Resulting from Plastic Hinging in the Columns, Piers, or Bents The force resulting from plastic hinging at the top andor bottom of the column shall be calculated after the preliminary design of the columns is complete. The forces resulting from plastic hinging are recommended for determining design forces for most components as specified in Articles 7.2.3 through 7.2.6. Alternate conservative design forces are specified if forces resulting from plastic hinging are not calculated. The procedures for calculating these forces for single column and pier supports and bents with two or more columns are given in the following subsections. 7.2.2(A) Single Columns and Piers The forces shall be calculated for the two principal axes of a column and in the weak direction of a pier or bent as follows: Step I . Determine the column overstrength plastic moment capacities. For reinforced concrete columns, use a strength reduction factor (4) of 1.3 and for structural steel columns use 1.25 times the nominal yield strength. (Note: This corresponds to the normal use of a strength reduction factor for reinforced concrete. In this case it provides an increase in the ultimate strength.) For both materials use the maximum elastic column axial load from Article 3.9 added to the column dead load. Step 2. Using the column overstrength plastic moments, calculate the corresponding column shear force. For flared columns this calculation shall be performed using the overstrength plastic moments at both the top and bottom of the flare with the appropriate column height. If the foundation of a column is significantly below ground level, consideration should be given to the possibility of the plastic hinge forming above the foundation. If this can occur the column length between plastic hinges shall be used to calculate the column shear force. (a) Axial Forces-unreduced maximum and minimum seismic axial load of Article 3.9 plus the dead load. (b) Moments-as calculated in Step 1. (c) Shear Force-as calculated in Step 2. 7.2.2(B) Bents with Two or More Columns The forces for bents with two or more columns shall be calculated both in the plane of the bent and perpendicular to the plane of the bent. Perpendicular to the plane of the bent the forces shall be calculated as for single columns in accordance with Article 7.2.2(A). In the plane of the bent the forces shall be calculated as follows: Step I . Determine the column overstrength plastic moment capacities. For reinforced concrete use a strength reduction factor (+) of 1.3 and for structural steel use 1.25 times the nominal yield strength. (Note: This corresponds to the normal use of a strength reduction factor for reinforced concrete. In this case it provides an increase in the ultimate strength.) For both materials use the axial load corresponding to the dead load. Step 2. Using the column overstrength plastic moments calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the bent.
Note that, if a partial-height wall exists between the columns, the effective column height is taken from the top of the wall. For flared columns and foundations below ground level, see Article 7.2.2(A) Step 2. For pile bents the length of pile above the mud line shall be used to calculate the shear force. Step 3. Apply the bent shear force to the top of the bent (center of mass of the superstructure above the bent) and determine the axial forces in the columns due to overturning when the column overstrength plastic moments are developed. Step 4. Using these column axial forces combined with the dead load axial forces, determine revised column overstrength plastic moments. With the revised overstrength plastic moments calculate the column shear forces and the maximum shear force for the bent.
If the maximum shear force for the bent is not within 10% of the value previously determined, use this maximum bent shear force and return to Step 3. The forces in the individual columns in the plane of a bent corresponding to column hinging, are: 7.2.2(B) DIVISION IA-S EISMIC DESIGN (a) Axial Forces-the maximum and minimum axial load is the dead load plus, or minus, the axial load determined from the final iteration of Step 3. (b) Moments-the column overstrength plastic moments corresponding to the maximum con~pressive axial load specified in (a) above, with a strength reduction factor of 1.3 for reinforced concrete and 1.25 times the nominal yield strength for structural steel. (c) Shear Force-the shear force corresponding to the column overstrength moments in (b) above, noting the provisions in Step 2 above. 7.2.3 Column and Pile Bent Design Forces Design forces for columns and pile bents shall be the following: (a) Axial Forces-the minimum and maximum design force shall either be the elastic design values determined in Article 3.9 added to the dead load, or the values corresponding to plastic hinging of the column and determined in Article 7.2.2. Generally, the values corresponding to column hinging will be smaller. (b) Moments-the modified design moments determined in Article 7.2.1. (c) Shear Force-either the elastic design value determined from Article 7.2.1 using an R-Factor of 1 for the column or the value corresponding to plastic hinging of the column as determined in Article 7.2.2. Generally, the value corresponding to column hinging will be significantly smaller. 7.2.4 Pier Design Forces The design forces shall be those determined in Article 7.2.1 except if the pier is designed as a column in its weak direction. If the pier is designed as a column the design forces in the weak direction shall be as specified in Article 7.2.3 and all the design requirements for columns of Article 7.6 shall apply. (Note: When the forces due to plastic hinging are used in the weak direction the combination of forces specified in Article 3.9 is not applicable.) 7.2.5 Connection Design Forces The design forces shall be those determined in Article 7.2.1 except that for superstructure connections to columns and column connections to cap beams or footings, the alternate forces specified in 7.2.5(C) below are recommended. Additional design forces at connections are as follows: 467 7.2.5(A) Longitudinal Linkage Forces Positive horizontal linkage shall be provided between adjacent sections of the superstructure at supports and expansion joints within a span.
The linkage shall be designed for a minimum force of the Acceleration Coefficient times the weight of the lighter of the two adjoining spans or parts of the structure. If the linkage is at a point where relative displacement of the sections of superstructure is designed to occur during seismic motions, sufficient slack must be allowed in the linkage so that the linkage force does not start to act until the design displacement is exceeded. Where linkage is to be provided at columns or piers, the linkage of each span may be attached to the column or pier rather than between adjacent spans. Positive linkage shall be provided by ties, cables, dampers, or an equivalent mechanism. Friction shall not be considered a positive linkage.
7.2.5(B) Hold-Down Devices Hold-down devices shall be provided at all supports or hinges in continuous structures, where the vextical seismic force due to the longitudinal horizontal seismic load opposes and exceeds 50% but is less than 100% of the dead load reaction. In this case, the minimum net upward force for the hold-down device shall be 10% of the dead load downward force that would be exerted if the span were simply supported. If the vertical seismic force (Q) due to the longitudinal horizontal seismic load opposes and exceeds 100 percent of the dead load reaction (DR), the net upwards force for the holddown device shall be 1.2(Q - DR) but it shall not be less than that specified in the previous paragraph. 7.2.5(C) Column and Pier Connections to Cap Beams and Footings The recommended connection design forces between the superstructure and columns, columns and cap beams, and columns and spread footings or pile caps are the forces developed at the top and bottom of the columns due to column hinging and determined in Article 7.2.2. The smaller of these or the values specified in Article 7.2.1 may be used. Note that these forces should be calculated after the column design is complete and the overstrength moment capacities have been obtained. 7.2.6 Foundation Design Forces The design forces for foundations including footings, pile caps, and piles may be either those forces determined in Article 7.2.1(B) or the forces at the bottom of the columns corresponding to column plastic hinging as 468 HIGHWAY BRIDGES determined in Article 7.2.2. Generally, the values corresponding to column hinging will be significantly smaller.
When the columns of a bent have a common footing the final force distribution at the base of the columns from Step 4 of Article 7.2.2(B) may be used for the design of the footing in the plane of the bent. This force distribution produces lower shear forces and moments on the footing because one exterior column may be in tension and the other in compression due to the seismic overturning moment. This effectively increases the ultimate moments and shear forces on one column and reduces them on the other. 7.2.7 Abutment and Retaining Wall Design Forces The components connecting the superstructure to an abutment (e.g., bearings and shear keys) shall be designed to resist the forces specified in Article 7.2.1. Design requirements for abutments are given in Article 7.4.3 for SPC C and Article 7.4.5 for SPC D. 7 3 DESIGN DISPLACEMENT FOR SEISMIC PERFORMANCE CATEGORIES C AND D The seismic design displacements shall be the maximum of those determined in accordance with Article 3.8 or those specified in Article 7.3.1. 7.3.1 Minimum Support Length Requirements for Seismic Performance Categories C and D Bridges classified as SPC C or D shall meet the following requirement: Bearing seats supporting the expansion ends of girders, as shown in Figure 3.10, shall be designed to provide a minimum support length N (in. or mm), measured normal to the face of an abutment or pier, not less than that specified below. N = (1 2 + 0.03L + 0.1 2H) (1 0.000125S3 (in.) + (7-3A) or, N = (305 + 2.5L + 10H) (1 0.000125S2)(mm) + 7.2.6 S = angle of skew of support in degrees measured from a line normal to the span. and H is given by one of the following: for abutments, H is the average height, in feet for Equation (7-3A) or meters for Equation (7-3B), of columns supporting the bridge deck to the next expansion joint. H = 0 for single span bridges. for columns andlor piers, H is the column or pier height in feet for Equation (7-3A) or meters for Equation (7-3B). for hinges within a span, H is the average height of the adjacent two columns or piers in feet for Equation (7-3A) or meters for Equation (7-3B). Positive horizontal linkages shall be provided at all superstructure expansion joints, including those joints within a span, as specified in Article 7.2.5. Relative displacements between different segments of the bridge should be carefully considered in the evaluation of the results determined in accordance with Article 3.8. Relative displacements arise from effects that are not easily included in the analysis procedure but should be considered in determining the design displacements.They include the following: (a) Torsional displacements of bridge decks on skewed supports. (b) Rotation andlor lateral displacements of the foundations. (c) Out-of-phase displacements of different segments of the bridge.
This is especially important in determining seat widths at expansion joints. (d) Out-of-phase rotation of abutments and columns induced by traveling seismic waves. 7.4 FOUNDATIONAND ABUTMENT DESIGN REQUIREMCNTS FOR SEISMIC PERFORMANCE CATEGORIES C AND D 7.4.1 General (7-3B) where, L = length, in feet for Equation (7-3A) or meters for Equation (7-3B), of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck. For hinges within a span, L shall be the sum of L, and L2, the distances to either side of the hinge. For single span bridges L equals the length of the bridge deck. These lengths are shown in Figure 3.10. This section includes only those foundation and abutment requirements that are specifically related to seismic resistant construction in SPC C and D. It assumes compliance with all requirements that are necessary to provide support for vertical and lateral loads other than those due to earthquake motions. These include, but are not limited to, provisions for the extent of foundation investigation, fills, slope stability, bearing and lateral soil pressures, drainage, settlement control, and pile requirements and capacities. 7.4.1 DIVISION IA-SEISMIC DESIGN Foundation and abutment seismic design requirements for SPC C are given in Articles 7.4.2 and 7.4.3. Requirements for bridges in SPC D are given in Articles 7.4.4 and 7.4.5. 7.4.2 Foundation Requirements for Seismic Performance Category C Foundation and abutment seismic design requirements for SPC C are given in the following subsections. 7.4.2(A) Investigation In addition to the normal site investigation report, the Engineer may require the submission of a report which describes the results of an investigation to determine potential hazards and seismic design requirementsrelated to (1) slope instability, (2) liquefaction, (3) fill settlement, and (4) increases in lateral earth pressure, all as a result of earthquake motions. Seismicallyinduced slope instability in approach fills or cuts may displace abutments and lead to significant differential settlement and structural darnage. Fill settlement and abutment displacements due to lateral pressure increases may lead to bridge access problems and structural damage. Liquefaction of saturated cohesionless fills or foundation soils may contribute to slope and abutment instability, and could lead to a loss of foundation bearing capacity and lateral pile support. Liquefaction failures of the above type have led to bridge failures during past earthquakes. Further, the above report should include a determination of the potential for surface rupture due to faulting or differential ground displacement (lurching), as a result of earthquake motions. 7.4.2(B) Foundation Design The design forces for the foundations shall be those specified in Article 7.2.6. The soil strength capable of being mobilized by the foundations shall be established in the site investigation report. Because of the dynamic cyclic nature of seismic loading, the ultimate capacity of the foundation supporting medium should be used in conjunction with these load combinations. Due consideration shall be given to the magnitude of the seismically induced foundation settlement that the bridge can withstand. Transient foundation uplift or rocking involving separation from the subsoil of up to one-half of an end bearing foundation pile group or up to one-half of the contact area of foundation footings is permitted under seismic loading, provided that foundation soils are not susceptible to loss of strength under the imposed cyclic loading. 469 For saturated sand and soft clay foundation soils, due consideration shall be given to the potential for soil strength loss under the imposed cyclic loading in assessing the ultimate capacity of foundations. General comments on soil strength and stiffness mobilized during earthquakes, foundation uplift, lateral loading of piles, soil-structure interaction and foundation design in environments susceptible to liquefaction are provided in the Commentary. 7.4.2(C) Special Pile Requirements The following special pile requirementsare in addition to the requirements for piles in other applicable specifications. Piles may be used to resist both axial and lateral loads. The minimum depth of embedment, together with the axial and lateral pile capacities, required to resist seismic loads shall be determined by means of the design criteria established in the site investigation report. Note that the ultimate capacity of the piles should be used in designing for seismic loads.
All piles shall be adequately anchored to the pile footing or cap. Concrete piles shall be anchored by embedment of sufficient length of pile reinforcement (unless special anchorage is provided) to develop uplift forces but in no case shall this length be less than the development length required for the reinforcement. Each concretefilled pipe pile shall be anchored by at least four reinforcing steel dowels with a minimum steel ratio of 0.01 embedded sufficiently as required for concrete piles. Timber and steel piles, including unfilled pipe piles, shall be provided with anchoring devices to develop all uplift forces adequately but in no case shall these forces be less than 10% of the allowable pile load. All concrete piles shall be reinforced to resist the design moments, shears, and axial loads. The following special requirements for concrete piles shall apply: 1. Anchorage.
The longitudinal reinforcement of all concrete piles shall be anchored to the pile footing or cap to develop a force of at least 1 .25A,fy where A, is the area of longitudinal reinforcement in the concrete pile and fyis its nominal yield strength. 2. Confinement Length. The upper end of every pile shall be reinforced as a potential plastic hinge region, except where it can be established that there is no possibility of any significant lateral deflections in the pile resulting from deformation. The potential plastic hinge region shall, as a minimum, be considered to extend from the underside of the pile cap over a length of not less than two pile diameters or 24 inches (610 mil- 470 HIGHWAY BRIDGES limeters). If an analysis of the bridge and pile system indicates that a plastic hinge can form at a lower level, the transverse reinforcement requirements of (3) shall extend to that level. Note the special requirements for pile bents given in Article 7.6.2(C), (D), and (E). 3. Volumetric Ratio for Confinement. The volumetric ratio of transverse reinforcement to the distance specified in (2) shall be as required for columns in Article 7.6.2(D). 4. Cast-in-Place Concrete Piles. Longitudinal steel shall be provided for cast-in-place concrete piles for the full length of the pile. The upper two-thirds of the pile shall have a minimum longitudinal steel ratio of 0.0075 provided by at least four bars. Spiral reinforcement or equivalent ties of Y4 inch (6 millimeters) diameter or larger shall be provided at 9 inches (225 millimeters) maximum pitch, except for the top 4 feet (1.2 meters) where the pitch shall be 3 inches (75 millimeters maximum, and where the volumetric ratio shall conform to Article 7.6.2(D). 5. Precast Concrete Piles. Longitudinal reinforcing steel shall be provided for each precast concrete pile with a minimum steel ratio of 0.01 provided by at least four bars. Spiral reinforcement ties in precast, including prestressed, concrete piles shall be No. 3 bars or larger and shall be provided at 9 inches (225 millimeters) maximum pitch except for the top 4 feet (1.2 meters) where the pitch shall be 3 inches (75 millimeters) and the volumetric ratio shall conform to 7.6.2(D). 6. Precast-PrestressedPiles. Ties in precast-prestressed piles shall conform to the requirements of precast piles. 7.4.3 Abutment Requirements for Seismic Performance Category C In addition to the provisions outlined in this section, consideration should be given to the mechanism of transfer of superstructure transverse inertial forces to the bridge abutments. Adequate resistance to lateral pressure should be provided by wing walls or abutment keys to minimize lateral abutment displacements. 7.4.3(A) Free-Standing Abutments For free-standing abutments or retaining walls which may displace horizontally without significant restraint (e.g., superstructure supported by sliding bearings), the pseudo-static Mononobe-Okabe method of analysis is recommended for computing lateral active soil pressures during seismic loading. A seismic coefficient equal to one-half the acceleration coefficient (kh= 0.5A) is recommended.
The effects of vertical acceleration may be omitted. Abutments should be proportioned to slide rather 7.4.2(C) than tilt, and provisions should be made to accommodate small horizontal seismically induced abutment displacements when minimal damage is desired at abutment supports. Abutment displacements of up to 1OAinches(250A millimeters) may be expected. The seismic design of free-standing abutments should take into account forces arising from seismically induced lateral earth pressures, additional forces arising from wall inertia effects and the transfer of seismic forces from the bridge deck through bearing supports which do not slide freely (e.g., elastomeric bearings). For free-standing abutments which are restrained from horizontal displacement by anchors or batter piles, the magnitudes of seismically induced lateral earth pressures are higher than those given by the Mononobe-Okabe method of analysis. As a first approximation, it is recommended that the maximum lateral earth pressure be computed by using a seismic coefficient kh = 1.5A in conjunction with the Mononobe-Okabe analysis method. 7.4.3(B) Monolithic Abutments For monolithic abutments where the abutment forms an integral part of the bridge superstructure, maximum earth pressures acting on the abutment may be assumed equal to the maximum longitudinal earthquake force transferred from the superstructure to the abutment. To minimize abutment damage, the abutment should be designed to resist the passive pressure capable of being mobilized by the abutment backfill, which should be greater than the maximum estimated longitudinal earthquake force transferred to the abutment. It may be assumed that the lateral active earth pressure during seismic loading is less than the superstructure earthquake load. When longitudinal seismic forces are also resisted by piers or columns, it is necessary to estimate abutment stiffness in the longitudinal direction in order to compute the proportion of earthquake load transferred to the abutment. 7.4.4 Additional Requirements for Foundations for Seismic Performance Category D Foundation design requirements for bridges classified as SPC D shall meet the requirements of Article 7.4.2 plus the additional requirements of this section. 7.4.4(A) Investigation The Engineer may require the submission of a written report which includes, in addition to the requirements of Article 7.4.2, a site-specific study to investigate the influence of cyclic loading on the deformation and strength characteristics of foundation soils. Potential progressive degradation in the stiffness and strength characteristics of 7.4.4(A) DMSION IA-SEISMIC DESIGN saturated sands and soft clays should be given particular attention. More detailed analyses of slope and/or abutment settlement during earthquake loading should be undertaken. 7.4.4(B) Foundation Design The design forces for foundations shall be those specified in Article 7.2.6. 7.4.5 Additional Requirements for Abutments for Seismic Performance Category D In addition to the requirementsoutlined in Article 7.4.3 consideration should be given to the mechanism of transfer of superstructure longitudinal and transverse inertia forces to the abutments, and also to abutment-soil interaction.
To minimize potential loss of bridge access arising from abutment damage, monolithic or end diaphragm construction is strongly recommended for short span bridges. Settlement or approach slabs providing structural support between approach fills and abutments are recommended for all bridges classified as SPC D. Slabs shall be adequately linked to abutments using flexible ties. 7.5 STRUCTURAL STEEL DESIGN REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORIES C AND D 7.5.1 General Design and construction of structural steel columns and connections shall conform to the requirements of Division I and to the additional requirements of this section. Either Service Load or Load Factor design may be used. If Service Load design is used the allowable stresses are permitted to increase by 50%. It should be noted that when Service Load design is used for SPC C and D a conservative design may result because elastic design forces will be required for the design of most components unless the forces resulting from plastic hinging of the columns are used per Article 7.2.2. 7.5.2 P-delta Effects Where axial and flexural stresses are determined by considering secondary bending resulting from the design P-delta effects (moments induced by the eccentricity resulting from the seismic displacements and the column axial force), all axially loaded members may be proportioned in accordance with Division I, Article 10.36 or 10.54. 471 EXCEPTIONS: 1. The effective length factor, K, in the plane of bending may be assumed to be unity in the calculation of Fa, FL, Fa, or F,. 2. The coefficient C, is computed as for the cases where joint translation is prevented.
7.6 REINFORCED CONCRETE DESIGN REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORIES C AND D 7.6.1 General Design and construction of cast-in-place monolithic reinforced concrete columns, pier footings and connections shall conform to the requirements of Division I and to the additional requirements of this section. Either Service Load or Load Factor design may be used.
If Service Load design is used the allowable stresses are permitted to increase by 33Y3%.It should be noted that when Service Load design is used for SPC C and D a conservative design may result because elastic design forces will be required for the design of most components unless the forces resulting from plastic hinging of the columns are used per Article 7.2.2. 7.6.2 Column Requirements For the purpose of these provisions, a vertical support is considered to be a column if the ratio of the clear height to the maximum plan dimensions of the suppo~tis equal to or greater than 2.5. Note that the maximum plan dimension is taken at the minimum section of the flare for a flared column.
For supports with a ratio less than 2.5, the provisions for piers of Article 7.6.3 shall apply. For columns the provisions of this section are applicable. Note that a pier may be designed as a pier in its strong direction and a column in its weak direction. 7.6.2(A) Vertical Reinforcement The area of longitudinal reinforcement shall not be less than 0.01 or more than 0.06 times the gross cross-section area A,. EXCEPTION: Division I, Article 8.18.2.1 applies to columns where a larger cross-section is used for architectural reasons.
7.6.2(B) Flexural Strength The biaxial strength of columns shall not be less than that required for the bending moments determined in 472 HIGHWAY BRIDGES Article 7.2.3. The design of the column shall be checked for both the minimum and maximum axial loads specified in Article 7.2.3. The strength reduction factors of Division I, Article 8.16 shall be replaced for both spirally and tied reinforced columns by the value of 0.50 when the stress due to the maximum axial load for the column exceeds 0.20f:. The value of may be increased linearly from 0.50 to the value for flexure (0.90) when the stress due to the maximum axial load is between 0.20f,' and 0. Moment magnification for slenderness effects (Division I, Article 8.16.5) shall be considered in the design of the column. + 7.6.2(B) pected plastic hinge regions, generally located at the top and bottom of columns and pile bents, as specified in this subsection. The largest of these requirements or those of Article 7.6.2(C) shall govern; these requirements are not in addition to those of Article 7.6.2(C). The transverse reinforcement for confinement shall have a yield strength not more than that of the longitudinal reinforcement and the spacing shall be as specified in Article 7.6.2(E). The volumetric ratio of spiral reinforcement (p,) for a circular column shall be either that required in Division I, Article 8.18 or, 7.6.2(C) Column Shear and Transverse Reinforcement The factored design shear force V, of Division I, Equation (8-46) on each principal axis of each column and pile bent shall be the value determined in Article 7.2.3. The factored shear stress v, shall be computed using V, specified above and the strength reduction factor for shear of Division I, Article 8.16.1.2. The amount of transverse reinforcement shall be at least that specified by Division I, Article 8.16.6. In the end regions of the top and bottom of the column and pile bents, the following provisions shall apply in addition to those of Division I: 1. The shear strength of the concrete, V,, shall be in accordance with Division I, Article 8.16.6.2 when the axial load associated with the shear produces an average compression stress in excess of 0. lfd over the core concrete area of the support members. As the average compression stress increases from 0 to O.lff the strength V, increases linearly from 0 to the value given by Division I, Article 8.16.6.2. 2.
The end region shall be assumed to extend from the soffit of girders or cap beams at the top of columns, or the top of foundations at the bottom of columns, a distance not less than the minimum of (a) the maximum cross-sectional dimension of the column, (b) one-sixth of the clear height of the column, or (c) 18 inches (450 millimeters). 3. The end region of a pile bent shall be the same as specified for columns at the top of the pile bent, and three pile diameters below the calculated point of moment fixity to one pile diameter, but not less than 18 inches (450 millimeters) above the mud line at the bottom of the pile bent. 7.6.2(D) Transverse Reinforcement for Conjnement at Plastic Hinges The cores of columns, pile bents, and drilled shafts shall be confined by transverse reinforcement in the ex- whichever is greater. The total cross-sectional area (Ash)of rectangular hoop (stirrup) reinforcement for a rectangular column shall be either, whichever is greater, where: a = vertical spacing of hoops (stirrups) in inches (millimeters) with a maximum of 4 inches (100 millimeters). 4 = area of column core measured to the outside of the transverse spiral reinforcement. A, = gross area of column. Ash = total cross-sectional area in square inches (square millimeters) of hoop (stirrup) reinforcement including supplementarycross-ties having a vertical spacing of an inches (millimeters) and crossing a section having a core dimension of h, inches (millimeters). Note that this should be calculated for both principal axes of a rectangular column. ff = specified compressive strength of concrete in psi m a) . fyh = yield strength of hoop or spiral reinforcement in psi (MPa). 7.6.2(D) DIVISION IA-SEISMIC DESIGN core dimension of tied column in inches (millimeters) in the direction under consideration. p, = ratio of volume of spiral reinforcement to total volume of concrete core (out-to-out of spirals). h, = Transverse hoop reinforcement may be provided by single or overlapping hoops. Cross-ties having the same bar size as the hoop may be used. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar. A crosstie is a continuous bar having a hook of not less than 135" with an extension of not less than six-diameter, but not less than 3 inches (76 millimeters), at one end and a hook of not less than 90" with an extension of not less than six-diameter at the other end. The hooks shall engage peripheral longitudinal bars. The 90" hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end. A hoop is a closed tie or continuously wound tie. A closed tie may be made up of several reinforcing elements with 135" hooks having a six-diameter, but not less than 3 inches (76 millimeters), extension at each end. A continuously wound tie shall have at each end a 135" hook with a six-diameter, but not less than 3 inches (76 millimeters), extension that engages the longitudinal reinforcement. 7.6.2(E) Spacing of Transverse Reinforcement for Confinement 1. Transverse reinforcement for confinement shall be provided at the top and bottom of the column over a length equal to the maximum cross-sectional column dimension or one-sixth of the clear height of the column, whichever is the larger, but not less than 18 inches (450 millimeters). Transverse reinforcement shall be extended into the top and bottom connections as specified in Article 7.6.4. 2.
Transverse reinforcement for confinement shall be provided at the top of piles in pile bents over the same length as specified for columns. At the bottom of piles in pile bents, transverse reinforcement for confinement shall be provided over a length extending from three pile diameters below the calculated point of moment fixity to one pile diameter but not less than 18 inches (450 millimeters) above the mud line. 3. The maximum spacing for reinforcement shall not exceed the smaller of one-quarter of the minimum member dimension or 4 inches (100 millimeters). 4. Lapping of spiral reinforcement in the transverse confinement regions specified in 1 and 2 shall not be permitted. Connections of spiral reinforcement in this region must be full strength lap welds. 7.6.2(F) Splices Splices shall be in accordance with those specified in Division I, Article 8.32 and the additional requirements of 473 this Article. Lap splices shall be permitted only within the center half of column height, and the splice length shall not be less than 16 inches (400 millimeters) or 60 bar diameters, whichever is greater. The maximum spacing of the transverse reinforcement over the length of the splice shall not exceed the smaller of 4 inches (100 millimeters) or one-quarter of the minimum member dimension. Welded splices and approved mechanical splices that conform to the current provisions of ACI 318 may be used for splicing provided that splices shall not be used on any two adjacent bars in the same layer of longitudinal reinforcement at the same section and that the distance between splices of adjacent bars is greater than 24 inches (600 millimeters) as measured along the longitudinal axis of the column. 7.6.3 Pier Requirements The provisions of this article are applicable to the design for the strong direction of a pier. The weak direction of a pier may be designed as a column and the provisions of Article 7.6.2 are then applicable. In this case, the Response Modification Factor for columns may be used to determine the design forces in Article 7.2.1. If the pier is not designed as a column in its weak direction, the limitations for shear stress in this article are applicable. The minimum reinforcement ratio both horizontally, ph, and vertically p,, in any pier shall not be less than 0.0025.
Reinforcement spacing either horizontally or vertically shall not exceed 18 inches (457 millimeters). The reinforcement required for shear shall be continuous and shall be distributed uniformly. ph = the ratio of horizontal shear reinforcement area to gross concrete area of a vertical section. p, = the ratio of vertical shear reinforcement area to the gross concrete area of a horizontal section. The allowable shear stress, v,, in the pier shall be determined in accordance with the following equation: The allowable shear stress shall not exceed 8 e . For lightweight aggregate concrete, the limiting shear stress, v,, calculated from Equation (7-8), shall be multiplied by 0.75. Ttvo curtains of reinforcement shall be used and the reinforcement ratios p, and ph shall be equal. The reinforcement required by shear shall be uniformly distributed. Splices in horizontal pier reinforcement shall be staggered and splices in the two curtains shall not occur at the same location.
474 7.6.4 HIGHWAY BRIDGES Column Connections Acolumn connection as referred to in this section is the vertical extension of the column area into the adjoining member.
The design force for the connection between the column and the cap beam superstructure, pile cap, or spread footing shall be that specified in Article 7.2.5(C). The development length for all longitudinal steel shall be that required for a steel stress of 1 .25fyas given in Division I, Articles 8.24 through 8.32. Column transverse reinforcement required by Article 7.6.2(D) shall be continued for a distance equal to onehalf the maximum column dimension but not less than 15 inches (375 millimeters) from the face of the column connection into the adjoining member. The shear stress in the joint of a frame or bent, in the for direction under consideration, shall not exceed 12for lightnormal-weight aggregate concrete or 9weight aggregate concrete. 7.6.5 7.6.4 Construction Joints in Piers and Columns Construction joints in piers and columns resisting seismic forces shall be designed and constructed to resist the design forces at the joint. Where shear is resisted at a construction joint solely by dowel action and friction on a roughened concrete surface, the total shear force across the joint shall not exceed Vj determined from the following formula: where AVfis the total area of reinforcement (including flexural reinforcement), P, is the minimum axial load specified in Article 7.2.3 for columns and Article 7.2.4 for piers, and is the strength reduction factor for shear of Division I, Article 8.16. + Division I1 CONSTRUCTION INTRODUCTION This Division of the Standard Specijications for Highway Bridges includes the basic technical constructionspecifications needed for the construction of bridges and other major transportation structures. They generally represent current practices in the United States and are consistent with the AASHTO Design Specifications for Bridges which are contained in Division I. They are provided to be used either as part of the specifications for projects or as a guide for agencies in developing their own standards. When so used, uniformity and the efficiencies associated therewith may be realized. These technical specifications do not include the clauses needed for the administration of a contract and were written to be used in conjunction with general provisions such as those in the AASHTO Guide Specijications for Highway Construction. Other comparable sets of general provision clauses currently in use by many States can also be used to cover the administration requirements for construction contracts. The Guide Specifications and these Standard Specifications are intended to be complementary and to provide for the principal and most widely used items of work required for the construction of major transportation structures. Note that these specifications do not identify the date of specifications, which are included by reference, such as the AASHTO Standard Specijications for Transportation Materials and Methods of Testing and Sampling.As required by the AASHTO Guide Specification, the edition of such specifications incorporated by reference will be the edition in effect on the date of advertisement for proposals for the project. Sufficient detail may not be included in these specifications to suit local or unusual conditions or unique designs. The many differences in climate, geology, customs, statutes and regulations prevent the writing of a more detailed national construction specification. Therefore, the user is expected to supplement or alter the requirements of these specifications, as needed, in the project special provisions. A Commentary is provided to assist the user in developing such special provisions. These specifications were extensively revised under NCHRP 12-34 in 1989 and approved by AASHTO Highway Subcommittee on Bridges and Structures in 1990. Section 1 STRUCTURE EXCAVATION AND BACKFILL 1.1 GENERAL 1.2 WORKING DRAWINGS Structure excavation shall consist of the removal of all material, of whatever nature, necessary for the construction of foundations for bridges, retaining walls, and other major structures in accordance with the plans or as directed by the Engineer. If not otherwise provided for in the contract, structure excavation shall include the furnishing of all necessary equipment and the construction and subsequent removal of all cofferdams, shoring, and water control systems which may be necessary for the execution of the work.
It shall also include, if not otherwise specified, the placement of all necessary backfill, including any necessary stockpiling of excavated material which is to be used in backfill, and the disposing of excavated material, which is not required for backfill, in roadway embankments or as provided for excess and unsuitable material in Subsection 203.02, AASHTO Guide Specifications for Highway Construction. If the contract does not include a separate pay item or items for such work, structure excavation shall include all necessary clearing and grubbing and the removal of existing structures within the area to be excavated. Classification, if any, of excavation will be indicated on the plans and set forth in the proposal. The removal and disposal of buried natural or manmade objects are included in the class of excavation in which they are located, unless such removal and disposal are included in other items of work. However, in &case of a buried manmade object, if (1) its removal requires the use of methods or equipment not used for other excavation on the project, (2) its presence was not indicated on the plans or in the special provisions, (3) its presence could not have been ascertained by site investigation, including contact with identified utilities within the area, and (4) the Contractor so requests in writing prior to its removal, the removal and disposal of such object will be paid for as extra work, and its volume will not be included in the measured quantity of excavation. Whenever specified, the Contractor shall provide working drawings, accompanied by calculations where appropriate, of excavation procedures, embankment construction and backfilling operations. This plan shall show the details of shoring, bracing, slope treatment or other protective system proposed for use and shall be accompanied by design calculations and supporting data in sufficient detail to pennit an engineering review of the proposed design. The working drawings and plans for protection from caving shall be submitted sufficiently in advance of proposed use to allow for their review, revision, if needed, and approval without delay to the work. Working drawings must be approved by the Engineer prior to performance of the work involved and such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work. 1.3 MATERIALS Material used for backfill shall be free of frozen lumps, wood or other degradable matter and shall be of a grading such that the required compaction can be consistently obtained using the compaction methods selected by the Contractor. Permeable material for underdrains shall conform to AASHTO Guide S~ecifi;fications for Highway Construetion, 704.01- 1.4 CONSTRUCTION 1.441 Depth of Footings The elevation of the bottoms of footings, as shown on the plans, shall be considered as approximate only and the Engineer may order, in writing, such changes in dimensions or elevation of footings as may be necessary to secure a satisfactory foundation. 478 HIGHWA;Y BRIDGES 1.4.2 Foundation Preparation and Control of Water 1.4.2.1 General All substructures, where practical, shall be constructed in open excavation and, where necessary, the excavation shall be shored, braced, or protected by cofferdams constructed in accordance with the requirements contained in Article 3.3, "Cofferdams and Shoring." When footings can be placed in the dry without the use of cofferdams, backforms may be omitted with the approval of the Engineer, and the entire excavation filled with concrete to the required elevation of the top of the footing. The additional concrete required shall be furnished and placed at the expense of the Contractor. Temporary water control systems shall conform to the requirements contained in Article 3.4, "Temporary Water Control Systems." 1.4.2 foundation material to grade shall not be made until just before the footing is to be placed. Where the material below the bottom of footings not supported by piles has been disturbed, it shall be removed and the entire space filled with concrete or other approved material at the Contractor's expense. Under footings supported on piles, the over-excavation or disturbed volumes shall be replaced and compacted as directed by the Engineer. 1.4.2.5 Approval of Foundation After each excavation is completed, the Contractor shall notify the Engineer, and no concrete or other footing material shall be placed until the Engineer has approved the depth of the excavation and the character of the foundation material. 1.4.2.2 Excavations Within Channels When excavation encroaches upon a live stream bed or channel, unless otherwise permitted, no excavation shall be made outside of caissons, cribs, cofferdams, steel piling, or sheeting, and the natural stream bed adjacent to the structure shall not be disturbed without permission from the Engineer. If any excavation or dredging is made at the site of the structure before caissons, cribs, or cofferdams are sunk or are in place, the Contractor shall, without extra charge, after the foundation base is in place, backfill all such excavation to the original ground surface or river bed with material satisfactory to the Engineer. Material temporarily deposited within the flow area of streams from foundation or other excavation shall be removed and the stream flow area freed from obstruction thereby. 1.4.2.3 Foundations on Rock When a foundation is to rest on rock, the rock shall be freed from all loose material, cleaned and cut to a firm surface, either level, stepped, or roughened, as may be directed by the Engineer.
All seams shall be cleaned out and filled with concrete, mortar, or grout before the footing is placed. Where blasting is required to reach footing level, any loose, fractured rock caused by overbreak below bearing level shall be removed and replaced with concrete or grouted at the Contractor's expense. 1.4.2.4 Other Foundations When a foundation is to rest on an excavated surface other than rock, special care shall be taken not to disturb the bottom of the excavation, and the final removal of the Backfill material shall conform to the provisions of Article 1.3. If sufficient material of suitable quality is not available from excavation within the project limits, the Contractor shall import such material as directed by the Engineer. All spaces excavated and not occupied by abutments, pi

of the concrete. Falsework or formwork for deck slabs on girder bridges shall be supported directly on the girders so that there will be no appreciable differential settlement during placing of the concrete. Girders shall be braced and tied to resist any forces that would cause rotation or torsion in the girders caused by the placing of concrete for diaphragms or deck. Welding of falsework support brackets or braces to structural steel members or reinforcing steel will not be allowed unless specifically permitted. 3.2.3 Formwork Design and Construction 3.23.1 General Forms shall be of wood, steel, or other approved material and shall be mortar tight and of sufficient rigidity to prevent objectional distortion of the formed concrete surface due to pressure of the concrete and other loads incidental to the construction operations.
Forms for concrete surfaces exposed to view shall produce a smooth surface of uniform texture and color substantially equal to that which would be obtained with the use of plywood conforming to the National Institute of Standards and Technology Product Standard PSI for Exterior B-B Class I Plywood. Panels lining such forms shall be arranged so that the joint lines form a symmetrical pattern conforming to the general lines of the structure. The same type of form lining material shall be used throughout each element of a structure. Such forms shall be sufficiently rigid so that the undulation of the concrete surface shall not exceed l/s inch when checked with a 5-foot-long straightedge or template. All sharp comers shall be filleted with approximately 3/4-inch chamfer strips. Concrete shall not be deposited in the forms until all work connected with constructing the forms has been completed, all debris has been removed, all materials to be embedded in the concrete have been placed for the unit to be cast, and the Engineer has inspected the forms and materials. 3.2.3.2 Design The structural design of formwork shall conform to ACI Standard, "Recommended Practice for Concrete Formwork," (ACI 347) or some other generally accepted 485 standard. In selecting the hydrostatic pressure to be used in the design of forms, consideration shall be given to the maximum rate of concrete placement to be used, the effects of vibration, the temperature of the concrete and any expected use of set-retarding admixtures or pozzolanic materials in the concrete mix. 3.2.3.3 Construction Forms shall be set and held true to the dimensions, lines and grades of the structure prior to and during the placement of concrete. Forms may be given a bevel or draft at projections, such as copings, to ensure easy removal. Prior to reuse, forms shall be cleaned, inspected for damage and, if necessary, repaired. When forms appear to be defective in any manner, either before or during the placement of concrete, the Engineer may order the work stopped until defects have been corrected. Forms shall be treated with form oil or other approved release agent before the reinforcing steel is placed.
Material which will adhere to or discolor the concrete shall not be used. Except as provided herein, metal ties or anchorages within the forms shall be so constructed as to permit their removal to a depth of at least 1 inch from the face without injury to the concrete. Ordinary wire ties may be used only when the concrete will not be exposed to view and where the concrete will not come in contact with salts or sulfates. Such wire ties, upon removal of the forms, shall be cut back at least '/4 inch from the face of the concrete with chisels or nippers; for green concrete, nippers shall be used. Fittings for metal ties shall be of such design that, upon their removal, the cavities that are left will be of the smallest possible size. The cavities shall be filled with cement mortar and the surface left sound, smooth, even, and uniform in color. When epoxy-coated reinforcing steel is required, all metal ties, anchorages or spreaders which will remain in the concrete shall be of corrosion resistant material or coated with a dielectric material. For narrow walls and columns, where the bottom of the form is inaccessible, an access opening shall be provided in the forms for cleaning out extraneous material immediately before placing the concrete. 3.2.3.4 Tube Forms Tubes used as forms to produce voids in concrete slabs shall be properly designed and fabricated or otherwise treated to make the outside surface waterproof. Prior to concrete placement such tubes shall be protected from the weather and stored and installed by methods that prevent distortion or damage. The ends of tube forms shall be cov- 486 HIGHWAY BRIDGES 3.2.3.4 - ered with caps that shall be made mortar tight and waterproof. If wood or other material that expands when moist is used for capping tubes, a premolded rubber joint filler 1/4 inch in thickness shall be used around the perimeter of the caps to permit expansion. A PVC vent tube shall be provided near each end of each tube. These vents shall be constructed to provide positive venting of the voids. After exterior form removal, the vent tube shall be trimmed to within 1/2 inch of the bottom surface of the finished concrete. Anchors and ties for tube forms shall be adequate to prevent displacement of the tubes during concrete placement. 3.2.3.5 Stay-in-Place Forms Stay-in-place deck soffit forms, such as corrugated metal or precast concrete panels, may be used if shown on the plans or approved by the Engineer. Prior to the use of such forms the Contractor shall provide a complete set of details to the Engineer for review and approval. The detailed plans for structures, unless otherwise noted, are dimensioned for the use of removable forms and any changes necessary to accommodate stay-in-place forms, if approved, shall be at the expense of the Contractor. 3.2.4 Removal of Falsework and Forms 3.2.4.1 General Falsework or forms shall not be removed without approval of the Engineer. In the determination of the time for the removal of falsework and forms, consideration shall be given to the location and character of the structure, the weather, the materials used in the mix,and other conditions influencing the early strength of the concrete. Methods of removal likely to cause overstressing of the concrete or damage to its surface shall not be used. Supports shall be removed in such a manner as to permit the structure to uniformly and gradually take the stresses due to its own weight.
For arch structures of two or more spans, the sequence of falsework release shall be as specified or approved. 3.2.4.2 Time of Removal If field operations are not controlled by beam or cylinder tests, the following minimum periods of time, exclusive of days when the temperature is below 40F, shall have elapsed after placement of concrete before falsework is released or forms are removed: Falsework for: Spans over 14 feet Spans of 14 feet or less Bent caps not yet supporting girders Forms: Not supporting the dead weight of the concrete For interior cells of box girders and for railings 14 days 10 days 10 days 24 hours 12 hours If high early strength is obtained with Type III cement or by the use of additional cement, these periods may be reduced as directed. When field operations are controlled by cylinder tests, the removal of supporting forms or falsework shall not begin until the concrete is found to have the specified compressive strength, provided further that in no case shall supports be removed in less than 7 days after placing the concrete. In addition to the above time requirements: Forms shall not be removed until the concrete has sufficient strength to prevent damage to the surface. Falsework for post-tensioned portions of structures shall not be released until the prestressing steel has been tensioned. Falsework supporting any span of a continuous or rigid frame bridge shall not be released until the aforementioned requirements have been satisfied for all of the structural concrete in that span and in the adjacent portions of each adjoining span for a length equal to at least one-half the length of the span where falsework is to be released. Unless otherwise specified or approved, falsework shall be released before the railings, copings or barriers are placed for all types of bridges. For arch bridges, the time of falsework release relative to the construction of elements of the bridge above the arch shall be as shown on the plans or directed by the Engineer. 3.2.4.3 Extent of Removal All falsework and forms shall be removed except: Portions of driven falsework piles more than 1 foot below subgrade within roadbeds, or 2 feet below the original ground or finished grade outside of roadbeds, or 2 feet below the established limits of any navigation channel. Footing forms where their removal would endanger the safety of cofferdams or other work. 3.2.4.3 DMSION 11--CONSTRUCTION Forms from enclosed cells where access is not provided. Deck forms in the cells of box girder bridges that do not interfere with the future installation of utilities shown on the plans. 3.3 COFFERDAMS AND SHORING 3.3.1 General Cofferdams and shoring consist of those structures used to temporarily hold the surrounding earth and water out of excavations and to protect adjacent property and facilities during construction of the permanent work. Cofferdams shall be constructed to adequate depths, generally well below the bottom of the excavation, and to adequate heights to seal off all water. They shall be safely designed and constructed, and be made as watertight as is necessary for the proper performance of the work which must be done inside them. In general, the interior dimensions of cofferdams shall be such as to give sufficient clearance for the construction of forms and the inspection of their exteriors, and to permit pumping from outside the forms. Cofferdams which are tilted or moved laterally during the process of sinking shall be righted, reset, or enlarged so as to provide the necessary clearance. This shall be solely at the expense of the Contractor. When water cannot be controlled so that footing concrete can be placed in the dry, a cofferdam shall be employed, and a concrete seal conforming to the requirements of Section 8, "Concrete Structures" placed underwater below the elevation of the footing. When such a seal is shown on the plans, the Engineer will determine if a cofferdam and seal is required, the depth of the seal to be used, and the required cure time.
Such determination will be based on conditions existing at the time of construction. When a concrete seal is not shown on the plans, the Contractor shall make these determinations, and shall be fully responsible for the performance of the seal. After the seal has cured, the cofferdam shall then be pumped out and the balance of the masonry placed in the dry. When weighted cofferdams are employed and the weight is utilized to partially overcome the hydrostatic pressure acting against the bottom of the foundation seal, special anchorage such as dowels or keys shall be provided to transfer the entire weight of the cofferdam into the foundation seal. During the placing and curing of a foundation seal, the elevation of the water inside the cofferdam shall be controlled to prevent any flow through the seal, and if the cofferdam is to remain in place, it shall be vented or ported at or below low water level. Shoring shall be adequate to support all loads imposed and shall comply with any applicable safety regulations. 487 3.3.2 Protection of Concrete Cofferdams shall be constructed so as to protect green concrete against damage from a sudden rising of the stream and to prevent damage to the foundation by erosion. No struts or braces shall be used in cofferdams or shoring systems in such a way as to extend into or through the permanent work, without written permission from the Engineer. 3.3.3 Removal Unless otherwise provided or approved, cofferdams, and shoring with all sheeting and bracing shall be removed after the completion of the substructure, with care being taken not to disturb or otherwise injure the finished work. 3.4 TEMPORARY WATER CONTROL SYSTEMS 3.4.1 General Temporary water control systems consist of dikes, bypass channels, flumes and other surface water diversion works, cut-off walls and pumping systems, including wellpoint and deep well systems, used to prevent water from entering excavations for structures. 3.4.2 Drawings Working drawings for temporary water control systems, when required, shall include details of the design and the equipment, operating procedures to be employed, and location of point or points of discharge. The design and operation shall conform to all applicable water pollution control requirements. 3.4.3 Operations Pumping from the interior of any foundation enclosure shall be done in such manner as to preclude the possibility of the movement of water through any fresh concrete. No pumping will be permitted during the placing of concrete or for a period of at least 24 hours thereafter, unless it be done from a suitable sump separated from the concrete work by a watertight wall or other effective means subject to approval of the Engineer. Pumping to unwater a sealed cofferdam shall not commence until the seal has set sufficiently to withstand the hydrostatic pressure. Pumping from wellpoints or deep wells shall be regulated so as to avoid damage by subsidence to adjacent property.
488 HIGHWAY BRIDGES 3.5 TEMPORARY BRIDGES 3.5.1 General Temporary bridges include detour bridges for use by the public, haul road bridges and other structures, such as conveyor bridges, used by the Contractor. Temporary bridges shall be constructed, maintained and removed in a manner that will not endanger the work or the public. 3.5.2 Detour Bridges When a design is furnished by the Department, detour bridges shall be constructed and maintained to conform to such design or an approved alternativedesign.
When permitted by the specifications, the Contractor may submit a proposed alternative design. Any alternative design must be equivalent in all respects to the design and details furnished by the Department and is subject to approval by the Engineer. The working drawings and design calculations for any alternative design must be signed by a Registered Professional Engineer. When a design is not furnished by the Department, the Contractor shall prepare the design and furnish working drawings to the Engineer for approval. The design shall provide the clearances, alignment, load capacity and other design parameters specified or approved. The design shall conform to the Standard Specifcations for Highway Bridges adopted by AASHTO. If design live loads are not otherwise specified, an HS I1 15-44 loading shall be used. The working drawings and design calculations shall be signed by a Registered Professional Engineer. 3.5.3 Haul Bridges When haul road bridges or other bridges which are not for public use are proposed for construction over any right-of-way which is open to the public or over any railroad, working drawings showing complete design and de- 3.5 tails, including the maximum loads to be carried, shall be submitted to the Engineer for approval. Such drawings shall be signed by a Registered Professional Engineer. The design shall conform to AASHTO design standards when applicable or to other appropriate standards. 3.5.4 Maintenance The maintenance of temporary bridges for which working drawings are required shall include their replacement in case of partial or complete failure. The Department reserves the right, in case of the Contractor's delay or inadequate progress in making repairs and replacement, to furnish such labor, materials, and supervision of the work as may be necessary to restore the structure for proper movement of traffic. The entire expense of such restoration and repairs shall be considered a part of the cost of the temporary structure and where such expenditures are incurred by the Department, they shall be charged to the Contractor. 3.6 MEASUREMENT AND PAYMENT Unless otherwise provided, payment for temporary works shall be considered to be included in the payment for the various items of work for which they are used and no separate payment will be made therefore. When an item for concrete seals for cofferdams is included in the bid schedule, such concrete will be measured and paid for as provided in Section 8, "Concrete Structures." When an item or items for temporary bridges, cofferdams, shoring systems or water control systems is included in the bid schedule, payment will be the lump sum bid for each such structure or system which is listed on the bid schedule and which is constructed and removed in accordance with the contract requirements. Such payment includes full compensation for all costs involved with the furnishing of all materials and the construction, maintenance, and removal of such temporary works. Section 4 DRIVEN FOUNDATION PILES 4.1 DESCRIPTION This work shall consist of furnishing and driving foundation piles of the type and dimensions designated on the plans or in the special provisions including cutting off or building up foundation piles when required. This specification also covers providing test piles and performing loading tests. Piling shall conform to and be installed in accordance with these specifications, and at the location, and to the elevation, penetration, and bearing capacity shown on the plans or as directed by the Engineer. Any improperly driven, broken, or otherwise defective pile shall be corrected to the satisfaction of the Engineer by removal and replacement, or the driving of an additional pile, at no extra cost. Except when test piles are required, the Contractor shall furnish the piles in accordance with the dimensions shown on the plans or special provisions. When test piles are required, the pile lengths shown on the plans are for estimating purposes only and the actual lengths to be furnished for production piles will be determined by the Engineer after the test piles have been driven. The lengths given in the Engineer's order list will include only the lengths anticipated for use in the completed structure. The Contractor shall, without added compensation, increase the lengths shown or ordered to provide for fresh heading and for such additional length as may be necessary to suit the method of operation. 4.2 MATERIALS 4.2.1 Steel Piles The structural steel used for foundation piling shall conform to the Specification for Structural Steel for Bridges, AASHTO M 270 (ASTM A 709) Grades 36,50, or 50W, or to the Specification for Piling for Use in Marine Environment, ASTM A 690. 4.2.1.1 Painting Unless otherwise provided, when steel piles or steel pile shells extend above the ground surface or water sur- face they shall be protected by the paint system specified for painting new steel in a high pollution or coastal environment as described in Section 13, "Painting." This protection shall extend from an elevation 2 feet below the water or ground surface to the top of the exposed steel. 4.2.2 Timber Piles Timber piles shall conform to the requirements of the Specification for Wood Products, AASHTO M 168. Timber piles shall be treated or untreated as indicated on the plans or in the special provisions. Preservative treatment shall conform to the requirements of Section 17, "Preservative Treatment of Wood." The method of storing and handling shall be such as to avoid injury to the piles. Special care shall be taken to avoid breaking the surface of treated piles. Canthooks, dogs, or pike-poles shall not be used. Cuts or breaks in the surface of treated piling and bolt holes shall be treated as specified in Article 16.3.3, "Treated Timber." 4.2.3 Concrete Piles Concrete piles shall consist of either precast concrete piles or cast-in-place concrete piles cast in steel shells. cement shall to the requireStructures," and unless anments in Section 8, GcConcrete other class is shown on the plans or specified, concrete shall be Class A.
Reinforcingsteel shall conform to the requirements of Section 9, "Reinforcing Steel," and prestressing shall conform to the requirements of Section 10, "Prestressing." Steel shells for cast-in-place concrete piles shall be of not less than the thickness shown on the plans.
The Contractor shall furnish shells of greater thickness if necessary to provide sufficient strength and rigidity to permit driving with the equipment selected for use without damage, and to prevent distortion caused by soil pressures or the driving of adjacent piles. The shells shall also be watertight to exclude water during the placing of concrete. The shells may be cylindrical or tapered, step-tapered, or a combination of either, with cylindrical sections. 490 HIGHWAY BIUDGES 4.3 MANUFACTURE OF PILES 4.3.1 Precast Concrete Piles 4.3.1.1 Forms Forms for precast concrete piles shall conform to the general requirements for concrete form work as provided in Section 3, "Temporary Works." Forms shall provide access for vibration and consolidation of the concrete. 4.3 4.3.1.5.1 Working Drawings The Contractor shall submit two sets of working drawings to the Engineer at the job site for prestressed concrete piles. Said drawings shall show the pile dimensions, materials, prestressing methods, tendon arrangement and prestressing forces proposed for use and, any addition or rearrangement of reinforcing steel from that shown on the plans. Construction of the piles shall not begin until the drawings have been approved by the Engineer.
4.3.1.2 Casting 4.3.1.6 Storage and Handling Handling and placing of concrete shall conform to the requirements of Section 8, "Concrete Structures," and these specifications. Special care shall be taken to place the concrete so as to produce satisfactory bond with the reinforcement and avoid the formation of "stone pockets," honeycomb, or other such defects. To secure uniformity, the concrete in each pile shall be placed continuously and shall be compacted by vibrating or by other means acceptable to the Engineer. The forms shall be overfilled, the surplus concrete screeded off, and the top surfaces finished to a uniform, even texture similar to that produced by the forms. Portions of piling exposed to view shall be finished in accordance with the provisions governing the finishing of concrete columns. Other piling shall not be finished except as set forth above. Removal of forms, curing, storing, transporting, and handling of precast concrete piles shall be done in such a manner as to avoid excessive bending stresses, cracking, spalling, or other injurious results. Piles to be used in sea water or in sulfate soils shall be handled so as to avoid surface abrasions or other injuries exposing the interior concrete. 4.3.2 Cast-in-Place Concrete Piles 4.3.2.1 Inspection of Metal Shells At all times prior to the placing of concrete in the driven shells, the Contractor shall have available a suitable light for the inspection of each shell throughout its entire length. 4.3.2.2 Placing Concrete 4.3.1.4 Curing and Protection Concrete piles shall be cured as provided in Section 8, "Concrete Structures," and these Specifications. As soon as the piles have set sufficiently to avoid damage, they shall be removed from the forms and stacked in a curing pile separated from each other by wood-spacing blocks. No pile shall be driven until at least 21 days after casting and, in cold weather, for a longer period as determined by the Engineer. Concrete piles for use in sea water or sulfate soils shall be cured for not less than 30 days before being used. Concrete shall be protected from freezing until the compressive strength reaches at least 0.8 f:. 4.3.1.5 Prestressing Prestressing of concrete piles shall conform to the provisions of Section 10, "Prestressing." No concrete shall be placed until all driving within a radius of 15 feet of the pile has been completed, or all driving within the above limits shall be discontinued until the concrete in the last pile cast has set at least 5 days. Concrete for cast-in-place piles shall be dense and homogeneous. In lieu of the provisions concerning vibration of concrete as specified in Article 8.7.4, vibration or rodding of concrete for cast-in-place piles will only be required to a depth of 5 feet below the ground surface. Concrete shall be placed for each pile in a single continuous operation with the flow of concrete directed down the center of the pile so as to consolidate the concrete by impact. Accumulations of water in shells shall be removed before the concrete is placed. After the concrete has hardened, the top surface shall be cut back to remove laitance and to expose the aggregate as specified in Article 8.8.
4.4 DIVISION 11--CONSTRUCTION 4.4 DRIVING PILES 4.4.1 Pile Driving Equipment Driving equipment that damages the piling shall not be used.
All pile driving equipment, including the pile driving hammer, hammer cushion, drive head, pile cushion and other appurtenances to be furnished by the Contractor shall be approved in advance by the Engineer before any driving can take place. Pursuant to obtaining this approval, the Contractor shall submit, at least 2 weeks before pile dnving is to begin, a description of pile driving equipment to the Engineer. Whenever the bearing capacity of piles is specified to be determined by Method B, "Wave Equation Analysis," the Contractor shall also submit calculations, based on a wave equation analysis, demonstrating that the piles can be driven with reasonable effort to the ordered lengths without damage. The following hammer eff~cienciesshall be used in a wave equation analysis: Hammer Type Efficiency in Percent Single acting airlstearn Double acting aidsteam Diesel 67 50 72 In addition to the other requirements of these specifications, the criteria which the Engineer will use to evaluate the driving equipment consists of both the required number of hammer blows per inch and the pilestresses at the required ultimate pile capacity. The required number of hammer blows indicated by calculations at the required bearing capacity shall be between 3 and 10 per inch for the driving equipment to be acceptable. In addition, for the driving equipment to be acceptable, the pile stresses, which are indicated by the calculations, to be generated by the driving equipment shall not exceed the values where pile damage impends. The point of irnpending damage in steel piles is defined herein as a cornpressive driving s a s s s of 90% of the yield point of the pile material. For concrete piles, tensile stresses shall not exceed 3 multiplied by the square root of the concrete compressive strength, f,', plus the effective prestress value, i.e., (3 + prestress), ~ and compressive stresses shall not exceed 85% of the compressive strength minus the effective prestress value, i.e. (0.85 f,' - prestress). For timber piles, the compressive driving stress shall not exceed three times the allowable static design strength listed on the plans.
These criteria will be used in evaluating calculated results to determine acceptability of the Contractor's proposed driving system. 491 During pile driving operations, the Contractor shall use the approved system. Any change in the driving system will only be considered after the Contractor has submitted revised pile driving equipment data and calculations. The Contractor will be notified of the acceptance or rejection of the driving system changes within 7 calendar days of the Engineer's receipt of the requested change. The time required for submission, review, and approval of a revised driving system shall not constitute the basis for a contract time extension to the Contractor.
Approval of pile driving equipment shall not relieve the Contractor of his responsibility to drive piles, free of damage, to the bearing and tip elevation shown on the plans or specified in the special provisions. 4.4.1.1 Hammers 4.4.1.1.1 General Piles may be driven with a drop hammer, an airisteam hammer, or diesel hammer conforming to these specifications. Pile driving hammers, other than drop hammers, shall be of the size needed to develop the energy required to drive piles at a penetration rate of not less than 0.10 inch per blow at the required bearing value. 4.4.1.1.2 Drop Hammers Drop (gravity) hammers shall not be used for concrete piles or for piles whose design load capacity exceeds 30 tons. When gravity hammers are permitted, the ram shall weigh not less than 2,000 pounds and the height of drop shall not exceed 15 feet. In no case shall the ram weight of gravity hammers be less than the combined weight of the drive cap and pile. All gravity hammers shall be equipped with hammer guides to insure concentric impact on the drive head or pile cushion. 4.4.1.1.3 Air Steam Hammers The weight of the striking part of adsteam hammers used shall not be less than lh the weight of pile and drive weigh less than cap, and in no case shall the sang 2,750 pounds. The plant and equipment furnished for aidsteam hammers shall have sufficient capacity to maintain, under working conditions, the pressure at the hammer specified by the manufacturer. 4.4.1.1.4 Diesel Hammers Open-end (single acting) diesel hammers shall be equipped with a device to permit the Engineer to determine hammer stroke at all times during pile driving operations. Closed-end (double acting) diesel hammers shall , 492 HIGHWAY BRIDGES be equipped with a bounce chamber pressure gauge, in good working order, mounted near ground level so as to be easily read by the Engineer. A correlation chart of bounce chamber pressure and delivered hammer energy shall be provided by the Contractor. 4.4.1.1.5 Vibratory Hammers Vibratory or other pile driving methods may be used only when specifically allowed by the Special Provisions or in writing by the Engineer. Except when pile lengths have been determined from load test piles, the bearing capacity of piles driven with vibratory hammers shall be verified by redriving the first pile driven in each group of 10 piles with an impact hammer of suitable energy to measure the pile capacity before driving the remaining piles in the group. 4.4.1.1.6 Additional Equipment or Methods In case the required penetration is not obtained by the use of a hammer complying with the above minimum requirements, the Contractor may be required to provide a hammer of greater energy or, when permitted, resort to supplemental methods such as jetting or preboring. 4.4.1.2 Driving Appurtenances 4.4.1.2.1 Hammer Cushion All impact pile driving equipment except gravity hammers shall be equipped with a suitable thickness of hammer cushion material to prevent damage to the hammer or pile and to insure uniform driving behavior.
Hammer cushions shall be made of durable, manufactured materials, which will retain uniform propertiesduring driving. Wood, wire rope, and asbestos hammer cushions shall not be used. A striker plate shall be placed on the hammer cushion to insure uniform compression of the cushion material. The hammer cushion shall be inspected in the presence of the Engineer when beginning pile driving and after each 100 hours of pile driving. The hammer cushion shall be replaced by the Contractor before driving is permitted to continue whenever there is a reduction of hammer cushion thickness exceeding 25% of the original thickness. 4.4.1.2.2 Pile Drive Head Piles driven with impact hammers shall be fitted with an adequate drive head to distribute the hammer blow to the pile head.
The drive head shall be axially aligned with the hammer and the pile. The drive head shall be guided by the leads and not be free-swinging. The drive head shall fit around the pile head in such a manner as to prevent transfer of torsional forces during driving while maintaining proper alignment of hammer and pile. 4.4.1.1.4 For steel and timber piling, the pile heads shall be cut squarely and a drive head provided to hold the longitudinal axis of the pile in line with the axis of the hammer. For precast concrete and prestressed concrete piles, the pile head shall be plane and perpendicular to the longitudinal axis of the pile to prevent eccentric impacts from the drive head. For special types of piles, appropriate driving heads, mandrels or other devices shall be provided so that the piles may be driven without damage. 4.4.1.2.3 Pile Cushion The heads of concrete piles shall be protected by a pile cushion when the nature of the driving is such as to unduly injure them. When plywood is used, the minimum thickness placed on the pile head prior to driving shall not be less than 4 inches. Anew pile cushion shall be provided if, during driving, the cushion is either compressed more than one-half the original thickness or begins to burn. The pile cushion dimensions shall be such as to distribute the blow of the hammer throughout the cross section of the pile. 4.4.1.2.4 Leads Pile driving leads which support the pile and the hammer in proper positions throughout the driving operation shall beused. Leads shall be constructed in a manner that affords freedom of movement of the hammer while maintaining alignment of the hammer and the pile to insure concentric impact for each blow.
The leads shall be of sufficient length to make the use of a follower unnecessary and shall be so designed as to permit proper alignment of battered piles. 4.4.1.2.5 Followers Followers shall only be used when approved in writing by the Engineer, or when specifically allowed in the special provisions. When a follower is permitted, in order to venfy that adequate pile penetration is being attained to develop the desired pile capacity, the first pile in each bent and every 10th pile driven thereafter shall be furnished sufficiently long and shall be driven full length without a follower. The follower and pile shall be held and maintained in equal and proper alignment during driving. The follower shall be of such-material and dimensions to permit the piles to be driven to the length determined necessary from the driving of the full length piles. The final position and alignment of the first two piles installed with followers in each substructure unit shall be verified to be in accordance with the location tolerances specified in Article 4.4.3 before additional piles are installed. 4.4.1.2.6 DMSION II-CONSTRUCTION 4.4.1.2.6 Jets Jetting shall only be permitted if approved in writing by the Engineer or when specifically allowed in the special provisions. When jetting is not required, but approved after the Contractor's request, the Contractor shall determine the number of jets and the volume and pressure of water at the jet nozzles necessary to freely erode the material adjacent to the pile without affecting the lateral stability of the final in-place pile. The Contractor shall be responsible for all damage to the site caused by jetting operations. When jetting is specifically required in the special provisions, the jetting plant shall have sufficient capacity to deliver at all times a pressure equivalent to at least 100 pounds per square inch at two %-inch jet nozzles. In either case unless otherwise indicated by the Engineer, jet pipes shall be removed when the pile tip is a minimum of 5 feet above prescribed tip elevation and the pile shall be driven to the required bearing capacity with an impact hammer. Also, the Contractor shall control, treat if necessary, and dispose of all jet water in a manner satisfactory to the Engineer. 4.4.2 Preparation for Driving 4.4.2.1 Site Work 4.4.2.1.1 Excavation In general, piles shall not be driven until after the excavation is complete. Any material forced up between the piles shall be removed to the correct elevation before concrete for the foundation is placed. 4.4.2.1.2 Preboring to Facilitate Driving When required by the special provisions, the Contractor shall prebore holes at pile locations to the depths shown on the plans, specified in the special provisions, or allowed by the Engineer.
Prebored holes shall be smaller than the diameter or diagonal of the pile cross section and sufficient to allow penetration of the pile to the specified depth.
If subsurface obstructions, such as boulders or rock layers are encountered, the hole diameter may be increased to the least dimension which is adequate for pile installation. Any void space remaining around the pile after completion of driving shall be filled with sand or other approved material. The use of spuds (a short strong driven member which is removed to make a hole for inserting a pile), shall not be permitted in lieu of preboring, unless specifically allowed by the special provisions or in writing by the Engineer.
4.4.2.1.3 Predrilled Holes in Embankments Piles to be driven through newly constructed embankments shall be driven in holes drilled or spudded through 493 the embankment when the depth of the new embankment is in excess of 5 feet. The hole shall have a diameter of not less than the greatest dimension of the pile cross section plus 6 inches. After driving the pile, the space around the pile shall be filled to ground surface with dry sand or pea gravel. Material resulting from drilling holes shall be disposed of as approved by the Engineer. 4.4.2.2 Preparation of Piling In addition to squaring up pile heads prior to driving, piles shall be further prepared for driving as described below. 4.4.2.2.1 Collars When timber piles are required to be driven to more than 35 tons bearing or when driving conditions otherwise require it, collars, bands, or other devices shall be provided to protect piles against splitting and brooming. 4.4.2.2.2 Pointing Timber piles shall be pointed where soil conditions require it. When necessary, the piles shall be shod with metal shoes of a design satisfactory to the Engineer, the points of the piles being carefully shaped to secure an even and uniform bearing on the shoes. 4.4.2.2.3 Pile Shoes and Lugs Pile shoes used to protect all types of piles when hard driving is expected and pile lugs used to increase the bearing capacity of steel piles shall be of the types shown on the plans and shall be used at the locations specified or ordered by the Engineer. Steel pile shoes shall be fabricated from cast steel conforming to ASTM A 27. Such pile shoes or lugs used at the option of the Contractor shall be of a type approved by the Engineer. 4.4.3 Driving Piles shall be driven to the minimum tip elevations and bearing capacity shown on the plans, specified in the special provisions or approved by the Engineer. Piles that heave more than 1/4 inch upward during the driving of adjacent piles shall be redriven.
4.4.3.1 Driving of Test Piles Test piles and piles for static load tests, when shown on the shall be-furnished to the lengths ordered and driven at the locations and to the elevations directed by the Engineer before other piles in the area represented by the test are ordered or driven. AU test piles shall be driven 496 HIGHWAY BRIDGES ing to check for heave. Any pile which heaves more than Y4 inch shall be redriven or jacked to the original elevation prior to testing. Unless otherwise specified in the contract, a minimum 3-day waiting period shall be observed between the driving of any anchor piles or the load test pile and the commencement of the load test. 4.4.5 Splicing of Piles 4.4.5.1 Steel Piles Full-length piles shall be used where practicable. If splicing is permitted, the method of splicing shall be as shown on the plans or as approved by the Engineer. The arc method of welding shall be preferred when splicing steel piles. Welding shall only be performed by certified welders. 4.4.5.2 Concrete Piles Concrete piles shall not be spliced, other than to produce short extensions as permitted herein, unless specifically allowed by the plans, the special provisions, or by the Engineer in writing.
Short extensions or "build-ups" may be added to the tops of reinforced concrete piles to correct for unanticipated events. After the driving is completed, the concrete at the end of the pile shall be cut away, leaving the reinforcing steel exposed for a length of 40 diameters. The final cut of the concrete shall be perpendicular to the axis of the pile. Reinforcement similar to that used in the pile shall be securely fastened to the projecting steel and the necessary form work shall be placed, care being taken to prevent leakage along the pile. The concrete shall be of not less than the quality used in the pile. Just prior to placing concrete, the top of the pile shall be thoroughly flushed with water, allowed to dry, then covered with a thin coating of neat cement, mortar, or other suitable bonding material. The forms shall remain in place not less than 7 days and shall then be carefully removed and the entire exposed surface of the pile finished as previously specified.
4.4.5.3 Timber Piles Timber piles shall not be spliced unless specifically allowed by the plans, special provisions, or by the Engineer in writing. 4.4.6 Defective Piles The procedure incident to the driving of piles shall not subject them to excessive and undue abuse producing 4.4.4.5 crushing and spalling of the concrete, injurious splitting, splintering and brooming of the wood, or excessive deformation of the steel. Manipulation of piles to force them into proper position, considered by the Engineer to be excessive, will not be permitted. Any pile damaged by reason of internal defects or by improper driving or driven out of its proper location or driven below the butt elevation fixed by the plans or by the Engineer shall be corrected at the Contractor's expense by one of the following methods approved by the Engineer for the pile in question: The pile shall be withdrawn and replaced by a new and, if necessary, a longer pile. A second pile shall be driven adjacent to the defective or low pile. The pile shall be spliced or built up as otherwise provided herein or a sufficient portion of the footing extended to properly embed the pile. All piles pushed up by the driving of adjacent piles or by any other cause shall be driven down again. All such remedial materials and work shall be furnished at the Contractor's expense. 4.4.7 Pie Cut-off 4.4.7.1 General All piles shall be cutoff to a true plane at the elevations required and anchored to the structure, as shown on the plans. All cutoff lengths of piling shall remain the property of the Contractor and shall be properly disposed of. 4.4.7.2 Timber Piles Timber piles which support timber caps or grillage shall be sawed to conform to the plane of the bottom of the superimposed structure. In general, the length of pile above the elevation of cutoff shall be sufficient to permit the complete removal of all material injured by driving, but piles driven to very nearly the cutoff elevation shall be carefully adzed or otherwise freed from all "broomed," splintered, or otherwise injured material. Immediately after making final cutoff on treated timber foundation piles, the cut area shall be given two liberal applications of preservative followed by a heavy application of coal-tar roofing cement or other approved sealer. Treated timber piles which will have the cutoff exposed in the structure shall have the cut area treated with three coats of a compatible preservative material meeting the requirements of AWPA Standard M4. A minimum time period of 2 hours shall elapse between each application. 4.5 DIVISION II-CONSTRUCTION 4.5 MEASUREMENT AND PAYMENT 4.5.1 Method of Measurement 4.5.1.1 Timber, Steel, and Concrete Piles 497 load tests made at the option of the Contractor will not be included in the quantity measured for payment. Anchor and test piles for load tests, whether incorporated into the permanent structure or not, will be measured as provided for Piles Furnished and Piles Driven and will be paid for under the appropriate pay item. 4.5.1.1.1 Piles Furnished The quantities of each type of pile to be paid for will be the sum of the lengths in feet of the piles, of the types and lengths indicated on the plans or ordered in writing by the Engineer, furnished in compliance with the material requirements of these specifications and stockpiled or, in the case of driven cast-in-place concrete piles, installed in good condition at the site of the work by the Contractor, and accepted by the Engineer. The footage of piles, including test piles, furnished by the Contractor to replace piles which were previously accepted by the Engineer, but were subsequently damaged prior to completion of the contract will not be included. When extensions of piles are necessary, the extension length ordered in writing by the Engineer will be included in the linear footage of piling furnished. 4.5.1.1.2 Piles Driven The quantities of driven piles of each type to be paid for will be the number of acceptable piles of each type that were driven. Preboring, jetting, or other methods used for facilitating pile driving procedures when either required or permitted will not be measured, and payment will be considered included in the unit price paid for the Piles Driven. 4.5.1.2 P i e Splices, Pile Shoes, and Pile Lugs When pile splices, protective pile tip shoes or soil shear lugs are shown on the plans, the number of pile splices, shoes, or lugs measured for payment will be those shown on the plans, or ordered in writing by the Engineer, and actually installed on piles used in the work. No payment will be made for splices, shoes, or lugs used at the option of the Contractor. When not shown on the plans or specified to be used, pile splices, shoes, or lugs ordered by the Engineer will be paid for as extra work. 4.5.1.3 Load Tests The quantity of load tests to be paid for will be the number of load tests completed and accepted, except that 4.5.2 Basis of Payment The quantities,determined as provided, will be paid for at the contract price per unit of measurement, respectively, for each of the general pay items listed below for each size and type of pile shown in the bid schedule. Pay Item Piles, Furnished Piles, Driven Test Piles, Furnished Test Piles, Driven Pile Load Test (Static) Pile Load Test (Dynamic) Splices Pile Shoes Pile Lugs Pay Unit Linear Foot Each Linear Foot Each Each Each Each Each Each Payment for furnishing piles includes full compensation for all costs involved in the furnishing and delivery of all piles, including steel shells for cast-in-place driven piles, to the project site and all costs involved in the furnishing and placing of concrete and reinforcing steel for cast-in-place concrete piles. Payment for driving piles includes full compensation for all costs involved in the actual driving and cutting off of piles and pile shells, and for all costs for which compensation is not provided for under other pay items involved with the furnishing of labor, equipment, and materials used to construct the piles as shown on the plans and as specified or ordered. When mobilization of plant and equipment for the project is not paid for separately, payment for driving piles also includes full compensation for the cost of mobilization of all equipment needed for the handling and driving piles after the piles have been delivered to the project site. Payment for load tests includes full compensation for providing labor, equipment, and materials needed to perform the load tests as specified. Payment under the appropriate pay items for pile splices, shoes, and lugs includes full compensation for all costs involved with furnishing all materials and performing the work involved with attaching or installing splices, shoes, or lugs to the piles. Section 5 DRILLED PILES AND SHAFTS 5.1 DESCRIPTION This work shall consist of constructing drilled foundation shafts, with or without bell footings, including the placing of reinforcing steel and concrete all in accordance with the plans, these specifications and the special provisions. 5.2 SUBMITTALS 5.2.1 Contractor Qualifications (Recommended where permitted by state law) (a) The Contractor shall have a minimum of 3 years experience in constructingshaft foundations of similar size, depth and site conditions within the past 5 years. Prior to shaft construction the Contractor shall submit written documentation of the three years experience to the Engineer for verification and acceptance. The submittal shall include at least three projects on which the Contractor has previously been engaged in shaft construction with satisfactory results. A brief description of each project and the owner's contact person's name and current phone number shall be included for each project listed. (b) On-site supervisors shall have a minimum 2 years experience in construction of shaft foundations, and drill operators shall have a minimum 1 year experience. Prior to the start of work, the Contractor shall submit a list identifying the on-site supervisors and drill operators who will be assigned on the project. The list shall contain a summary of each individual's experience. (c) The Engineer will approve or reject the Contractor's qualifications and field personnel within 10 working days after receipt of the submission. Work shall not be started on any shaft until the Contract

equipment,and procedures used by the Contractor are sufCicient to produce a shaft excavation which meets the requirements of the plans and specifications.
Production shaft construction shall not be started until the required test shaft@)has been successfully completed. The Contractor shall revise his methods and equipment as necessary at any time during the construction of the test shaft hole to satisfactorily complete the excavation. The location of the test shaft shall be as shown on the plans, or as directed by the Engineer. The diameter and depth of the test shaft excavation shall be the same diameter and depth as the production drilled shafts shown on the plans. The test shaft holes shall be filled with concrete in the same manner that production shafts will be constructed unless a different backfill material is shown on the plans. When the Contractor fails to satisfactorily demonstrate the adequacy of his methods, procedures or equipment, additional test shafts shall be provided at no additional cost to the Department, until a successful test shaft has been constructed in accordance with the Engineerapproved construction methods. When shown on the plans, the reaming of bells at specified test shaft holes will be required to establish the feasibility of belling in a specific soil strata. 5.4.16 Construction Tolerances The following construction tolerances shall be maintained in constructing drilled shafts. HIGHWAY BRIDGES 504 (a) Shafts shall be constructed so that the center at the top of the shaft is within the following tolerances: Shaft Diameter Tolerance 2'-0" or less 3'4" 4'-0" 5'-0" or larger 3" 3l/2" 4" 6" (b) Shafts shall be within 1.5% of plumb.
For rock excavation, the allowable tolerance can be increased to 2% max. (c) After all the shaft concrete is placed, the portion of the shaft reinforcing steel cage embedded in the shaft shall be no more than 1 inch above and no more than 3 inches below plan position, and shall be at least 1 inch below the top of the shaft. (d) The minimum diameter of the drilled shaft shall be the diameter shown on the plans for diameters 24 inches or less, and not more than 1 inch less than the diameter shown on the plans for diameters greater than 24 inches. The maximum shaft diameter shall be the diameter shown in the plans, plus 6 inches. (e) The bearing area of bells shall be excavated to the plan bearing area as a minimum. All other dimensions for the bells shall be as shown on the approved working drawings. (f) The top elevation of the shaft shall be within 2 iqches of the plan top of shaft elevation. (g) The bottom of the shaft excavation shall be normal to the axis of the shaft within 3/4 inch per foot of shaft diameter. During drilling or excavation of the shaft, the Contractor shall make frequent checks on the plumbness, alignment, and dimensions of the shaft. Any deviation exceeding the allowable tolerances shall be corrected with a procedure approved by the Engineer. Drilled shaft excavations constructed in such a manner that the concrete shaft cannot be completed within the required tolerances are unacceptable. Correction methods shall be submitted by the Contractor for the Engineer's approval. Approval will be obtained before continuing with the drilled shaft construction. Materials and work necessary to effect correction for out-of-tolerance drilled shaft excavations shall be furnished at no additional cost to the Department. 5.4.17 Integrity Testing When called for in the special provisions, the completed shaft will be subjected to nondestructivetesting to determine the extent of any defects that may be present in the shaft. Work and materials required for testing which are to be furnished by the Contractor shall be as shown on the plans or special provisions. 5.4.16 In the event testing discloses voids or discontinuities in the concrete which, as determined by the Engineer, indicate that the drilled shaft is not structurally adequate, the shaft shall be rejected. The Contractor shall repair, replace or supplement the defective work in a manner approved by the Engineer. The construction of additional drilled shafts shall be discontinued until the Contractor demonstrates the adequacy of the shaft construction method and any subsequent method changes to the satisfaction of the Engineer.Any additional work required as a result of shaft defects shall be at no additional cost to the Department. 5.5 DRILLED SHAFT LOAD TESTS When the contract documents include load testing, all tests shall be completed before construction of any production drilled shafts. The Contractor shall allow two weeks after the last load test for the analysis of the load test data by the Engineer before specified drilled shaft tip elevations will be provided for production shafts. The locations of load test shafts and reaction shafts, the maximum loads to be applied, the test equipment to be furnished by the Contractor, and the actual performance of the load testing shall be as shown on the plans or specified in the special provisions. After testing is completed, the test shafts and any reaction shafts, if not also to be used as production shafts, shall be cutoff at an elevation 3 feet below the finished ground surface. The portion of the shafts cutoff shall be disposed of by the Contractor in a manner approved by the Engineer.
NOTE: Load tests should generally be performed as a separate contract in advance of the bridge construction. 5.6 MEASUREMENT AND PAYMENT 5.6.1 Measurement 5.6.1.1 Drilled Shaft Drilled shafts, complete in place, will be measured by the linear foot for each size of shaft listed in the schedule of bid items.
Measurement will be along the centerline of the shaft based on the tip and shaft cut-off elevations shown on the plans or as ordered by the Engineer. 5.6.1.2 Bell Footings Bell footings will be measured by the cubic yard, computed by using the dimensions and shape specified on the plans or as revised by the Engineer. The bell shall consist of the volume outside the plan or authorized dimensions of the shaft, which will extend to the bottom of the bell for the purpose of measurement. 5.6.1.3 DMSION II-CONSTRUCTION 5.6.1.3 Test Shafts Test shafts of the specified diameter will be measured from the elevation of the ground at the time drilling begins, by the linear foot of acceptable test shaft drilled. 5.6.1.4 Test Bells Test bells will be measured by the cubic yard computed by using the dimensions specified in Article 5.6.1.2. 5.6.1.5 Exploration Holes Exploration holes will be measured by the linear foot measured from the bottom of shaft elevation to the bottom of the exploration hole, for each authorized hole drilled. 5.6.1.6 Permanent Casing Permanent casing will be measured by the linear foot for each size of casing authorized to be used. Measurementwill be along the casing from top of casing or top of shaft, whicheveris lower, to the bottom of the casing at each shaft location where permanent casing is authorized and used. 5.6.1.7 Load Tests Load tests will be measured by the number of load tests performed for each designated pile load capacity. 505 equipment and incidentals necessary to complete the bell footings. 5.6.2.3 Test Shafts Test shafts of the specified diameter will be paid for at the contract unit price per linear foot for test shafts. Such payment shall be full compensation for excavation and concrete or backfill material including all labor, materials, equipment, and incidentals necessary to complete the test shafts. 5.6.2.4 Test Bells Test bells of the diameter and shape specified or authorized and approved will be paid for at the contract unit price per cubic yard for test bells.
Such payment shall be full compensation for excavation and concrete or backfill material including all labor, materials, equipment, and incidentals necessary to complete the test bells, except for unexpected obstructions. 5.6.2.5 Exploration Holes When specified or shown in the plans, exploration holes for soil samples or rock cores will be paid for at the contract unit price per linear foot for exploration holes. Such payment shall be full compensation for drilling or coring the holes, extracting and packaging the samples or cores and delivering them to the Department and all other expenses necessary to complete the work. 5.6.2 Payment 5.6.2.6 Permanent Casing 5.6.2.1 Drilled Shaft Drilled shafts will be paid for at the contract price per lineal foot for drilled shaft of the diameter specified. Such payment shall be considered to be full compensationfor all costs involved with shaft excavation, disposal of excavated material, and the furnishing and placing of concrete and reinforcing steel, including all labor, materials, equipment, temporary casing, and incidentals necessary to complete the drilled shafts, except for unexpected obstructions. 5.6.2.2 Bell Footings Bell footings constructed to the specified or authorized dimensions will be paid for at the contract unit price per cubic yard for bell footings. Such payment shall be full compensation for excavation, and concrete beyond the diameter of the drilled shaft including all labor, materials, Permanent casing will be paid for at the contract unit price per linear foot for permanent casing. Such payment shall be full compensation for furnishing and placing the casing above the costs attributable to the work paid for under associated pay items. 5.6.2.7 Load Tests Load tests will be paid for at the contract unit price for each load test. Such payment shall be full compensation for all costs related to the performance of the load tests. 5.6.2.8 Unexpected Obstructions Removal of unexpected obstructions will be paid for by force account. Section 6 GROUND ANCHORS submittal. No work on ground anchors shall begin until working drawings have been approved in writing by the Engineer. Such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work. 6.1 DESCRIPTION This work shall consist of designing, furnishing, installing, testing, and stressing permanent cement-grouted ground anchors in accordance with the plans, these specifications, and the special provisions. 6.2 WORKING DRAWINGS 6.3 MATERIALS At least 4 weeks before work is to begin, the Contractor shall submit to the Engineer for review and approval complete working drawings and design calculations describing the ground anchor system or systems intended for use. The submittal shall include the following: 6.3.1 Prestressing Steel Ground anchor tendons shall consist of single or multiple elements of prestressing steel, anchorage devices and, if required, couplers conforming to the requirements described in Section 10, "Prestressing." The following materials are acceptable for use as ground anchor tendons: (1) A ground anchor schedule giving: (a) Ground anchor number, (b) Ground anchor design load, (c) Type and size of tendon, (d) Minimum total anchor length, (e) Minimum bond length, (f) Minimum tendon bond length, and (g) Minimum unbonded length.
(2) A drawing of the ground anchor tendon and the corrosion protection system, including details for the following: (a) Spacers separating elements of tendon and their location, (b) Centralizers and their location, (') Unbonded length P~~~~~~~~ (d) Bond length corrosion protection system, (e) Anchorage and trumpet, (f) Anchorage corrosion protection system, (g) Drilled or formed hole size, (h) Level of each stage of grouting, and (i) Any revisions to structure details necessary to accommodate the ground anchor system intended for use. (3) The grout mix design and procedures for placing the grout. AASHTO Designation: M 203 (ASTM Designation A 416 - Uncoated, 7-wire strand) ASTM Designation: A 886lA886M (Indented, 7-wire strand) ASTM Designation: A 882lA 882M (Epoxy coated, 7-wire strand) 6.3.2 Grout Cement shall be Type I, 11, or III Portland Cement conforming to AASHTO M 85. Cement used for grouting shall be fresh and shall not contain any lumps or other indications of hydration or "pack set." Aggregate shall conform to the requirements for fine aggregate described in Section 8, "Concrete Structures." Admixtures may be used in the grout subject to the approval of the Engineer. Expansive admixtures may only be added to the grout used for filling sealed encapsulations, trumpets, and anchorage covers. Accelerators shall not be used. Water for mixing grout shall be potable, clean and free of injurious quantities of substances known to be harmful to Portland cement or prestressing steel. The Engineer will approve or reject the Contractor's working drawings within 4 weeks of receipt of a complete 507 508 HIGHWAY BRIDGES 6.3.3 Steel Elements Bearing plates shall be fabricated from steel conforming to AASHTO M 270 (ASTM A 709) Grade 36 minimum, or be a ductile iron casting conforming to ASTM A 536. Trumpets used to provide a transition from the anchorage to the unbonded length corrosion protection shall be fabricated from a steel pipe or tube conforming to the requirements of ASTM A 53 for pipe or ASTM A 500 for tubing. Minimum wall thickness shall be 0.20 inches. Anchorage covers used to enclose exposed anchorages shall be fabricated from steel, steel pipe, steel tube, or ductile cast iron conforming to the requirements of AASHTO M 270 (ASTM A 709) Grade 36 for steel, ASTM A 53 for pipe, ASTM A 500 for tubing, and ASTM A 536 for ductile cast iron. Minimum thickness shall be 0.10 inches. 6.3.4 Corrosion Protection Elements Corrosion inhibiting grease shall conform to the requirements of the Post Tensioning Institute's "Specifications for Unbonded Single Strand Tendons," Section 3.2.5. Sheath for the unbonded length of a tendon shall consist of one of the following: (1) Seamless polyethylene (PE) tube having a minimum wall thickness of 60 mils plus or minus 10 mils. The polyethylene shall be cell classification 334413 by ASTM D 3350. (2) Seamless polypropylene tube having a minimum wall thickness of 60 mils plus or minus 10 mils. The polypropylene shall be cell classification PP210B55542-11 by ASTM D 4101. (3) Heat shrinkable tube consisting of a radiation crosslinked polyolefin tube internally coated with an adhesive sealant. The minimum tube wall thickness before shrinking shall be 24 mils. The minimum adhesive sealant thickness shall be 20 mils. (4) Corrugated polyvinyl chloride (PVC) tube having a minimum wall thickness of 30 mils. Encapsulation for the tendon bond length shall consist of one of the following: (1) Corrugated high density polyethylene (HDPE) tube having a minimum wall thickness of 30 mils and conforming to AASHTO M 252 requirements. (2) Deformed steel tube or pipe having a minimum wall thickness of 25 mils. (3) Corrugated polyvinyl chloride (PVC) tube having a minimum wall thickness of 30 mils. 6.3.3 (4) Fusion-bonded epoxy conforming to the requirements of AASHTO M 284, except that it shall have a film thickness of 15 mils. 6.3.5 Miscellaneous Elements Bondbreaker for a tendon shall consist of smooth plastic tube or pipe that is resistant to aging by ultra-violet light and that is capable of withstanding abrasion, impact and bending during handling and installation. Spacers for separation of elements of a multi-element tendon shall permit the free flow of grout. They shall be fabricated from plastic, steel, or material which is not detrimental to the prestressing steel. Wood shall not be used. Centralizers shall be fabricated from plastic, steel, or material which is not detrimental to either the prestressing steel or any element of the tendon corrosion protection. Wood shall not be used. The centralizer shall be able to maintain the position of the tendon so that a minimum of 0.5 inches of grout cover is obtained on the tendons, or over the encapsulation. 6.4 FABRICATION Tendons for ground anchors may be either shop or field fabricated from materials conforming to the requirements of Article 6.3. Tendons shall be fabricated as shown on the approved working drawings. The tendon shall be sized so that the maximum test load does not exceed 80% of the minimum guaranteed ultimate strength of the tendon.
6.4.1 Bond Length and Tendon Bond Length The Contractor shall determine the bond length necessary to satisfy the load test requirements. The minimum bond length shall be 10 feet in rock, 15 feet in soil or the minimum length shown on the plans. The minimum tendon bond length shall be 10 feet. 6.4.1.1 Grout Protected Ground Anchor Tendon Spacers shall be placed along the tendon bond length of multi-element tendons so that the prestressing steel will bond to the grout. They shall be located at 10-foot maximum centers with the upper one located a maximum of 5 feet from the top of the tendon bond length and the lower one located a maximum of 5 feet from the bottom of the tendon bond length. Centralizers shall be placed along the bond length. They shall be located at 10-foot maximum centers with the upper one located a maximum of 5 feet from the top of the bond length and the lower one located 1 foot from the bot- 6.4.1.1 DIVISION II-CONSTRUCTION tom of the bond length. Centralizers are not required on tendons installed utilizing a hollow-stem auger if it is grouted through the auger and the drill hole is maintained full of a stiff grout (9-inch slump or less) during extraction of the auger. A combination centralizer-spacer may be used. Centralizers are not required on tendons installed utilizing a pressure injection system in coarse-grained soils using grouting pressures greater than 150 psi. 6.4.1.2 Encapsulation Protected Ground Anchor Tendon The tendon bond length shall be encapsulated by a grout-filled corrugated plastic or deformed steel tube, or by a fusion-bonded epoxy coating. The tendon can be grouted inside the encapsulation prior to inserting the tendon in the drill hole or after the tendon has been placed in the drill hole. Punching holes in the encapsulation and allowing the grout to flow from the encapsulation to the drill hole, or vice versa, will not be permitted. The tendon shall be centralized within the encapsulation and the tube sized to provide an average of 0.20 inches of grout cover for the prestressing steel. Spacers and centralizers shall be used to satisfy the same requirements specified in Article 6.4.1.1 for grout protected ground anchor tendons. The anchorage device of tendons protected with fusion-bonded epoxy shall be electrically isolated from the structure.
6.4.2 Unbonded Length The unbonded length of the tendon shall be a minimum of 15 feet or as indicated on the plans or approved working drawings. Corrosion protection shall be provided by a sheath completely filled with corrosion inhibiting grease or grout, or a heat shrinkable tube. If grease is used to fill the sheath, provisions shall be made to prevent it from escaping at the ends. The grease shall completely coat the tendon and fill the interstices between the wires of sevenwire strands. Continuity of corrosion protection shall be provided at the transition from the bonded length to unbonded length of the tendon. If the sheath provided is not a smooth tube, then a separate bondbreaker must be provided to prevent the tendon from bonding to the anchor grout surrounding the unbonded length. 6.43 Anchorage and Trumpet Nonrestressable anchorages may be used unless restressable anchorages are designated on the plans or spec- 509 ified in the special provisions. Bearing plates shall be sized so that the bending stresses in the plate and average bearing stress on the concrete, if applicable, do not exceed the allowable stresses described in Division I, Article 9.21.7.2, "Bearing Strength." The size of bearing plates shall not be less than that shown on the plans or on the approved working drawings. The trumpet shall be welded to the bearing plate. The trumpet shall have an inside diameter at least 0.25 inches greater than the diameter of the tendon at the anchorage. The trumpet shall be long enough to accommodate movements of the structure during testing and stressing. For strand tendons with encapsulation over the unbonded length, the trumpet shall be long enough to enable the tendons to make a transition from the diameter of the tendon in the unbonded length to the diameter of the tendon at the anchorhead without damaging the encapsulation. Trumpets filled with corrosion-inhibiting grease shall have a permanent Buna-N rubber or approved equal seal provided between the trumpet and the unbonded length corrosion protection. Trumpets filled with grout shall have a temporary seal provided between the trumpet and the unbonded length corrosion protection. 6.4.4 Tendon Storage and Handling Tendons shall be stored and handled in such a manner as to avoid damage or corrosion.
Damage to tendon prestressing steel as a result of abrasions, cuts, nicks, welds and weld splatter will be cause for rejection by the Engineer. Grounding of welding leads to the prestressing steel is not permitted. A slight rusting, provided it is not sufficient to cause pits visible to the unaided eye, shall not be cause for rejection. Prior to inserting a tendon into the drilled hole, its corrosion protection elements shall be examined for damage. Any damage found shall be repaired in a manner approved by the Engineer. 6.5 INSTALLATION The Contractor shall select the drilling method, the grouting procedure and grouting pressure to be used for the installation of the ground anchor as necessary to satisfy the load test requirements. 6.5.1 Drilling The drilling method used may be core drilling, rotary drilling, percussion drilling, auger drilling or driven casing. The method of drilling used shall prevent loss of ground above the drilled hole that may be detrimental to 510 HIGHWAY BRIDGES the structure or existing structures. Casing for anchor holes, if used, shall be removed, unless permitted by the Engineer to be left in place. The location, inclination, and alignment of the drilled hole shall be as shown on the plans. Inclination and alignment shall be within plus or minus 3" of the planned angle at the bearing plate, and within plus or minus 12 inches of the planned location at the ground surface (point of entry). 6.5.2 Tendon Insertion The tendon shall be inserted into the drilled hole to the desired depth without difficulty. When the tendon cannot be completely inserted it shall be removed and the drill hole cleaned or redrilled to permit insertion. Partially inserted tendons shall not be driven or forced into the hole. 6.53 Grouting A neat cement grout or sand-cement grout conforming to Article 6.3.2 shall be used. Admixtures, if used, shall be mixed in quantities not to exceed the manufacturer's recommendations. The grouting equipment shall produce a grout free of lumps and undispersed cement. A positive displacement grout pump shall be used. The pump shall be equipped with a pressure gauge to monitor grout pressures. The pressure gauge shall be capable of measuring pressures of at least 150 psi or twice the actual grout pressures used, whichever is greater. The grouting equipment shall be sized to enable the grout to be pumped in one continuous operation. The mixer shall be capable of continuously agitating the grout. The grout shall be injected from the lowest point of the drill hole. The grout may be pumped through grout tubes, casing, hollow-stem augers or drill rods. The grout may be placed before or after insertion of the tendon. The quantity of the grout and the grout pressures shall be recorded. The grout pressures and grout takes shall be controlled to prevent excessive heave of the ground or fracturing of rock formations. Except where indicated below, the grout above the top of the bond length may be placed at the same time as the bond length grout, but it shall not be placed under pressure. The grout at the top of the drill hole shall stop 6 inches from the back of the structure or from the bottom of the trumpet, whichever is lowest. If the ground anchor is installed in a fine-grained soil using a drilled hole lxger than 6 inches in diameter, then the grout above the top of the bond length shall be placed after the ground anchor has been load tested. The entire drill hole may be grouted at the same time if it can be 6.5.1 demonstrated that the ground anchor system does not derive a significant portion of its load resistance from the soil above the bond length portion of the ground anchor. If grout protected tendons are used for ground anchors anchored in rock, then pressure grouting techniques shall be utilized. Pressure grouting requires that the drill hole be sealed and that the grout be injected until a 50-psi grout pressure can be maintained on the grout within the bond length for a period of 5 minutes. Upon completion of grouting, the grout tube may remain in the drill hole provided it is filled with grout.
After grouting, the tendon shall not be loaded for a minimum of 3 days. 6.5.4 Trumpet and Anchorage The corrosion protection surrounding the unbonded length of the tendon shall extend into the trumpet a minimum of 6 inches beyond the bottom seal in the trumpet. The corrosion protection surrounding the unbonded length of the tendon shall not contact the bearing plate or the anchorhead during load testing or stressing. The bearing plate and anchorhead shall be placed perpendicular to the axis of the tendon. The trumpet shall be completely filled with corrosion inhibiting grease or grout. The grease may be placed any time during construction. The grout shall be placed after the ground anchor has been load tested. The Contractor shall demonstrate that the procedures selected for placement of either grease or grout will produce a completely filled trumpet. Anchorages not encased in concrete shall be covered with a corrosion inhibiting grease-filled or grout-filled steel enclosure. 6.5.5 Testing and Stressing Each ground anchor shall be load tested by the Contractor. No load greater than 10% of the design load may be applied to the ground anchor prior to load testing.
The test load shall be simultaneously applied to the entire tendon. 6.5.5.1 Testing Equipment A dial gauge or vernier scale capable of measuring displacements to 0.001 inches shall be used to measure ground anchor movement. It shall have adequate travel so total ground anchor movement can be measured without resetting the device. A hydraulic jack and pump shall be used to apply the test load. The jack and a calibrated pressure gauge shall be used to measure the applied load. The pressure gauge 6.5.5.1 DIVISION 11--CONSTRUCTION shall be graduated in 100-psi increments or less. When the theoretical elastic elongation of the total anchor length at the maximum test load exceeds the ram travel of the jack, the procedure for recycling the jack ram shall be included in the working drawings. Each increment of test load shall be applied as rapidly as possible. A calibrated reference pressure gauge shall be available at the site. The reference gauge shall be calibrated with the test jack and pressure gauge. An electrical resistance load cell and readout shall be provided when performing a creep test. The stressing equipment shall be placed over the ground anchor tendon in such a manner that the jack, bearing plates, load cell and stressing anchorage are axially aligned with the tendon and the tendon is centered within the equipment. 6.5.5.2 Performance Test Five percent of the ground anchors or a minimum of three ground anchors, whichever is greater shall be performance tested in accordance with the following procedures. The Engineer shall select the ground anchors to be performance tested. The remaining anchors shall be tested in accordance with the proof test procedures. The performance test shall be made by incrementally loading and unloading the ground anchor in accordance with the following schedule unless a different maximum test load and schedule are indicated on the plans. The load shall be raised from one increment to another immediately after recording the ground anchor movement. The ground anchor movement shall be measured and recorded to the nearest 0.001 inches with respect to an independent fixed reference point at the alignment load and at each increment of load. The load shall be monitored with a pressure gauge. The reference pressure gauge shall be placed in series with the pressure gauge during each performance test. If the load determined by the reference pressure gauge and the load determined by the pressure gauge differ by more than lo%, the jack, pressure gauge and reference pressure gauge shall be recalibrated. At load increments other than the maximum test load, the load shall be held just long enough to obtain the movement reading.
Performance Test Schedule Load Load 511 AL 0.25DL 0.50DL 0.75DL 1.00DL 1.20DL 1.33DL* (Max.
test load) Reduce to lock-off load (Art. 6.5.5.6) Where: AL = Alignment load DL = Design load for ground anchor * = Graph required.
See last paragraph in this Article 6.5.5.2. The maximum test load in a performance test shall be held for 10 minutes.
The jack shall be repumped as necessary in order to maintain a constant load. The load-hold period shall start as soon as the maximum test load is applied and the ground anchor movement shall be measured and recorded at 1 minute, 2, 3 , 4 , 5 , 6 , and 10 minutes. If the ground anchor movements between 1 minute and 10 minutes exceeds 0.04 inches, the maximum test load shall be held for an additional 50 minutes. If the load hold is extended, the ground anchor movement shall be recorded at 15 minutes, 20,25,30,45, and 60 minutes. A graph shall be constructed showing a plot of ground anchor movement versus load for each load increment marked with an asterisk (*) in the performance test schedule and a plot of the residual ground anchor movement of the tendon at each alignment load versus the highest previously applied load. Graph format shall be approved by the Engineer prior to use. 6.5.5.3 Proof Test The proof test shall be performed by incrementally loading the ground anchor in accordance with the following schedule unless a different maximum test load and schedule are indicated on the plans. The load shall be raised from one increment to another immediately after recording the ground anchor movement. The ground anchor movement shall be measured and recorded to the nearest 0.001 inches with respect to an independent fixed reference point at the alignment load and at each increment of load. The load shall be monitored with a pressure gauge. At load increments other than the maximum test load, the load shall be held just long enough to obtain the movement reading. 512 HIGHWAY BRIDGES Proof Test Schedule Load Load - AL 0.25DL 0.50DL 1.00DL 1.20DL 1.33DL (Max.
test load) Reduce to lock-off load 0.75DL where: AL = Alignment load DL = Design load for ground anchor The maximum test load in a proof test shall be held for 10 minutes. The jack shall be repumped as necessary in order to maintain a constant load. The loadhold period shall start as soon as the maximum test load is applied and the ground anchor movement shall be measured and recorded at 1 minute, 2, 3,4, 5, 6, and 10 minutes. If the ground anchor movement between 1 minute and 10 minutes exceeds 0.04 inches, the maximum test load shall be held for an additional 50 minutes. If the load hold is extended, the ground anchor movement shall be recorded at 15 minutes, 20, 30, 45, and 60 minutes. A graph shall be constructed showing a plot of ground anchor movement versus load for each load increment in the proof test. Graph format shall be approved by the Engineer prior to use. 6.5.5.4 Creep Test Creep tests shall be performed if required by the plans or special provisions. The Engineer shall select the ground anchors to be creep tested. The creep test shall be made by incrementally loading and unloading the ground anchor in accordance with the performance test schedule used. At the end of each loading cycle, the load shall be held constant for the observation period indicated in the creep test schedule below unless a different maximum test load is indicated on the plans. The times for reading and recording the ground anchor movement during each observation period shall be 1 minute, 2,3,4,5,6,10,15,20,25,30,45,60,75,90,100, 120, 150, 180,210, 240, 270, and 300 minutes as appropriate. Each load-hold period shall start as soon as the test load is applied. In a creep test the pressure gauge and reference pressure gauge will be used to measure the applied load, and the load cell will be used to monitor small changes of load during a constant load-hold period. The jack shall be repumped as necessary in order to maintain a constant load. 6.5.5.3 Creep Test Schedule Observation Period (Minutes) A graph shall be constructed showing a plot of the ground anchor movement and the residual movement measured in a creep test as described for the performance test. Also, a graph shall be constructed showing a plot of the ground anchor creep movement for each load hold as a function of the logarithm of time. Graph formats shall be approved by the Engineer prior to use. 6.5.5.5 Ground Anchor Load Test Acceptance Criteria A performance-tested or proof-tested ground anchor with a 10-minute load hold is acceptable if the: (1) Ground anchor resists the maximum test load with less than 0.04 inches of movement between 1 minute and 10 minutes; and (2) Total movement at the maximum test load exceeds 80% of the theoretical elastic elongation of the unbonded length. (3) Total movement at the maximum test load may not exceed the theoretical elastic elongation of the unbonded length plus 50% of the theoretical elastic elongation of the bonded length. [Criterion (3) applies only for a performance-tested ground anchor in competent rock.] A performance-tested or proof-tested ground anchor with a 60-minute load hold is acceptable if the: (1) Ground anchor resists the maximum test load with a creep rate that does not exceed 0.08 inches in the last log cycle of time; and (2) Total movement at the maximum test load exceeds 80% of the theoretical elastic elongation of the unbonded length. (3) Total movement at the maximum test load may not exceed the theoretical elastic elongation of the unbonded length plus 50% of the theoretical elastic elongation of the bonded length. [Criterion (3) applies only for a performance-tested ground anchor in competent rock.] 6.5.5.5 DIVISION 11--CONSTRUCTION A creep-tested ground anchor is acceptable if the: (1) Ground anchor carries the maximum test load with a creep rate that does not exceed 0.08 inches in the last log cycle of time; and (2) Total movement at the maximum test load exceeds 80% of the theoretical elastic elongation of the unbonded length. (3) Total movement at the maximum test load may not exceed the theoretical elastic elongation of the unbonded length plus 50% of the theoretical elastic elongation of the bonded length. [Criterion (3) applies only for a performance-tested ground anchor in competent rock.] If the total movement of the ground anchor at the maximum test load does not exceed 80% of the theoretical elastic elongation of the unbonded length, the ground anchor shall be replaced at the Contractor's expense. A ground anchor which has a creep rate greater than 0.08 inches per log cycle of time can be incorporated into the structure at a design load equal to one-half of its failure load. The failure load is the load resisted by the ground anchor after the load has been allowed to stabilize for 10 minutes.
When a ground anchor fails, the Contractor shall modify the design and/or the installation procedures. These modifications may include, but are not limited to, installing a replacement ground anchor, reducing the design load by increasing the number of ground anchors, modifying the installation methods, increasing the bond length or changing the ground anchor type. Any modification which requires changes to the structure shall be approved by the Engineer.
Any modifications of design or con- 513 struction procedures shall be without additional cost to the Department and without extension of contract time. Retesting of a ground anchor will not be permitted, except that regrouted ground anchors may be retested. 6.5.5.6 Lock Off Upon successful completion of the load testing, the ground anchor load shall be reduced to the lock-off load indicated on the plans and transferred to the anchorage device. The ground anchor may be completely unloaded prior to lock-off.
After transferring the load and prior to removing the jack, a lift-off load reading shall be made. The lift-off load shall be within 10% of the specified lock-off load. If the load is not within 10% of the specified lock-off load, the anchorage shall be reset and another lift-off load reading shall be made. This process shall be repeated until the desired lock-off load is obtained. 6.6 MEASUREMENT AND PAYMENT Ground anchors will be measured and paid for by the number of units installed and accepted as shown on the plans or ordered by the Engineer. No change in the number of ground anchors to be paid for will be made because of the use by the Contractor of an alternative number of ground anchors. The contract unit price paid for ground anchors shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in installing the ground anchors (including testing), complete in place, as shown on the plans and as specified in these specifications and the special provisions, and as directed by the Engineer. Section 7 EARTH RETAINING SYSTEMS 7.1 DESCRIPTION This work shall consist of furnishing and installing earth retaining systems in accordance with the plans, these specifications, and the special provisions. Earth retaining systems include concrete and masonry gravity walls, reinforced concrete retaining walls, sheet pile and soldier pile walls (with and without ground anchors or other anchorage systems), crib and cellular walls, and mechanically stabilized earth walls. 7.2 WORKING DRAWINGS Working drawings and design calculations shall be submitted to the Engineer for review and approval at least 4 weeks before work is to begin. Such submittals shall be required (1) for each alternative proprietary or nonproprietary earth retaining system proposed as permitted or specified in the contract, (2) when complete details for the system to be constructed are not included in the plans, and (3) when otherwise required by the special provisions or these specifications. Working drawings and design calculations shall include the following: (a) Existing ground elevations that have been verified by the Contractor for each location involving construction wholly or partially in original ground.
(b) Layout of wall that will effectively retain the earth but not less in height or length than that shown for the wall system in the plans. (c) Complete design calculations substantiating that the proposed design satisfies the design parameters in the plans and in the special provisions. (d) Complete details of all elements required for the proper construction of the system, including complete material specifications. (e) Earthwork requirements including specifications for material and compaction of backfill. (f) Details of revisions or additions to drainage systems or other facilities required to accommodate the system. (g) Other information required in the plans or special provisions or requested by the Engineer. The Contractor shall not start work on any earth retaining system for which working drawings are required until such drawings have been approved by the Engineer. Approval of the Contractor's working drawings shall not relieve the Contractor of any of his responsibility under the contract for the successful completion of the work. 7.3 MATERIALS 7.3.1 Concrete 7.3.1.1 Cast-in-Place Cast-in-place concrete shall conform to the requirements of Section 8, "Concrete Structures." The concrete shall be Class A unless otherwise indicated in the contract documents. 7.3.1.2 Pneumatically Applied Mortar Pneumatically applied m o m shall conform to the requirements of Section 24, "Pneumatically Applied Mortar." 7.3.1.3 Precast Elements The materials, manufacturing, storage, handling, and erection of precast concrete elements shall conform to the requirements in Article 8.13, "Precast Concrete Members." Unless otherwise shown on the plans or on the approved working drawings, Portland cement concrete used in precast elements shall conform to Class A (AE) with a minimum compressive strength at 28 days of 4,000 psi. 7.3.1.4 Segmental Concrete Facing Blocks Masonry concrete blocks used as wall facing elements shall have a minimum compressive strength of 4,000 psi and a water absorption limit of 5%. In areas of repeated freeze-thaw cycles, the facing blocks shall be tested in accordance with ASTM C 1262 to demonstrate durability. The facing blocks shall meet the requirements of ASTM 516 HIGHWAY BRIDGES C 1372, except that acceptance regarding durability under this testing method shall be achieved if the weight loss of each of 4 of the 5 specimens at the conclusion of 150 cycles does not exceed 1 % of its initial weight. Blocks shall also meet the additional requirements of ASTM C 140. Facing blocks directly exposed to spray from deiced pavements shall be sealed after erection with a water resistant coating or be manufactured with a coating or additive to increase freeze-thaw resistance. 7.3.2 Reinforcing Steel Reinforcing steel shall conform to the requirements of Section 9, "Reinforcing Steel." 7.3.3 Structural Steel Structural steel shall conform to AASHTO M 270 (ASTM A 709) Grade 36 unless otherwise specified. 7.3.4 Timber Timber shall conform to the requirements of Section 16, "Timber Structures"and Article 4.2.2, "Timber Piles." 7.3.5 Drainage Elements 7.3.5.1 Pipe and Perforated Pipe Pipe and perforated pipe shall conform to subsections 708 and 709 of the AASHTO Guide SpeciJications for Highway Construction. 7.3.5.2 Geotextile Geotextile shall conform to AASHTO M 288. 7.3.5.3 Permeable Material Permeable material shall conform to subsection 704 of the AASHTO Guide SpeciJications for Highway Construction unless otherwise specified in the contract or the approved working drawings.
7.3.5.4 Geocomposite Drainage Systems Geocomposite drainage systems shall conform to the requirements specified in the special provisions or the approved working drawings.
7.3.1.4 7.3.6 Structure Backfill Material 7.3.6.1 General All structure backfill material shall consist of material free from organic material or other unsuitable material as determined by the Engineer. Gradation will be determined by AASHTO T 27. Grading shall be as follows unless otherwise specified. Sieve Size Percent Passing 3 in. NO. 4 No. 30 No. 200 100 35-100 20-100 0-15 7.3.6.2 Crib and Cellular Walls Structure backfill material for crib and cellular walls shall be of such character that it will not sift or flow through openings in the wall. For wall heights over 20 feet (6 meters), the following grading shall be required: Sieve Size Percent Passing 3 in. NO. 4 NO. 30 No. 200 100 25-70 5-20 0-5 7.3.6.3 Mechanically Stabilized Earth Walls Structure backfill material for MSE walls shall consist of material free from organic material or other unsuitable material as determined by the Engineer. Gradation shall be determined by AASHTO T 27. Grading shall be as follows unless otherwise specified: Sieve Size Percent Passing 4 in.+ No. 40 No. 200 100 0-60 0-15* +For the soil to be considered to be nonaggressive, the maximum soil particle size for geosynthetic reinforcement shall be 0.75 inches unless full scale installation damage tests are conducted in accordance with ASTM D 5818, or if epoxy coatings are used for steel reinforcements. "Plasticity index (PI), as determined by AASHTO T 90, shall not exceed 6. The material shall exhibit an angle of internal friction of not less than 34", as determined by the standard Di- 7.3.6.3 DIVISION II-CONSTRUCTION rect Shear Test, AASHTO T 236, on the portion finer than the No. 10 sieve, utilizing a sample of the material compacted to 95% of AASHTO T 99, Methods C or D (with oversized correction as outlined in Note 7) at optimum moisture content. No testing is required for backfills where 80% of the sizes are greater than 0.75 inches.
The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30% after four cycles or a sodium sulfate soundness loss of less than 15% after five cycles determined in accordance with AASHTO T 104. The soil shall also have an organic content of less than or equal to 1% measured in accordance with AASHTO T 267 for material finer than the No. 10 sieve. The soil backfill electrochemicalrequirements for steel soil reinforcement are as follows: pH of 5 to 10 Resistivity of not less than 3,000 ohm-cm Chlorides not greater than 100 ppm Sulfates not greater than 200 ppm If the resistivity is greater than or equal to 5,000 ohm-cm, the chlorides and sulfates requirements may be waived. The soil backfill electrochemicalrequirements for permanent geosynthetic reinforcement are as follows: 517 foundation material is indicated, the Contractor shall perform the excavation to the limits shown. Materials excavated shall be replaced with structure backfill material meeting the requirements for the particular earth retaining system to be constructed unless a different material is specified in the special provisions. The material shall be compacted to a density not less than 95% of the maximum density as determined by AASHTO T 99, Methods C or D (with oversize correction as outlined in Note 7).
7.4.3 Structure Backfill Placement of structure backfill material shall conform to the requirements of Articles 1.4.3 and 7.6. Material used shall conform to the requirements of Article 7.3.6. 7.5 DRAINAGE Drainage facilities shall be constructed in accordance with the details shown on the plans or approved working drawings, the special provisions, and these Specifications. 7.5.1 Concrete Gutters Recommended test methods for soil chemical property determination include AASHTO T 289 for pH, AASHTO T 288 for resistivity, AASHTO T 291 for chlorides, and AASHTO T 290 for sulfates. Concrete gutters shall be constructed to the profile indicated on the plans or the approved working drawings. Pneumatically applied mortar shall conform to the requirements of Section 24, "

7.7 MEASUREMENT AND PAYMENT Unless otherwise designated in the special provisions, earth retaining systems will be measured and paid for by the square foot. The square meter (square foot) area for payment will be based on the vertical height and length of each section built except in the case when alternative earth retaining systems are permitted in the contract documents. When alternative earth retaining systems are permitted, the square meter (square foot area) for payment will be based on the vertical height and length of each section of the system type designated as the basis of payment whether or not it is actually constructed. The vertical height of each section will be taken as the difference in elevation on the outer face from the bottom of the lowermost face element for systems without footings, and from the top of footing for systems with footings, to the top of the wall, excluding any barrier. The contract price paid per square meter (square foot) for earth retaining systems shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in constructing the earth retaining systems including-but not limited to-earthwork, piles, footings, and drainage systems, complete in place as shown on the plans, as specified in these specifications and as directed by the Engineer. Full compensation for revisions to drainage system, or other facilities made necessary by the use of an alternative earth retaining system shall be considered as included in the contract price paid per square meter (square foot) for earth retaining system and no adjustmentin compensation will be made therefore. REFERENCES Elias, V., 1996, Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Federal Highway Administration, NO. FHWA-DP.82-2. Elias, V., and Christopher, B.R., 1996, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design 7.7 DIVISION 11-CONSTRUCTION and Construction Guidelines, Federal Highway Administration, No. FHWA-DP.82.1. Federal Highway Administration, 1991, Scour at Bridges, Technical Advisory, T 5150.20, U.S. Department of Transportation, Federal Highway Administration, 64 p. 523 Simac, M.
R., Bathurst, R. J., Berg, R. R., and Lothspeich, S. E., 1993, Design Manual for Segmental Retaining Walls (Modular Concrete Block Retaining Wall Systerns), First Edition, NCMA, Herndon, Virginia. Section 8 CONCRETE STRUCTURES 8.1 GENERAL quate arrangements will be provided for curing, finishing, and protecting the concrete. 8.1.1 Description This work shall consist of furnishing, placing, finishing, and curing concrete in bridges, culverts, and miscellaneous structures in accordance with these specifications and conforming to the lines, grades, and dimensions shown on the plans. The work includes elements of structures constructed by cast-in-place and precast methods using either plain (unreinforced), reinforced, or prestressed concrete or any combination thereof. The requirements of this section are not applicable to precast box culvert structures, which are addressed in Section 27. 8.2 CLASSES OF CONCRETE 8.2.1 General The class of concrete to be used in each part of the structure shall be as specified or shown on the plans.
If not shown or specified, the Engineer will designate the class of concrete to be used. 8.2.2 Normal Weight Concrete Eight classes of normal weight concrete are provided for in these Specifications as listed in Table 8.2. 8.1.2 Related Work 8.2.3 Lightweight Concrete Other work involved in the construction of concrete structures shall be as specified in the applicable sections of this Specification. Especially applicable are Section 3 for forms and falsework, Section 9 for reinforcing steel, and Section 10 for prestressing. Lightweight concrete shall conform to the requirements specified in the special provisions or shown on the plans. When the special provisions require the use of natural sand for a portion or all of the fine aggregate, the natural sand shall conform to AASHTO M 6. 8.1.3 Construction Methods 8.3 MATERIALS Whenever the specifications permit the Contractor to select the method or equipment to be used for any operation, it shall be the Contractor's responsibility to employ methods and equipment which will produce satisfactory work under the conditions encountered and which will not damage any partially completed portions of the work. Falsework and forms shall conform to the requirements of Section 3, "Temporary Works." Generally, all concrete shall be fully supported until the required strength and age has been reached. However, the slip form method will be permitted for the construction of pier shafts and railings providing the Contractor's plan assures that: (1) the results will be equal in all respect to those obtained by the use of fixed forms, and (2) ade- 8.3.1 Cements Portland cements shall conform to the requirements of AASHTO M 85 (ASTM C 150) and Blended Hydraulic cements shall conform to the requirements of AASHTO M 240 (ASTM C 595).
For Type 1PPortland-pozzolan cement, the pozzolan constituent shall not exceed 20% of the weight of the blend and the loss on ignition of the pozzolan shall not exceed 5%. Unless otherwise specified, only Type I, 11, or III Portland Cement, Types IA, IIA, or U A i r Entrained Portland Cement, or Types IP or IS Blended Hydraulic cements shall be used. 'Qpes IA, IIA, and IIIA cements may be used only in concrete where air entrainment is required. 526 HIGHWAY BRIDGES 8.3.1 TABLE 8.2 a Specified Compressive Strength (28 Days) Minimum Cement Content Maximum WaterICement Ratio Air Content Range Size of Coarse Aggregate Per AASHTO M 43 Class of Concrete Pounds Per CY Lbs Per Lb Percent Square Openings A A(AE) 611 611 0.4ga 0.45 6-+ 1-112 1 in. to No. 4 1 in.
to No. 4 4,000 4,000 B 517 0.58 - 2 in. to No. 3 and 1 in. to No. 4b 2,400 B(AE) 517 0.55 5 r 1-112 2 in. to No. 3 and 1 in. to No. 4b 2,400 C WE) 658 658 0.49 0.45 7 + 1-112 112 in. to No. 4 112 in. to No. 4 4,000 4,000 P 564 0.49" As specified elsewhere 1 in. to No. 4 or 314 in. to No. 4 As specified elsewhere S 658 0.58 - 1 in. to No. 4 - (Ib~in.~) For concrete in or over saltwateror exposed to deicing chemicals, the maximum waterlcement ratio shall be 0.45.
Coarse Aggregate for Class B and Class B(AE) shall be furnished in two separate sizes as shown. Low-alkali cements conforming to the requirementsof AASHTO M 85 for low-alkali cement shall be used when specified or when ordered by the Engineer as a condition of use for aggregates of limited alkali-silica reactivity. Unless otherwise permitted, the product of only one mill of any one brand and type of cement shall be used for like elements of a structure that are exposed to view, except when cements must be blended for reduction of any excessive air-entrainment where air-entraining cement is used. 8.3.2 Water Water used in mixing and curing of concrete shall be subject to approval and shall be reasonably clean and free of oil, salt, acid, alkali, sugar, vegetable, or other injurious substances. Water will be tested in accordance with, and shall meet the suggested requirements of AASHTO T 26.
Water known to be of potable quality may be used without test. Where the source of water is relatively shallow, the intake shall be so enclosed as to exclude silt, mud, grass, or other foreign materials. Mixing water for concrete in which steel is embedded shall not contain a chloride ion concentration in excess of 1,000 ppm or sulphates as SO4 in excess of 1,300 ppm. 8.3.3 Fine Aggregate Fine aggregate for concrete shall conform to the requirements of AASHTO M 6.
8.3-4 Charse Aggregate Coarse aggregate for concrete shall conform to the requirements of AASHTO M 80. 8.3.5 Lightweight Aggregate Lightweight aggregate for concrete shall conform to the requirements of AASHTO M 195 (ASTM C 330). 8.3.6 Air-Entraining and Chemical Admixtures Air-entraining admixtures shall conform to the requirements of AASHTO M 154 (ASTM C 260). Chemical admixtures shall conform to the requirements of AASHTO M 194 (ASTM C 494). Unless otherwise specified, only Type A (Water-reducing), Type B (Retarding), Type D (Water-reducingand retarding), Q p e F (Water-reducing, high range) or Type G (Water-reducing, high range and retarding) shall be used. 8.3.6 DIVISION II-CONSTRUCTION Admixtures containing chloride ion (Cl) in excess of 1% by weight of the admixture shall not be used in reinforced concrete. Admixtures in excess of 0.1% shall not be used in prestressed concrete. A Certificate of Compliance signed by the manufacturer of the admixture shall be furnished to the Engineer for each shipment of admixture used in the work. Said Certificateshall be based upon laboratory test results from an approved testing facility and shall certify that the admixture meets the above specifications. If more than one admixture is used, the admixtures shall be compatible with each other and shall be incorporated into the concrete mix in correct sequence so that the desired effects of all admixtures are obtained. Air-entraining and chemical admixtures shall be incorporated into the concrete mix in a water solution. The water so included shall be considered to be a portion of the allowed mixing water.
527 For normal weight concrete the absolute volume method, such as described in American Concrete Institute Publication 211.1, shall be used in selecting mix proportions. For structural lightweight concrete, the mix proportions shall be selected on the basis of trial mixes with the cement factor rather than the waterlcement ratio being determined by the specified strength using methods such as those described in American Concrete Institute Publication 211.2. The mix design shall be based upon obtaining an average concrete strength sufficiently above the specified strength so that, considering the expected variability of the concrete and test procedures, no more than 1 in 10 strength tests will be expected to fall below the specified strength. Mix designs shall be modified during the course of the work when necessary to ensure compliance with strength and consistency requirements. 8.4.1.2 Trial Batch Tests 8.3.7 Mineral Admixtures Fly ash pozzolans and calcined natural pozzolans for use as mineral admixtures in concrete shall conform to the requirements of AASHTO M 295 (ASTM C 618).
The use of fly ash as produced by plants that utilize the limestone injection process or use compounds of sodium, ammonium or sulphur, such as soda ash, to control stack emissions shall not be used in concrete. A Certificate of Compliance, based on test results and signed by the producer of the mineral admixture certifying that the material conforms to the above specifications, shall be furnished for each shipment used in the work. 8.3.8 Steel Materials and installation of reinforcing and prestressing steel shall conform to the requirements of Sections 9, "Reinforcing Steel," and 10, "Prestressing," respectively. For classes A, A(AE) and P concrete, for lightweight concrete, and for other classes of concrete when specified or ordered by the Engineer, satisfactory performanceof the proposed mix design shall be verified by laboratory tests on trial batches. The results of such tests shall be furnished to the Engineer by the Contractor or the manufacturer of precast elements at the time the proposed mix design is submitted. For mix design approval, the strengths of a minimum of five test cylinders taken from a trial batch shall average at least 800 psi greater than the specified strength. If materials and a mix design identical to those proposed for use have been used on other work within the previous year, certified copies of concrete test results from this work which indicate full compliance with these specifications may be substituted for such laboratory tests. If the results of more than 10 such strength tests are available from historical records for the past year, average strength for these tests shall be at least 1.28 standard deabove the strength. viations 8.4.1.3 Approval 8.4 PROPORTIONING OF CONCRETE 8.4.1 M i Design 8.4.1.1 Responsibility and Criteria The Contractor shall design and be responsible for the performance of all concrete mixes used in structures.
The mix proportions selected shall produce concrete that is sufficiently workable and finishable for all uses intended and shall conform to the requirements in Table 8.2 and all other requirements of this section. All n i x designs, and any modifications thereto, shall be approved by the Engineer prior to use. Mix design data provided to the Engineer for each class of concrete required shall include the name, source, type, and brand of each of the materials proposed for use and the quantity to be used per cubic yard of concrete. 8.4.2 Water Content For calculating the waterlcement ratio of the mix, the weight of the water shall be that of the total free water in 528 HIGHWAY BRIDGES the mix which includes the mixing water, the water in any admixture solutions and any water in the aggregates in excess of that needed to reach a saturated-surface-dry condition. The amount of water used shall not exceed the limits listed in Table 8.2 and shall be further reduced as necessary to produce concrete of the consistencies listed in Table 8.3 at the time of placement: 8.4.2 (AASHTO M 85) cements are used and mineral admixtures are neither specified nor prohibited, the Contractor will be permitted to replace up to 20% of the required Portland cement with a mineral admixture. The weight of the mineral admixture used shall be equal to or greater than the weight of the Portland cement replaced. In calculating the waterlcement ratio of the mix, the weight of the cement shall be considered to be the sum of the weights of the Portland cement and the mineral admixture. TABLE 8.3 5 p e of Work Formed Elements: Sections Over 12 in. Thick Sections 12 in. Thick or Less Cast-in-place Piles and Drilled Shafts not Vibrated Concrete Placed Under Water Filling for Rip-rap Nominal Slump Inches Maximum Slump Inches 1-3 5 1-4 5 5-8 5-8 3-7 9 9 8 8.4.5 Air-Entrainingand Chemical Admixtures Air-entraining and chemical admixtures shall be used as specified. Otherwise, such admixtures may be used, at the option and expense of the Contractor when permitted by the Engineer, to increase the workability or alter the time of set of the concrete. 8.5 MANUFACTURE OF CONCRETE When Type F or G high range water reducing admixtures are used, the above listed slump limits may be exceeded as permitted by the Engineer.
When the consistency of the concrete is found to exceed the nominal slump, the mixture of subsequent batches shall be adjusted to reduce the slump to a value within the nominal range. Batches of concrete with a slump exceeding the maximum specified shall not be used in the work. If concrete of adequate workability cannot be obtained by the use of the minimum cement content allowed, the cement and water content shall be increased without exceeding the specified waterlcement ratio, or an approved admixture shall be used. 8.4.3 Cement Content The minimum cement content shall be as listed in Table 8.2 or otherwise specified. The maximum cement or cement plus mineral admixture content shall not exceed 800 pounds per cubic yard of concrete. The actual cement content used shall be within these limits and shall be sufficient to produce concrete of the required strength and consistency. 8.4.4 Mineral Admixtures Mineral admixtures shall be used in the amounts specified. In addition, when either Types I, 11, IV, or V The production of ready-mixed concrete shall conform to the requirements of AASHTO M 157 (ASTM C 94) and the requirements of this Article 8.5. The production of concrete with stationary mixers shall conform to the applicable requirements of AASHTO M 157 (ASTM C 94) and the requirements of this article. 8.5.1 Storage of Aggregates The handling and storage of concrete aggregates shall be such as to prevent segregation or contamination with foreign materials. The methods used shall provide for adequate drainage so that the moisture content of the aggregates is uniform at the time of batching. Different sizes of aggregate shall be stored in separate stock piles sufficiently removed from each other to prevent the material at the edges of the piles from becoming intermixed. When specified in Table 8.2 or in the special provisions, the coarse aggregate shall be separated into two or more sizes in order to secure greater uniformity of the concrete mixture. 8.5.2 Storage of Cement The Contractor shall provide suitable means for storing and protecting cement against dampness. Cement which for any reason has become partially set or which contains lumps of caked cement will be rejected. Cement held in storage for a period of over 3 months if bagged or 6 months if bulk, or cement which for any reason the Engineer may suspect of being damaged, shall be subject to a retest before being used in the work. 8.5.2 DIVISION II-CONSTRUCTION Copies of cement records shall be furnished to the Engineer, showing, in such detail, as he may reasonably require, the quantity used during the day or run at each part of the work. 8.5.3 Measurement of Materials Materials shall be measured by weighing, except as otherwise specified or where other methods are specifically authorized.The apparatus provided for weighing the aggregates and cement shall be suitably designed and constructed for this purpose. Each size of aggregate and the cement shall be weighed separately. The accuracy of all weighing devices shall be such that successive quantities can be measured to within 1% of the desired amount. Cement in standard packages (sack) need not be weighed, but bulk cement shall be weighed. The mixing water shall be measured by volume or by weight. The accuracy of measuring the water shall be within a range of error of not over 1%. All measuring devices shall be subject to approval and shall be tested, at the Contractor's expense, when deemed necessary by the Engineer. When volumetric measurements are authorized for projects, the weight proportions shall be converted to equivalent volumetric proportions.
In such cases, suitable allowance shall be made for variations in the moisture condition of the aggregates, including the bulking effect in the fine aggregate.
When sacked cement is used, the quantities of aggregates for each batch shall be exactly sufficient for one or more full sacks of cement and no batch requiring fractional sacks of cement will be permitted. 8.5.4 Batching and Mixing Concrete 8.5.4.1 Batching The size of the batch shall not exceed the capacity of the mixer as guaranteed by the manufacturer or as determined by the Standard Requirements of the Associated General Contractors of America. The measured materials shall be batched and charged into the mixer by means that will prevent loss of any materials due to effects of wind or other causes. 8.5.4.2 Mixing The concrete shall be mixed only in the quantity required for immediate use. Mixing shall be sufficient to thoroughly intermingle all mix ingredients into a uniform mixture. Concrete that has developed an initial set shall not be used. Retempering concrete by adding water will not be permitted.
529 For other than transit mixed concrete, the first batch of concrete materials placed in the mixer shall contain a sufficient excess of cement, sand, and water to coat the inside of the drum without reducing the required mortar content of the mix. When mixer performance tests, as described in AASHTO M 157, are not made, the required mixing time for stationary mixers shall be not less than 90 seconds nor more than 5 minutes. The minimum drum revolutions for transit mixers at the mixing speed recommended by the manufacturer shall not be less than 70 and not less than that recommended by the manufacturer. The timing device on stationary mixers shall be equipped with a bell or other suitable warning device adjusted to give a clearly audible signal each time the lock is released. In case of failure of the timing device, the Contractor will be permitted to operate while it is being repaired, provided he furnishes an approved timepiece equipped with minute and second hands. If the timing device is not placed in good working order within 24 hours, further use of the mixer will be prohibited until repairs are made. For small quantities of concrete needed in emergencies or for small noncritical elements of the work, concrete may be hand-mixed using methods approved by the Engineer. Between uses, any mortar coating inside of mixing equipment which sets or dries shall be cleaned from the mixer before use is resumed. 8.5.5 Delivery The organization supplying concrete shall have sufficient plant capacity and transporting apparatus to ensure continuous delivery at the rate required. The rate of delivery of concrete during concreting operations shall be such as to provide for the proper handling, placing, and finishing of the concrete. The rate shall be such that the interval between batches shall not exceed 20 minutes and shall be s cient to prevent joints within a monolithic pour caused by placing fresh concrete against concrete in which initial set has occurred. The methods of delivering and handling the concrete shall be such as will facilitate placing with the minimum of rehandling and without damage to the structure or the concrete. 8.5.6 Sampling and Testing Compliance with the requirements indicated in this Section shall be determined in accordance with the following standard methods of AASHTO or ASTM: 530 HIGHWAY BRIDGES Sampling Fresh Concrete, AASHTO T 141 (ASTM C 172) Weight Per Cubic Foot, Yield and Air Content (Gravimetric) of Concrete, AASHTO T 121 (ASTM C 138) Sieve Analysis of Fine and Coarse Aggregate, AASHTO T 27 Slump of Portland Cement Concrete, AASHTO T 119 (ASTM C 143) Air Content of Freshly Mixed Concrete by the Pressure Method, AASHTO T 152 (ASTM C 231) Specific Gravity and Absorption of Fine Aggregate, AASHTO T 84 (ASTM C 128) Specific Gravity and Absorption of Coarse Aggregate, AASHTO T 85 (ASTM C 127) Unit Weight of Structural Lightweight Concrete, ASTM C 567 Making and Curing Concrete Test Specimens in the Laboratory, AASHTO T 126 (ASTM C 192) Making and Curing Concrete Test Specimens in the Field, AASHTO T 23 (ASTM C 31) CompressiveStrength of Cylindrical Concrete Specimens, AASHTO T 22 (ASTM C 39) 8.5.7 Evaluation of Concrete Strength 8.5.7.1 Tests A strength test shall consist of the average strength of two compressive strength test cylinders fabricated from material taken from a single randomly selected batch of concrete, except that, if any cylinder should show evidence of improper sampling, molding, or testing, said cylinder shall be discarded and the strength test shall consist of the strength of the remaining cylinder. 8.5.7.2 For Controlling Construction Operations For determining adequacy of cure and protection, and for determining when loads or stresses can be applied to concrete structures, test cylinders shall be cured at the structure site under conditions that are not more favorable than the most unfavorable conditions for the portions of the structure which they represent as described in Article 9.4 of AASHTO T 23. Sufficient test cylinders shall be made and tested at the appropriate ages to determine when operations such as release of falsework, application of prestressing forces or placing the structure in service can occur. 8.5.6 8.5.7.3 For Acceptance of Concrete For determining compliance of concrete with a specified 28-day strength, test cylinders shall be cured under controlled conditions as described in Article 9.3 of AASHTO T 23 and tested at the age of 28 days. Samples for acceptance tests for each class of concrete shall be taken not less than once a day nor less than once for each 150 cubic yards of concrete or once for each major placement. Any concrete represented by a test which indicates a strength which is less than the specified 28-day compressive strength by more than 500 psi will be rejected and shall be removed and replaced with acceptable concrete. Such rejection shall prevail unless either: (1) The Contractor, at own expense, obtains and submits evidence of a type acceptable to the Engineer that the strength and quality of the rejected concrete is acceptable. If such evidence consists of cores taken from the work, the cores shall be obtained and tested in accordance with the standard methods of AASHTO T 24 (ASTM C 42) or, (2) The Engineer determines that said concrete is located where it will not create an intolerable detrimental effect on the structure and the Contractor agrees to a reduced payment to compensate the Department for loss of durability and other lost benefits.
8.5.7.4 For Control of Mix Design Whenever the average of three consecutive tests, which were made to determine acceptability of concrete, falls to less than 150 psi above the specified strength or any single test falls more than 200 psi below the specified strength, the Contractor shall, at own expense, make corrective changes in the materials, mix proportions or in the concrete manufacturing procedures before placing additional concrete of that class. Such changes must be approved by the Engineer prior to use. 8.5.7.5 Steam and Radiant Heat-Cured Concrete When a precast concrete member is steam or radiant heat-cured, the compressive strength test cylinders made for any of the above purposes shall be cured under conditions similar to the member. Such concrete will be considered to be acceptable whenever a test indicates that the concrete has reached the specified 28-day compressive strength provided such strength is reached not more than 28 days after the member is cast. 8.6 DMSION 11--CONSTRUCTION 8.6 PROTECTION OF CONCRETE FROM ENVIRONMENTAL CONDITIONS 8.6.1 General Precautions shall be taken as needed to protect concrete from damage due to weather or other environmental conditions during placing and curing operations. Concrete that has been frozen or otherwise damaged by weather conditions shall be either repaired to an acceptable condition or removed and replaced. The temperature of the concrete mixture immediately before placement shall be between 50°F and 90°F, except as otherwise provided herein. 8.6.2 Rain Protection Under conditions of rain, the placing of concrete shall not commence or shall be stopped unless adequate protection is provided to prevent damage to the surface mortar or damaging flow or wash of the concrete surface. 8.6.3 Hot Weather Protection When the ambient temperature is above 90°F, the forms, reinforcing steel, steel beam flanges, and other surfaces which will come in contact with the mix shall be cooled to below 90°F by means of a water spray or other approved methods. The temperature of the concrete at time of placement shall be maintained within the specified temperature range by any combination of the following: Shading the materials storage areas or the production equipment. Cooling the aggregates by sprinkling with water which conforms to the requirements of Article 8.3.2. Cooling the aggregates or water by refrigeration or replacing a portion or all of the mix water with ice that is flaked or crushed to the extent that the ice will completely melt during mixing of the concrete. Liquid nitrogen injection. 8.6.4 Cold Weather Protection 8.6.4.1 Protection During Cure When there is a probability of air temperatures below 35°F during the cure period, the Contractor shall submit for approval by the Engineer prior to concrete placement, a cold weather concreting and curing plan detailing the methods and equipment which will be used to assure that the required concrete temperatures are maintained. The 531 concrete shall be maintained at a temperature of not less than 45°F for the first six days after-placement except that when pozzolan cement or fly ash cement is used, this period shall be as follows: Percentage of Cement Replaced, by Weight, With Pozzolans Required Period of Controlled Temperature 8 days 9 Days 10 Days The above requirement for an extended period of controlled temperature may be waived if a compressive strength of 65% of the specified 28-day design strength is achieved in 6 days. If external heating is employed, the heat shall be applied and withdrawn gradually and uniformly so that no part of the concrete surface is heated to more than 90°F or caused to change temperature by more than 20°F in 8 hours. When requested by the Engineer, the Contractor shall provide and install two maximum-minimum type thermometers at each structure site. Such thermometers shall be installed as directed by the Engineer so as to monitor the temperature of the concrete and the surrounding air during the cure period. 8.6.4.2 Mixing and Placing When the air temperature is below 35"F, the temperature of the concrete at the time of placement in sections less than 12 inches thick shall be not less than 60°F. Regardless of air temperature, aggregates shall be free of ice, frost and frozen lumps when batched and concrete shall not be placed against any material whose temperature is 32°F or less. 8.6.4.3 Heating of Mix When necessary in order to produce concrete of the specified temperature, either the mix water or the aggregates, or both, shall be heated prior to batching. Heating shall be done in a manner which is not detrimental to the mix and does not prevent the entrainment of the required amount of air. The methods used shall heat the materials uniformly. Aggregates shall not be heated directly by gas or oil flame or on sheet metal over fire. Neither aggregates nor water shall be heated to over 150°F. If either are heated to over 100°F,they shall be mixed together prior to the addition of the cement so that the cement does not come into contact with materials which are in excess of 100°F. 532 HIGHWAY BRIDGES 8.6.5 8.6.5 Special Requirements for Bridge Decks 8.7 HANDLING AND PLACING CONCRETE During periods of low humidity, wind or high temperatures and prior to the application of curing materials, concrete being placed and finished for bridge decks shall be protected from damage due to rapid evaporation. Such protection shall be adequate to prevent premature crusting of the surface or an increase in drying cracking. Such protection shall be provided by raising the humidity of the surrounding air with fog sprayers operated upwind of the deck, the use of wind-breaks or sun-shades, additionally reducing of the temperature of the concrete, scheduling placement during the cooler times of days or nights, or any combination thereof. For bridge decks that are located over or adjacent to salt water or when specified, the maximum temperature of the concrete at time of placement shall be 80°F 8.7.1 General 8.6.6 Concrete Exposed to Salt Water Unless otherwise specifically provided, concrete for structures exposed to salt or brackish water shall be Class S for concrete placed under water and Class A for other work. Such concrete shall be mixed for a period of not less than 2 minutes and the water content of the mixture shall be carefully controlled and regulated so as to produce concrete of maximum impermeability. The concrete shall be thoroughly consolidated as necessary to produce maximum density and a complete lack of rock pockets. Unless otherwise indicated on the plans, the clear distance from the face of the concrete to the reinforcing steel shall be not less than 4 inches. No construction joints shall be formed between levels of extreme low water and extreme high water or the upper limit of wave action as determined by the Engineer. Between these levels the forms shall not be removed, or other means provided, to prevent salt water from coming in direct contact with the concrete for a period of not less than 30 days after placement. Except for the repair of any rock pockets and the plugging of form tie holes, the original surface as the concrete comes from the forms shall be left undisturbed. Special handling shall be provided for precast members to avoid even slight deformation cracks. 8.6.7 Concrete Exposed to Sulfate Soils or Water When the special provisions identify the area as containing sulfate soils or water, the concrete that will be in contact with such soil or water shall be mixed, placed, and protected from contact with soil or water as required for concrete exposed to salt water except that the protection period shall be not less than 72 hours. Concrete shall be handled, placed, and consolidated by methods that will not cause segregation of the mix and will result in a dense homogeneous concrete which is free of voids and rock pockets. The methods used shall not cause displacement of reinforcing steel or other materials to be embedded in the concrete. Concrete shall be placed and consolidated prior to initial set and in no case more than 1% hours after the cement was added to the mix.
Retempering the concrete by adding water to the mix shall not be done. Concrete shall not be placed until the forms, all materials to be embedded and, for spread footings, the adequacy of the foundation material have been inspected and approved by the Engineer. All mortar from previous placements, debris, and foreign material shall be removed from the forms and steel prior to commencing placement. The forms and subgrade shall be thoroughly moistened with water immediately before concrete is placed against them. Temporary form spreader devices may be left in place until concrete placement precludes their need, after which they shall be removed. Placement of concrete for each section of the structure shall be done continuously without interruption between planned construction or expansion joints. The delivery rate, placing sequence and methods shall be such that fresh concrete is always placed and consolidated against previously placed concrete before initial set has occurred in the previously placed concrete. During and after placement of concrete, care shall be taken not to injure the concrete or break the bond with reinforcing steel. Workmen shall not walk in fresh concrete. Platforms for workmen and equipment shall not be supported directly on any reinforcing steel. Once the concrete is set, forces shall not be applied to the forms or to reinforcing bars, which project from the concrete, until the concrete is of sufficient strength to resist damage. 8.7.2 Sequence of Placement Whenever a concrete placement plan or schedule is specified or approved, the sequence of placement shall conform to the plan. Unless otherwise specifically permitted by such a placement plan, the requirements of the following paragraphs shall apply. 8.7.2.1 Vertical Members Concrete for columns, substructure and culvert walls, and other similar vertical members shall be placed and al- 8.7.2.1 DIVISION II-CONSTRUCTION lowed to set and settle for a period of time before concrete for integral horizontal members, such as caps, slabs, or footings is placed. Such period shall be adequate to allow completion of settlement due to loss of bleed water and shall be not less than 12 hours for vertical members over 15 feet in height and not less than 30 minutes for members over 5 feet but not over 15 feet in height.
When friction collars or falsework brackets are mounted on such vertical members and unless otherwise approved, the vertical member shall have been in place at least 7 days and shall have attained its specified strength before loads from horizontal members are applied. 533 the culvert is constructed. For culverts whose wall height is 5 feet or less, the sidewalls and top slab may be placed in one continuous operation. For higher culvert walls the requirements for vertical members shall apply. 8.7.2.5 Precast Elements The sequence of placement for concrete in precast elements shall be such that sound well-consolidatedconcrete which is free of settlement or shrinkage cracks is produced throughout the member. 8.7.3 Placing Methods 8.7.2.2 Superstructures 8.7.3.1 General Unless otherwise permitted, no concrete shall be placed in the superstructure until substructure forms have been stripped sufficiently to determine the character of the supporting substructure concrete. Concrete for T-beam or deck girder spans whose depth is less than 4 feet may be placed in one continuous operation or may be placed in two separate operations; first, to the top of the girder stems, and second, to completion. For T-beam or deck girder spans whose depth is 4 feet or more and, unless the falsework is nonyielding, such concrete shall be placed in two operations and at least 5 days shall elapse after placement of stems before the top deck slab is placed. Concrete for box girders may be placed in two or three separate operations consisting of bottom slab, girder stems and top slab. In either case the bottom slab shall be placed first and, unless otherwise permitted by the Engineer, the top slab shall not be placed until the girder stems have been in place for at least 5 days. Concrete shall be placed as nearly as possible in its final position and the use of vibrators for extensive shifting of the mass of fresh concrete will not be permitted. Concrete shall be placed in horizontal layers of a thickness not exceeding the capacity of the vibrator to consolidate the concrete and merge it with the previous lift. In no case shall the depth of a lift exceed 2 feet. The rate of concrete placement shall not exceed that assumed for the design of the forms as corrected for the actual temperature of the concrete being placed. When placing operations would involve dropping the concrete more than 5 feet, the concrete shall be dropped through a tube fitted with a hopper head, or through other approved devices, as necessary to prevent segregation of the mix and spattering of mortar on steel and forms above the elevation of the lift being placed. This requirement shall not apply to cast-in-place piling when concrete placement is completed before initial set occurs in the firstplaced concrete.
8.7.2.3 Arches 8.7.3.2 Equipment The concrete in arch rings shall be placed in such a manner as to load the centering uniformly and symmetrically. Arch rings shall be cast in transverse sections of such size that each section can be cast in a continuous operation.
The arrangement of the sections and the sequence of placing shall be as approved and shall be such as to avoid the creation of initial stress in the reinforcement. The sections shall be bonded together by suitable keys or dowels. Arch barrels for culverts and, unless prohibited by the special provisions, other arches may be cast in a single continuous operation.
8.7.2.4 Box Culverts In general, the base slab or footings of box culverts shall be placed and allowed to set before the remainder of All equipment used to place concrete shall be of adequate capacity and designed and operated so as to prevent segregation of the mix or loss of mortar. Such equipment shall not cause vibrations that might damage the freshly placed concrete.
No equipment shall have aluminum parts which come in contact with the concrete. Between uses, the mortar coating inside of placing equipment which sets or dries out shall be cleaned from the equipment before use is resumed. Chutes shall be lined with smooth watertight material and, when steep slopes are involved, shall be equipped with baffles or reverses. Concrete pumps shall be operated such that a continuous stream of concrete without air pockets is produced. When pumping is completed, the concrete remaining in 534 HIGHWAY BRIDGES the pipeline, if it is to be used, shall be ejected in such a manner that there will be no contaminationof the concrete or separation of the ingredients. Conveyor belt systems shall not exceed a total length of 550 lineal feet, measured from end to end of the total assembly. The belt assembly shall be so arranged that each section discharges into a vertical hopper arrangement to the next section. To keep segregation to a minimum, scrapers shall be situated over the hopper of each section so as to remove mortar adhering to the belt and to deposit it into the hopper. The discharge end of the conveyor belt system shall be equipped with a hopper, and a chute or suitable deflectors to cause the concrete to drop vertically to the deposit area. 8.7.4 Consolidation AU concrete, except concrete placed under water and concrete otherwise exempt, shall be consolidated by mechanical vibration immediately after placement. The vibration shall be internal except that external form vibrators may be used for thin sections when the forms have been designed for external vibration. %brators shall be of approved type and design and of a size appropriate for the work. They shall be capable of transmitting vibration to the concrete at frequencies of not less than 4,500 impulses per minute. The Contractor shall provide a sufficient number of vibrators to properly compact each batch immediately after it is placed in the forms. The Contractor shall also have at least one spare vibrator immediately available in case of breakdown. Vibrators shall be manipulated so as to thoroughly work the concrete around the reinforcement and imbedded fixtures and into the comers and angles of the forms. Vibration shall be applied at the point of deposit and in the area of freshly deposited concrete. The vibrators shall be inserted and withdrawn out of the concrete slowly. The vibration shall be of sufficient duration and intensity to thoroughly consolidate the concrete, but shall not be continued so as to cause segregation. Vibration shall not be continued at any one point to the extent that localized areas of grout are formed. Application of vibrators shall be at points uniformly spaced and not farther apart than 1.5 times the radius over which the vibration is visibly effective. Vibration shall not be applied directly to, or through the reinforcement to sections or layers of concrete which have hardened to the degree that the concrete ceases to be plastic under vibration. Vibrators shall not be used to transport concrete in the forms. When immersion-type vibrators are used to consolidate concrete around epoxy-coated reinforcement, the vibrators shall be equipped with rubber or other nonmetallic coating. 8.7.3.2 Vibration shall be supplemented by such spading as is necessary to ensure smooth surfaces and dense concrete along form surfaces and in corners and locations impossible to reach with the vibrators. When approved by the Engineer, concrete for small noncritical elements may be consolidated by the use of suitable rods and spades. 8.7.5 Underwater Placement 8.7.5.1 General Only concrete used in cofferdams to seal out water may be placed under water unless otherwise specified or specifically approved by the Engineer. If other than Class S concrete is to be placed under water, the minimum cement content of the mix shall be increased by 10% to compensate for loss due to wash. To prevent segregation, concrete placed under water shall be carefully placed in a compact mass, in its final position, by means of a tremie, concrete pump, or other approved method, and shall not be disturbed after being deposited. Still water shall be maintained at the point of deposit and the forms under water shall be watertight. Cofferdams shall be vented during the placement and cure of concrete to equalize the hydrostatic pressure and thus prevent flow of water through the concrete.
Concrete placed under water shall be placed continuously from start to finish. The surface of the concrete shall be kept as nearly horizontal as practicable. To ensure thorough bonding, each succeeding layer of seal shall be placed before the preceding layer has taken initial set. For large pours, more than one tremie or pump shall be used to ensure compliance with this requirement. 8.7.5.2 Equipment A tremie shall consist of a watertight tube having a diameter of not less than 10 inches and fitted with a hopper at the top. The tremies shall be supported so as to pennit free movement of the discharge end over the entire top surface of the work and so as to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be sealed closed at the start of work so as to prevent water from entering the tube before the tube is filled with concrete. After placement has started the tremie tube shall be kept full of concrete to the bottom of the hopper.
If water enters the tube after placement is started, the tremie shall be withdrawn, the discharge end resealed, and the placement restarted.
When a batch is dumped into the hopper, the flow of concrete shall be induced by slightly raising the discharge end, always keep- 8.7.5.2 DMSION II-CONSTRUCTION ing it in the deposited concrete. The flow shall be continuous until the work is completed. When cofferdam struts prevent lateral movement of tremies, one tremie shall be used in each bay. Concrete pumps used to place concrete under water shall include a device at the end of the discharge tube to seal out water while the tube is first being filled with concrete. Once the flow of concrete is started, the end of the discharge tube shall be kept full of concrete and below the surface of the deposited concrete until placement is completed. 8.7.5.3 Cleanup Dewatering may proceed after test specimens cured under similar conditions indicate that the concrete has sufficient strength to resist the expected loads. All laitance or other unsatisfactory materials shall be removed from the exposed surface by scraping, chipping, or other means which will not injure the surface of the concrete before placing foundation concrete. 8.8 CONSTRUCTION JOINTS 8.8.1 General Construction joints shall be made only where located on plans, or shown in the pouring schedule, unless otherwise approved. All planned reinforcing steel shall extend uninterrupted through joints. In the case of emergency, construction joints shall be placed as directed by the Engineer and, if directed, additional reinforcing steel dowels shall be placed across the joint. Such additional steel shall be furnished and placed at the Contractor's expense. 8.8.2 Bonding Unless otherwise shown on the plans, horizontal joints may be made without keys and vertical joints shall be constructed with shear keys. Surfaces of fresh concrete at horizontal construction joints shall be rough floated sufficiently to thoroughly consolidate the surface and intentionally left in a roughened condition. Shear keys shall consist of formed depressions in the surface covering approximately one-third of the contact surface. The forms for keys shall be beveled so that removal will not damage the concrete. All construction joints shall be cleaned of surface laitance, curing compound and other foreign materials before fresh concrete is placed against the surface of the joint. Abrasive blast or other approved methods shall be used to clean horizontal construction joints to the 535 extent that clean aggregate is exposed. All construction joints shall be flushed with water and allowed to dry to a surface dry condition immediately prior to placing concrete. 8.8.3 Bonding and Doweling to Existing Structures When new concrete is shown on the plans to be bonded to existing concrete structures, the existing concrete shall be cleaned and flushed as specified above. When the plans show reinforcing dowels grouted into holes drilled in the existing concrete at such construction joints, the holes shall be drilled by methods that will not shatter or damage the concrete adjacent to the holes. The diameters of the drilled holes shall be 1/4 inch larger than the nominal diameter of the dowels unless shown otherw

8.11.2.1 Water Water shall conform to the requirements of Article 8.3.2. 8.11.2.2 Liquid Membranes Liquid membrane-formingcompounds for curing concrete shall conform to the requirements of AASHTO M 148 (ASTM C 309). 8.11.2.3 Waterproof Sheet Materials Waterproof paper, polyethylene film, and white burlap polyethylene sheet shall conform to the requirements of AASHTO M 171 (ASTM C 171). 8.11.3 Methods 8.11.3.1 Forms-In-Place Method Formed surfaces of concrete may be cured by retaining the forms in place without loosening for the required time. 8.11.3.2 Water Method Concrete surface shall be kept continuously wet by ponding, spraying or covering with materials that are kept continuously and thoroughly wet. Such materials may 8.11.1 consist of cotton mats, multiple layers of burlap or other approved materials which do not discolor or otherwise damage the concrete. 8.11.3.3 Liquid Membrane Curing Compound Method The liquid membrane method shall not be used on surfaces where a rubbed finish is required or on surfaces of construction joints unless it is removed by sand blasting prior to placement of concrete against the joint. Type 2, white pigmented, liquid membranes may be used only on the surfaces of bridge decks, on surfaces that will not be exposed to view in the completed work or on surfaces where their use has been approved by the Engineer. When membrane curing is used, the exposed concrete shall be thoroughly sealed immediately after the free water has left the surface. Formed surfaces shall be sealed immediately after the forms are removed and necessary finishing has been done.
The solution shall be applied by power-operated atomizing spray equipment in one or two separate applications. Hand-operated sprayers may be used for coating small areas. Membrane solutions containing pigments shall be thoroughly mixed prior to use and agitated during application. If the solution is applied in two increments, the second application shall follow the first application within 30 minutes. Satisfactory equipment shall be provided, together with means to properly control and assure the direct application of the curing solution on the concrete surface so as to result in a uniform coverage at the rate of 1 gallon for each 150 square feet of area. If rain falls on the newly coated concrete before the film has dried sufficiently to resist damage, or if the film is damaged in any other manner during the curing period, a new coat of the solution shall be applied to the affected portions equal in curing value to that above specified.
8.11.3.4 Waterproof Cover Method This method shall consist of covering the surface with a waterproof sheet material so as to prevent moisture loss from the concrete.
This method may be used only when the covering can be secured adequately to prevent moisture loss. The concrete shall be wet at the time the cover is installed. The sheets shall be of the widest practicable width and adjacent sheets shall overlap a minimum of 6 inches and shall be tightly sealed with pressure sensitive tape, mastic, glue, or other approved methods to form a complete waterproof cover of the entire concrete surface. The paper shall be secured so that wind will not displace it. Should any portion of the sheets be broken or 8.11.3.4 DIVISION II-CONSTRUCTION damaged before expiration of the curing period, the broken or damaged portions shall be immediately repaired. Sections that have lost their waterproof qualities shall not be used. 8.11.3.5 Steam or Radiant Heat Curing Method This method may be used only for precast concrete members manufactured in established plants.
Steam curing or radiant heat curing shall be done under a suitable enclosure to contain the live steam or the heat. Steam shall be low pressure and saturated. Temperature recording devices shall be employed as necessary to verify that temperatures are uniform throughout the enclosure and within the limits specified. The initial application of the steam or of the heat shall be from 2 to 4 hours after the final placement of concrete to allow the initial set of the concrete to take place. If retarders are used, the waiting period before application of the steam or of the radiant heat shall be increased to between 4 and 6 hours after placement. The time of initial set may be determined by the "Standard Method of Test for Time of Setting of Concrete Mixtures by Penetration Resistance," AASHTO T 197 (ASTM C 403), and the time limits described above may then be waived. During the waiting period, the temperature within the curing chamber shall not be less than 50°F and live steam or radiant heat may be used to maintain the curing - chamber at the proper minimum temperature. During this period the concrete shall be kept wet. Application of live steam shall not be directed on the concrete or on the forms so as to cause localized high temperatures. During the initial application of live steam or of radiant heat, the ambient temperature within the curing enclosure shall increase at an average rate not exceeding 40°F per hour until the curing temperature is reached. The maximum curing temperature within the enclosure shall not exceed 160°F. The maximum temperature shall be held until the concrete has reached the desired strength. In discontinuing the steam application, the ambient & temperature shall not decrease at a rate to exceed 40°F per hour until a temperature 20°F above the temperature of the air to which the concrete will be exposed has been reached. Radiant heat may be applied by means of pipes circulating steam, hot oil or hot water, or by electric heating elements. Radiant heat curing shall be done under a suitable enclosure to contain the heat, and moisture loss shall be minimized by covering all exposed concrete surfaces with a plastic sheeting or by applying an approved liquid membrane-curing compound to all exposed concrete surfaces. Top surfaces of concrete members to be used in composite construction shall be clear of residue of the membrane 541 curing compound so as not to reduce bond below design limits. Surfaces of concrete members to which other materials will be bonded in the finished structure shall be clear of residue of the membrane curing compound so as not to reduce bond below design limits. Unless the ambient temperature is maintained above 60°F, for prestressed members the transfer of the stressing force to the concrete shall be accomplished immediately after the steam curing or the heat curing has been discontinued. 8.11.4 Bridge Decks The top surfaces of bridge decks shall be cured by a combination of the liquid membrane curing compound method and the water method. The liquid membrane shall be Type 2, white pigmented, and shall be applied from finishing bridges progressively and immediately after finishing operations are complete on each portion of the deck. The water cure shall be applied not later than 4 hours after completion of deck finishing or, for portions of the decks on which finishing is completed after normal working hours, the water cure shall be applied not later than the following morning. 8.12 FINISHING FORMED CONCRETE SURFACES 8.12.1 General Surface finishes for formed concrete surfaces shall be classified as follows: Class 1. Ordinary Surface Finish Class 2. Rubbed Finish Class 3.
Tooled Finish Class 4. Sandblast Finish Class 5. Wire Brush, or Scrubbed Finish All concrete shall be given a Class 1, Ordinary Surface Finish, and in addition if further finishing is required, such other type of finish as is specified.
If not otherwise specified, exposed surfaces except the soffits of superstructures and the interior faces and bottoms of concrete girders shall also be given a Class 2, Rubbed Finish. Class 3, 4, or 5 type surface finishes shall be applied only where shown on the plans or specified. 8.12.2 Class 1-Ordinary Surface Finish Immediately following the removal of forms, fins, and irregular projections shall be removed from all surfaces HIGHWAY BRIDGES which are to be exposed or waterproofed. Bulges and offsets in such surfaces shall be removed with carborundum stones or discs. Localized poorly bonded rock pockets or honeycombed concrete shall be removed and replaced with sound concrete or packed mortar as specified in Article 8.14. If rock pockets, in the opinion of the Engineer, are of such an extent or character as to affect the strength of the structure materially or to endanger the life of the steel reinforcement, he or she may declare the concrete defective and require the removal and replacement of the portions of the structure affected. On all surfaces, the cavities produced by form ties and all other holes, broken comers or edges, and other defects shall be thoroughly cleaned, and after having been thoroughly saturated with water shall be carefully pointed and trued with a mortar conforming to Article 8.14. For exposed surfaces, white cement shall be added to the mortar in an amount sufficient to result in a patch which, when dry, matches the surrounding concrete. Mortar used in pointing shall be not more than 1 hour old. The concrete shall then be rubbed if required or the cure continued as specified under Article 8.10. Construction and expansion joints in the completed work shall be left carefully tooled and free of mortar and concrete. The joint filler shall be left exposed for its full length with clean and true edges. The resulting surfaces shall be true and uniform. Repaired surfaces, the appearance of which is not satisfactory, shall be "rubbed" as specified under Class 2, Rubbed Finish. 8.12.3 Class &Rubbed Finish After removal of forms, the rubbing of concrete shall be started as soon as its condition will permit. Immediately before starting this work, the concrete shall be thoroughly saturated with water. Sufficient time shall have elapsed before the wetting down to allow the mortar used in the pointing of rod holes and defects to thoroughly set.
Surfaces to be finished shall be rubbed with a medium coarse carborundum stone, using a small amount of mortar on its face. The mortar shall be composed of cement and fine sand mixed in proportions used in the concrete being finished. Rubbing shall be continued until form marks, projections, and irregularities have been removed, voids have been filled, and a uniform surface has been obtained. The paste produced by this rubbing shall be left in place. After other work which could effect the surface has been completed, the final finish shall be obtained by rubbing with a fine carborundum stone and water. This rubbing shall be continued until the entire surface is of a smooth texture and uniform color. 8.12.2 After the final rubbing is completed and the surface has dried, it shall be rubbed with burlap to remove loose powder and shall be left free from all unsound patches, paste, powder, and objectionable marks. When metal forms, fiber forms, lined forms or plywood forms in good condition are used, the requirement for a Class 2, Rubbed Finish may be waived by the Engineer when the uniformity of color and texture obtained with Class 1 finishing are essentially equal to that which could be attained with the application of a Class 2, Rubbed Finish.
In such cases, grinding with powered disc grinders or light sandblasting with fine sand or other means approved by the Engineer may be utilized in conjunction with Class 1 finishing.
8.12.4 Class 3--Tooled Finish Finish of this character for panels and other like work may be secured by the use of a bushhammer, pick, crandall, or other approved tool. Air tools, preferably, shall be employed. No tooling shall be done until the concrete has set for at least 14 days and as much longer as may be necessary to prevent the aggregate particles from being "picked" out of the surface. The finished surface shall show a grouping of broken aggregate particles in a matrix of mortar, each aggregate particle being in slight relief. 8.12.5 Class Aandblasted Finish The thoroughly cured concrete surface shall be sandblasted with hard, sharp sand to produce an even finegrained surface in which the mortar has been cut away, leaving the aggregate exposed. 8.12.6 Class 5 W i r e Brushed or Scrubbed Finish As soon as the forms are removed and while the concrete is yet comparatively green, the surface shall be thoroughly and evenly scrubbed with stiff wire or fiber brushes, using a solution of muriatic acid in the proportion of one part acid to four parts water until the cement film or surface is completely removed and the aggregate particles are exposed, leaving an even-pebbled texture presenting an appearance grading from that of fine granite to coarse conglomerate, depending upon the size and grading of aggregate used. When the scrubbing has progressed sufficiently to produce the texture desired, the entire surface shall be thoroughly washed with water to which a small amount of ammonia has been added, to remove all traces of acid. 8.13 DIVISION 11--CONSTRUCTION 8.13 PRECAST CONCRETE MEMBERS 8.13.1 General Precast concrete members shall be constructed and placed in the work in conformance with the details shown on the plans, specified or shown on the approved working drawings. If approved by the Engineer, the use of precasting methods may be used for elements of the work which are otherwise indicated to be constructed by the cast-in-place method. When such precasting is proposed, the Contractor shall submit working drawings showing construction joint details and any other information required by the Engineer. 8.13.2 Working Drawings Whenever specified or requested by the Engineer, the Contractor shall provide working drawings for precast members. Such drawings shall include all details not provided in the plans for the construction and the erection of the members and shall be approved before any members are cast. Such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work. 8.13.3 Materials and Manufacture The materials and manufacturing processes used for precast concrete members shall conform to the requirements of the other articles in this section except as those requirements are modified or supplemented by the provisions that follow. When precast members are manufactured in established casting yards, the manufacturer shall be responsible for the continuous monitoring of the quality of all materials and concrete strengths. Tests shall be performed in accordance with appropriate AASHTO or ASTM methods. The Engineer shall be allowed to observe all sampling and testing and the results of all tests shall be made available to the Engineer. Established, Precast Concrete Manufacturing Plants shall be certified under the Precast/Prestressed Concrete Institute (PCI) Certification Program or alternative equivalent program for the category of work being manufactured. Plant Quality Control personnel shall be certified in the PC1Quality Control Personnel Certification Program, Level 11. Plant Quality Control Managers shall be certified PC1 Level III. These requirements may be met by alternative experience and certification considered to be equivalent. 543 Precast members shall be cast on unyielding beds or pallets.
Special care shall be used in casting the bearing surfaces so that they will join properly with other elements of the structure. For prestressed precast units, several units may be cast in one continuous line and stressed at one time. Sufficient space shall be left between ends of units to permit access for cutting of tendons after the concrete has attained the required strength. The side forms may be removed as soon as their removal will not cause distortion of the concrete surface, providing that curing is not intempted. Members shall not be lifted from casting beds until their strength is sufficient to prevent damage. When cast-in-place concrete will later be cast against the top surfaces of precast beams or girders, these surfaces shall be finished to a coarse texture by brooming with a stiff coarse broom. Prior to shipment, such surfaces shall be cleaned of laitance or other foreign material by sandblasting or other approved methods. When precast members are designed to be abutted together in the finished work, each member shall be matchcast with its adjacent segments to ensure proper fit during erection. As the segments are match-cast they must be precisely aligned to achieve the final structure geometry. During the alignment, adjustments to compensate for deflections shall be made. 8.13.4 Curing Unless otherwise permitted, precast members shall be cured by either the water method or the steam or radiant heat method.
8.13.5 Storage and Handling Extreme care shall be exercised in handling and moving precast prestressed concrete members. Precast girders shall be transported in an upright position and the points of support and directions of the reactions with respect to the member shall be approximately the same during transportation and storage as when the member is in its final position. Prestressed concrete members shall not be shipped until tests on concrete cylinders, manufactured of the same concrete and cured under the same conditions as the girders, indicate that the concrete of the particular member has attained a compressive strength equal to the specified design compressive strength of the concrete in the member.
Care shall be taken during storage, hoisting, and handling of the precast units to prevent cracking or damage. Units damaged by improper storage or handling shall be replaced at the Contractor's expense. 544 HIGHWAY BRIDGES 8.13.6 Erection The Contractor shall be responsible for the safety of precast members during all stages of construction. Lifting devices shall be used in a manner that does not cause damaging bending or torsional forces. After a member has been erected and until it is secured to the structure, temporary braces shall be provided as necessary to resist wind or other loads.
Precast deck form panels shall be erected and placed so that the fit of mating surfaces shall be such that excessive grout leakage will not occur. If such fit is not provided, joints shall be dry-packed or sealed with an acceptable caulking compound prior to placing the cast-in-placeconcrete. End panels for skewed structures may be sawed to fit the skew.
8.13.7 Epoxy Bonding Agents for Precast Segmental Box Girders 8.13.7.1 Materials Epoxy bonding agents for match cast joints shall be thermosetting 100% solid compositions that do not contain solvent or any nonreactive organic ingredient except for pigments required for coloring. Epoxy bonding agents shall be of two components, a resin and a hardener.
The two components shall be distinctly pigmented, so that mixing produces a third color similar to the concrete in the segments to be joined, and shall be packaged in preportioned, labeled, ready-to-use containers. Epoxy bonding agents shall be formulated to provide application temperature ranges that will permit erection of match cast segments at substrate temperatures from 40°F to 115°F If two surfaces to be bonded have different substrate temperatures, the adhesive applicable at the lower temperature shall be used. Epoxy bonding agents shall be insensitive to damp conditions during application and, after curing, shall exhibit high bonding strength to cured concrete, good water resistivity, low creep characteristics, and tensile strength greater than the concrete. In addition, the epoxy bonding agents shall function as a lubricant during the joining of the match cast segments, as a filler to accurately match the surface of the segments being joined, and as a durable, watertight bond at the joint. Epoxy bonding agents shall be tested to determine their workability, gel time, open time, bond and compression strength, shear, and working temperature range.
The frequency of the tests shall be as stated in the Special Provisions of the Contract. The Contractor shall furnish the Engineer with samples of the material for quality assurance testing, and a certifi- 8.13.6 cation from a reputable independent laboratory indicating that the material has passed the required tests. Specific properties of epoxy and the test procedures to be used to measure these properties shall be as described in the following subarticles. 8.13.7.1.I Test I-Sag Flow of Mixed Epoxy Bonding Agent This test measures the application workability of the bonding agent. Testing Method: ASTM D 2730 for the designated temperature range. Specijication: Mixed epoxy bonding agent must not sag flow at %-inch minimum thickness at the designated minimum and maximum application temperature range for the class of bonding agents used. 8.13.7.1.2 Test 2-Gel Time of Mixed Epoxy Bondinn Anent Gel time is determined on samples mixed as specified in the testing method. It provides a guide for the period of time the mixing bonding agent remains workable in the mixing container during which it must be applied to the match-cast joint surfaces. Testing Method: ASTM D 2471 (except that 1 quart and 1 gallon quantities shall be tested). Specijication: 30 minutes minimum on 1 quart and 1 gallon quantities at the maximum temperature of the designated application temperature range. (Note: Gel time is not to be confused with open time specified in Test 3.) 8.13.7.1.3 Test 3-Open Time of Bonding Agent This test measures workability of the epoxy bonding agent for the erection and post-tensioning operations. As tested here, open time is defined as the minimum allowable period of elapsed time from the application of the mixed epoxy bonding agent to the precast segments until the two segments have been assembled together and temporarily post-tensioned. Testing Method: Open time is determined using test specimens as detailed in the Tensile Bending Test (Test 4). The epoxy bonding agent, at the highest specified application temperature, is mixed together and applied as instructed in Test 4 to the concrete prisms, which shall also be at the highest specified application temperature. The adhesive coated prisms shall be maintained for 60 minutes 8.13.7.1.3 DIVISION II-CONSTRUCTION at the highest specified application temperature with the adhesive coated surface or surfaces exposed and uncovered before joining together. The assembled prisms are then curved and tested as instructed in Test 4. Specijication: The epoxy bonding agent is acceptable for the specified application temperature only when essentially total fracturing of concrete paste and aggregate occurs with no evidence of adhesive failure. Construction situations may sometimes require application of the epoxy bonding agent to the precast section prior to erecting, positioning, and assembling. This operation may require epoxy bonding agents having prolonged open time. In general, where the erection conditions are such that the sections to be bonded are prepositioned prior to epoxy application, the epoxy bonding agent shall have a minimum open time of 60 minutes within the temperature range specified for its application. 8.13.7.1.4 Test 4-Three-Point Tensile Bending Test This test, performed on a pair of concrete prisms bonded together with epoxy bonding agent, determines the bonding strength between the bonding agent and concrete. The bonded concrete prisms are compared to a reference test beam of concrete 6 X 6 X 18 inches. Testing Method: 6 X 6 X 9-inch concrete prisms of 6,000-psi compressive strength at 28 days shall be sandblasted on one 6 X 6-inch side to remove mold release agent, laitance, etc., and submerged in clean water at the lower temperature of the specified application temperature range for 72 hours. Immediately on removing the concrete prisms from the water, the sandblasted surfaces shall be air-dried for 1 hour at the same temperature and 50% relative humidity and each shall be coated with approximately a %-inch layer of the mixed bonding agent. The adhesive coated faces of two prisms shall then be placed together and held with a clamping force normal to the bonded interface of 50 psi. The assembly shall then be wrapped in a damp cloth which is kept wet during the curing period of 24 hours at the lower temperature of the specified application temperature range. After 24 hours curing at the lower temperature of the application temperature range specified for the epoxy bonding agent, the bonded specimen shall be unwrapped, removed from the clamping assembly and immediately tested. The test shall be conducted using the standard AASHTO T 97 (ASTM C 78) test for flexural strength with third point loading and the standard MR unit.
At the same time the two prisms are prepared and cured, a companion test beam shall be prepared of the 545 same concrete, cured for the same period, and tested following AASHTO T 97 (ASTM C 78). Specijication: The epoxy bonding agent is acceptable if the load on the prisms at failure is greater than 90% of the load on the reference test beam at failure. 8.13.7.1.5 Test 5-Compression Strength of Cured Epoxy Bonding Agent This test measures the compressive strength of the epoxy bonding agent.
Testing Method: ASTM D 695. Speci]ication:Compressive strength at 77OF shall be 2,000 psi minimum after 24 hours cure at the minimum temperature of the designated application temperature range and 6,000 psi at 48 hours. 8.13.7.1.6 Test 6-Temperature Dejection of Epoxy Bonding Agent This test determines the temperature at which an arbitrary deflection occurs under arbitrary testing conditions in the cured epoxy bonding agent.
It is a screening test to establish performance of the bonding agent throughout the erection temperature range. Testing Method: ASTM D 648. Speci]ication: A minimum deflection temperature of 122°F at fiber stress loading of 264 psi is required on test specimens cured 7 days at 77OF. 8.13.7.1.7 Test 7-Compression and Shear Strength of Cured Epoxy Bonding Agent This test is a measure of the compressive strength and shear strength of the epoxy bonding agent compared to the concrete to which it bonds. The "slant cylinder" specimen with the epoxy bonding agent is compared to a reference test cylinder of concrete only. Testing Method: A test specimen of concrete is prepared in a standard 6 X 12-inch cylinder mold to have a height at midpoint of 6 inches and an upper surface with a 30" slope from the vertical. The upper and lower portions of the specimen with the slant surfaces may be formed through the use of an elliptical insert or by sawing a full-sized 6 X 12-inch cylinder. If desired, 3 X 6inch or 4 X 8-inch specimens may be used. After the 546 HIGHWAY BRIDGES specimens have been moist cured for 14 days, the slant surfaces shall be prepared by light sandblasting, stoning, or acid etching, then Gashing and drying the surfaces, and finally coating one of the surfaces with a 10-mil. thickness of the epoxy bonding agent under test. The specimens shall then be pressed together and held in position for 24 hours. The assembly shall then be wrapped in a damp cloth which shall be kept wet during an additional curing period of 24 hours at the minimum temperature of the designated application temperature range. The specimen shall then be tested at 77OF following AASHTO T 22 (ASTM C 39) procedures. At the same time as the slant cylinder specimens are made and cured, a companion standard test cylinder of the same concrete shall be made, cured for the same period, and tested following AASHTO T 22 (ASTM C 39). Spec$cation: The epoxy bonding agent is acceptable for the designated application temperature range if the load on the slant cylinder specimen is greater than 90% of the load on the companion cylinder. 8.13.7.2 Mixing and Installation of Epoxy Instructionsfurnished by the supplier for the safe storage, mixing, and handling of the epoxy bonding agent shall be followed. The epoxy shall be thoroughly mixed until it is of uniform color. Use of a proper-sized mechanical mixer operating at no more than 600 RPM will be required. Contents of damaged or previously opened containers shall not be used. Surfaces to which the epoxy material is to be applied shall be at least 40°F and shall be free from oil, laitance, form release agent, or any other material that would prevent the epoxy from bonding to the concrete surface. All laitance and other contaminants shall be removed by light sandblasting or by high pressure water blasting with a minimum pressure of 5,000 psi. Wet surfaces shall be dried before applying epoxy bonding agents. The surface shall be at least the equivalent of saturated surface dry (no visible water). Mixing shall not start until the segment is prepared for installation. Application of the mixed epoxy bonding agent shall be according to the manufacturer's instructions using trowel, rubber glove, or brush on one or both surfaces to be joined.
The coating shall be smooth and uniform and shall cover the entire surface with a minimum thickness of 1/16 inch applied on both surfaces or y8 inch if applied on one surface. Epoxy should not be placed within 3/8 inch of prestressing ducts to minimize flow into the ducts. A discernible bead line must be observed on all exposed contact areas after temporary post-tensioning. Erection operations shall be coordinated and conducted so as to complete the operations of apply- 8.13.7.1.7 ing the epoxy bonding agent to the segments, erection, assembling, and temporary post-tensioning of the newly joined segment within 70% of the open time period of the bonding agent. The epoxy material shall be applied to all surfaces to be joined within the first half of the gel time, as shown on the containers. The segments shall be joined within 45 minutes after application of the first epoxy material placed and a minimum average temporary prestress of 40 psi over the cross section should be applied within 70% of the open time of the epoxy material. At no point of the cross section shall the temporary prestress be less than 30 psi. The joint shall be checked immediately after erection to verify uniform joint width and proper fit. Excess epoxy from the joint shall be removed where accessible. All tendon ducts shall be swabbed immediately after stressing, while the epoxy is still in the nongelled condition, to remove or smooth out any epoxy in the conduit and to seal any pockets or air bubble holes that have formed at the joint. If the jointing is not completed within 70% of the open time, the operation shall be terminated and the epoxy bonding agent shall be completely removed from the surfaces. The surfaces must be prepared again and fresh epoxy shall be applied to the surface before resuming jointing operations. As general instructions cannot cover all situations, specific recommendations and instructions shall be obtained in each case from the Engineer in charge. 8.14 MORTAR AND GROUT 8.14.1 General This work consists of the making and placing of mortar and grout for use in concrete structures other than in prestressing ducts. Such uses include mortar for filling under masonry plates and for filling keyways between precast members where shown on the plans, mortar used to fill voids and repair surface defects, grout used to fill sleeves for anchor bolts, and mortar and grout for other such uses where required or approved. 8.14.2 Materials and Mixing Materials for mortar and grout shall conform to the requirements of Article 8.3. The grading of sand for use in grout or for use in mortar when the width or depth of the void to be filled is less than 3/4 inch shall be modified so that all material passes the No. 8 sieve. Type lA, air entraining, Portland cement shall be used when air entrainment is required for the concrete against which the grout or mortar is to be placed. 8.14.2 DIVISION 11-4 Unless otherwise specified or ordered by the Engineer, the proportion of cement to sand for mortar shall be one to two and for grout shall be one to one. Proportioning shall be by loose volume.
When nonshrink mortar or grout is specified, either a nonshrink admixture or an expansive hydraulic cement conforming to ASTM C 845 of a type approved by the Engineer, shall be used. Only sufficient water shall be used to permit placing and packing.
For mortar, only enough water shall be used so that the mortar will form a ball when squeezed gently in the hand. Mixing shall be done by either hand methods or with rotating paddle-type mixing machines and shall be continued until all ingredients are thoroughly mixed. Once mixed, mortar or grout shall not be retempered by the addition of water and shall be placed within 1 hour. 8.14.3 Placing and Curing Concrete areas to be in contact with the mortar or grout shall be cleaned of all loose or foreign material that would in any way prevent bond and the concrete surfaces and shall be flushed with water and allowed to dry to a surface dry condition immediately prior to placing the mortar or grout. The mortar or grout shall completely fill and shall be tightly packed into recesses and holes, on surfaces, under structural members, and at other locations specified. After placing, all surfaces of mortar or grout shall be cured by the water method as provided in Article 8.11 for a period of not less than 3 days. Keyways, spaces between structural members, holes, spaces under structural members, and other locations where mortar could escape shall be mortar-tight before placing mortar. No load shall be allowed on mortar that has been in place less than 72 hours, unless otherwise permitted by the Engineer. All improperly cured or otherwise defective mortar or grout shall be removed and replaced by the Contractor at own expense. 8.15.2 Earth Loads Whenever possible the sequence of placing backfill around structures shall be such that overturning or sliding forces are minimized. When the placement of backfill will cause flexural stresses in the concrete, and unless otherwise permitted by the Engineer, the placement shall not begin until the concrete has reached not less than 80% of its specified strength. 8.15.3 Construction Loads Light materials and equipment may be carried on bridge decks only after the concrete has been in place at least 24 hours, providing curing is not interfered with and the surface texture is not damaged. Vehicles needed for construction activities and weighing between 1,000 and 4,000 pounds, and comparable materials and equipment loads, will be allowed on any span only after the last placed deck concrete has attained a compressive strength of at least 2,400 pounds per square inch. Loads in excess of the above shall not be carried on bridge decks until the deck concrete has reached its specified strength. In addition, for post-tensioned structures, vehicles weighing over 4,500 pounds, and comparable materials and equipment loads, will not be allowed on any span until the prestressing steel for that span has been tensioned. Precast concrete or steel girders shall not be placed on substructure elements until the substructure concrete has attained 70% of its specified strength. Otherwise, loads imposed on existing, new or partially completed portions of structures due to construction operations shall not exceed the load-carrying capacity of the structure, or portion of structure, as determined by the Load Factor Design methods of AASHTO using Load Group IB. The compressive strength of concrete (f,') to be used in computing the load-carrying capacity shall be the smaller of the actual compressive strength at the time of loading or the specified compressive strength of the concrete. 8.15.4 Traffic Loads Traffic will not be permitted on concrete decks until at least 14 days after the last placement of deck concrete and until such concrete has attained its specified strength. 8.15 APPLICATION OF LOADS 8.16 MEASUREMENT AND PAYMENT 8.15.1 General 8.16.1 Measurement Loads shall not be applied to concrete structures until the concrete has attained sufficient strength and, when applicable, sufficient prestressing has been completed, so that damage will not occur. Except for concrete in components of the work for which payment is made under other bid items, all concrete for structures will be measured by either the cubic yard for each class of concrete included in the schedule of bid 548 HIGHWAY BRIDGES items or by the unit for each type of precast concrete member listed in the schedule of bid items.
When measured by the cubic yard, the quantity of concrete will be computed from the dimensions shown on the plans or authorized in writing by the Engineer with the following exceptions: The quantity of concrete involved in fillets, scorings and chamfers 1 square inch or less in cross-sectional area will not be included or deducted. Deductions for the volume of concrete displaced by concrete and timber piles embedded in the concrete will be made. Deductions for other embedded materials including reinforcing, structural and prestressing steel, expansion joint filler material, waterstops and deck drains will not be made. The volume of timber piles will be assumed to be 0.8 cubic foot per linear foot of pile. When there is a bid item for concrete to be used as a seal course in cofferdams, the quantity of such concrete to be paid for shall include the actual volume of concrete seal course in place, but in no case shall the total volume to be paid for exceed the cubical contents contained between the vertical surfaces 1 foot outside the neat lines of the seal course as shown on the plans. The thickness of seal course to be paid for shall be the thickness shown on the plans or ordered in writing by the Engineer. The number of precast concrete members of each type listed in the schedule of bid items will be the number of acceptable members of each type furnished and installed in the work. Expansion joint armor assemblies will be measured and paid for as provided for in Section 23, "Miscellaneous Metal." 8.16.1 Whenever an alternative or option is shown on the plans or permitted by the specifications, the quantities of concrete will be computed on the basis of the dimensions shown on the plans and no change in quantities measured for payment will be made because of the use by the Contractor of such alternatives or options. 8.16.2 Payment The cubic yards of concrete and the number of precast concrete members, as measured above for each type or class listed in the schedule of bid items, will be paid for at the contract prices per cubic yard or the contract prices per each member. Payment for concrete of the various classes and for precast concrete members of the various types shall be considered to be full compensation for the cost of furnishing all labor, materials, equipment, and incidentals, and for doing all the work involved in constructing the concrete work complete in place, as shown on the plans and specified. Such payment includes full compensation for furnishing and placing expansion joint fillers, sealed joints, waterstops, drains, vents, miscellaneous metal devices and the drilling of holes for dowels and the grouting of dowels in drilled holes, unless payment for such work is specified to be included in another bid item. In addition, payment for precast concrete members shall be considered to be full compensation for the cost of all reinforcing steel, prestressing materials and other items embedded in the member, and for the erection of the members. Section 9 REINFORCING STEEL 9.1 DESCRIPTION the point of shipment to the jobsite shall conform to the requirements of AASHTO M 284 (ASTM D 3963) or ASTM A 934, as specified in the contract documents. Epoxy-coated reinforcing bars shall be coated in a certified epoxy coating applicator plant in accordance with the Concrete Reinforcing Steel Institute's Voluntary Certification Program for Fusion-Bonded Epoxy Coated Applicator Plants, or equivalent. Epoxy-coated steel wire and welded wire fabric for reinforcement shall conform to the requirements of ASTM A 884, Class A. Each shipment of epoxy-coated reinforcing steel shall be accompanied with a Certificate of Compliance signed by the applicator of the coating certifying that the epoxycoated reinforcing bars conform to the requirements of AASHTO M 284 or ASTM A 934 or that the epoxycoated wire or welded wire fabric conforms to ASTM A 884, Class A. This work shall consist of furnishing and placing reinforcing steel in accordance with these Specifications and in conformity with the plans. 9.2 MATERIAL All reinforcing bars shall be deformed except that plain bars may be used for spirals and ties. Reinforcing steel shall conform to the requirements of the following specifications. 9.2.1 Uncoated Reinforcing Steel Deformed and Plain Billet-Steel Bars for Concrete Reinforcement-AASHTO M 31 (ASTM A 615). Grade 60 shall be used unless otherwise shown or specified. Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement-ASTMA 706. Rail-Steel Deformed and Plain Bars for Concrete Reinforcement-AASHTO M 42 including Supplementary Requirement S1 (ASTM A 616 including Supplementary Requirement Sl). Grade 60 steel shall be used unless otherwise shown or specified.
Deformed Steel Wire for Concrete ReinforcementAASHTO M 225 (ASTM A 496). Welded Plain Steel Wire Fabric for Concrete Reinforcement-AASHTO M 55 (ASTM A 185). Plain Steel Wire for Concrete ReinforcementAASHTO M 32 (ASTM A 82). Welded Deformed Steel Wire Fabric for Concrete Reinforcement-AASHTO M 221 (ASTM A 497).
Whenever steel reinforcing bars, other than bars conforming to ASTM A 706, are to be spliced by welding or when otherwise requested, a certified copy of the mill test report showing physical and chemical analysis for each heat or lot of reinforcing bars delivered shall be provided to the Engineer. 9.2.2 Epoxy-Coated Reinforcing Steel 9.3 BAR LISTS AND BENDING DIAGRAMS The reinforcing steel to be epoxy coated shall conform to Article 9.2.1. When epoxy coating of reinforcing bars is required, the coating materials and process, the fabrication, handling, identification of the bars, and the repair of damaged coating material that occurs during fabrication and handling to When the plans do not include detailed bar lists and bending diagrams, the Contractor shall provide such lists and diagrams to the Engineer for review and approval. Fabrication of material shall not begin until such lists have been approved. The approval of bar lists and bending diagrams shall in no way relieve the Contractor of responsi- 9.2.3 Stainless Steel Reinforcing Bars When required by the contract documents, deformed or plain stainless steel reinforcing bars shall conform to the requirements of ASTM A 955 M. 9.2.4 Mill Test Reports 549 550 HIGHWAY BRIDGES bility for the correctness of such lists and diagrams. Any expense incident to the revision of material furnished in accordance with such lists and diagrams to make it comply with the design drawings shall be borne by the Contractor. 9.4 FABRICATION 9.4.1 Bending Bar reinforcement shall be cut and bent to the shapes shown on the plans. Fabrication tolerances shall be in accordance with ACI 315. All bars shall be bent cold, unless otherwise permitted. Bars partially embedded in concrete shall not be field bent except as shown on the plans or specifically permitted. 9.3 ing. All systems for handling epoxy-coated reinforcement bars shall have adequately padded contact areas. All bundling bands shall be padded and all bundles shall be lifted with a strong back, multiple supports, or platform bridge so as to prevent bar-to-bar abrasion from sags in the bundle. Bars or bundles shall not be dropped or dragged. Epoxy-coated reinforcing steel shall be stored on wooden or padded supports. Epoxy-coated reinforcing steel shall be protected from sunlight, salt spray, and weather exposure. Provisions shall be made for air circulation arouad the coated reinforcement to minimize condensation under the protective covering.
9.6 PLACING AND FASTENING 9.4.2 Hooks and Bend Dimensions 9.6.1 General The dimensions of hooks and the diameters of bends measured on the inside of the bar shall be as shown on the plans. When the dimensions of hooks or the diameter of bends are not shown, they shall be in accordance with Division I, Article 8.23 or ACI 318, "Building Code Requirements for Reinforced Concrete." Steel reinforcement shall be accurately placed as shown on the plans and 6rmly held in position during the placing and consolidation of concrete. Bars shall be tied at all intersections around the perimeter of each mat and at not less than 2-foot centers or at every intersection, whichever is greater, elsewhere. Bundled bars shall be tied together at not more than 6-foot centers. For fastening epoxy-coated reinforcement, tie wire and metal clips shall be plastic-coated or epoxy-coated. If uncoated welded wire fabric is shipped in rolls, it shall be straightened into flat sheets before being placed. Welding of crossing bars (tack welding) will not be permitted for assembly of reinforcement unless authorized in writing by the Engineer. 9.4.3 Identification Bar reinforcement shall be shipped in standard bundles, tagged and marked in accordance with the Manual of Standard Practice of the Concrete Reinforcing Steel Institute. 9.5 HANDLING, STORING, AND SURFACE CONDITION OF REINFORCEMENT Steel reinforcement shall be stored above the surface of the ground on platforms, skids, or other supports and shall be protected from mechanical injury and surface dkterioration caused by exposure to conditions producing rust. When placed in the work, reinforcement shall be free from dirt, loose rust or scale, mortar, paint, grease, oil, or other nonmetallic coatings that reduce bond. Epoxy coatings of reinforcing steel in accord with standards in this article shall be permitted. Reinforcement shall be free from injurious defects such as cracks and laminations. Bonded rust, surface seams, surface irregularities, or mill scale will not be cause for rejection, provided the rninimum dimensions, cross-sectional area, and tensile properties of a hand wire brushed specimen meet the physical requirements for the size and grade of steel specified. Epoxy-coated reinforcing steel shall be handled and stored by methods that will not damage the epoxy coat- 9.6.2 Support Systems Reinforcing steel shall be supported in its proper position by use of precast concrete blocks, wire bar supports, supplementary bars or other approved devices. Such reinforcement supports or devices shall be of such height and placed at sufficiently frequent intervals so as to maintain the distance between the reinforcing steel and the formed surface or the top surface of deck slabs within Y 4 inch of that indicated on the plans. Platforms for the support of workers and equipment during concrete placement shall be supported directly on the forms and not on the reinforcing steel. 9.63 Precast Concrete Blocks Precast concrete blocks shall have a compressive strength not less than that of the concrete in which they 9.6.3 DIVISION n--CONSTRUCTION are to be embedded. The face of blocks in contact with forms for exposed surfaces shall not exceed 2 inches by 2 inches in size and shall have a color and texture that will match the concrete surface. When used on vertical or sloping surfaces, such blocks shall have an embedded wire for securing the block to the reinforcing steel. When used in slabs, either such a tie wire or, when the weight of the reinforcing steel is sufficient to firmly hold the blocks in place, a groove in the top of the block may be used. For epoxy-coated bars, such tie wires shall be plastic-coated or epoxy-coated.
9.6.4 Wire Bar Supports Wire bar supports, such as ferrous metal chairs and bolsters, shall conform to industry practice as described in the Manual of Standard Practice of the Concrete Reinforcing Steel Institute.
Such chairs or bolsters which bear against the forms for exposed surfaces shall be either Class 1-Maximum Protection (Plastic Protected) or Class 2, Type B-Moderate Protection (

10.3.2.3.8.2 Crack widths and crack patterns shall be recorded at the initial load of 0.8Fpu,at least three times at intervals of not less than 4 hours during the last 12 hours before termination of the sustained loading, and during loading to failure at 0.9Fpu.The maximum load shall also be reported. 10.3.2.3.9 Monotonic Loading Test 10.3.2.3.9.1 In a monotonic loading test, the load shall be increased to 0.9Fpuand held constant for 1 hour. The specimen shall then be preferably loaded to failure or, 556 HIGHWAY BRIDGES 10.3.2.3.9.1 if limited by the capacity of the loading equipment, to at least 1 .2Fpu. 10.4 PLACEMENT OF DUCTS, STEEL, AND ANCHORAGE HARDWARE 10.3.2.3.9.2 Crack widths and crack patterns shall be recorded at 0.9Fp, after the 1-hour period, and at l.OFpu. The maximum load shall also be reported. 10.4.1 Placement of Ducts 10.3.2.3.10 The strength of the anchorage zone must exceed: Specimens tested under cyclic or sustained loading . .
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. . .. IFp, Specimens tested under monotonic loading . . . . . .1.2Fpu The maximum crack width criteria specified below must be met for moderately aggressive environments. For higher aggressivity environments the crack width criteria shall be reduced by at least 50%. (1) No cracks greater than 0.010 inch at 0.8Fpu after completion of the cyclic or sustained loading, or at 0.9Fpu after the 1-hour period for monotonic loading. (2) No cracks greater than 0.016 inch at 0.9Fpufor cyclic or sustained loading, or at 1.OFpu for monotonic loading. 10.3.2.3.11 A test series shall consist of three test specimens. Each one of the tested specimens must meet the acceptance criteria. If one of the three specimens fails to pass the test, a supplementary test of three additional specimens is allowed. The three additional test specimen results must meet all acceptance criteria of Article 10.3.2.3.10. For a series of similar special anchorage devices, tests are only required for representative samples unless tests for each capacity of the anchorages in the series are required by the engineer-of-record. 10.3.2.3.12 Records of the anchorage device acceptance test shall include: Ducts shall be rigidly supported at the proper locations in the forms by ties to reinforcing steel which are adequate to prevent displacement during concrete placement. Supplementary support bars shall be used where needed to maintain proper alignment of the duct. Hold-down ties to the forms shall be used when the buoyancy of the ducts in the fluid concrete would lift the reinforcing steel. Joints between sections of duct shall be coupled with positive connections which do not result in angle changes at the joints and will prevent the intrusion of cement paste. After placing of ducts, reinforcement and forming is complete, an inspection shall be made to locate possible duct damage. All unintentional holes or openings in the duct must be repaired prior to concrete placing. Grout openings and vents must be securely anchored to the duct and to either the forms or to reinforcing steel to prevent displacement during concrete placing operations. After installation in the forms, the ends of ducts shall at all times be covered as necessary to prevent the entry of water or debris. 10.4.1.1 Vents and Drains All ducts for continuous structures shall be vented at the high points of the duct profile, except where the curvature is small, as in continuous slabs, and at additional locations as shown on the plans. Where freezing conditions can be anticipated prior to grouting, drains shall be installed at low point in ducts where needed to prevent the accumulation of water. Low-point drains shall remain open until grouting is started. The ends of vents and drains shall be removed 1 inch below the surface of the concrete after grouting has been completed, and the void filled with mortar. 10.4.2 Placement of Prestressing Steel (1) Dimensions of the test specimen. (2) Drawings and dimensions of the anchorage device, including all confining reinforcing steel. (3) Amount and arrangement of supplementary skin reinforcement. (4) Type and yield strength of reinforcing steel. (5) Type and compressive strength at time of testing of concrete. (6) Q p e of testing procedure and all measurements required in Articles 10.3.2.3.7 through 10.3.2.3.10 for each specimen. 10.4.2.1 Placement for Pretensioning Prestressing steel shall be accurately installed in the forms and held in place by the stressing jack or temporary anchors and, when tendons are to be draped, by holddown devices. The hold-down devices used at all points of change in slope of tendon trajectory shall be of an approved low-friction type. Prestressing steel shall not be removed from its protective packaging until immediately prior to installation in 10.4.2.1 557 DIVISION II-CONSTRUCTION the forms and placement of concrete. Openings in the packaging shall be resealed as necessary to protect the unused steel. While exposed, the steel shall be protected as needed to prevent corrosion. 10.4.2.2 Placement for Post-Tensioning All prestressing steel preassembled in ducts and installed prior to the placement of concrete shall be accurately placed and held in position during concrete placement.

When the prestressing steel is installed after the concrete has been placed, the Contractor shall demonstrate to the satisfaction of the Engineer that the ducts are free of water and debris immediately prior to installation of the steel. The total number of strands in an individual tendon may be pulled into the duct as a unit, or the individual strand may be pulled or pushed through the duct. Anchorage devices or block-out templates for anchorages shall be set and held so that their axis coincides with the axis of the tendon and anchor plates are normal in all directions to the tendon. The prestressing steel shall be distributed so that the force in each girder stem is equal or as required by the plans, except as provided herein. For box girders with more than two girder stems, at the Contractor's option, the prestressing force may vary up to 5% from the theoretical required force per girder stem provided the required total force in the superstructure is obtained and the force is distributed symmetrically about the center line of the typical section. 10.4.2.2.1 Protection of Steel After Installation Prestressing steel installed in members prior to placing and curing of the concrete, or installed in the duct but not grouted within the time limit specified below, shall be continuously protected against rust or other corrosion by means of a corrosion inhibitor placed in the ducts or directly applied to the steel. The prestressing steel shall be so protected until grouted or encased in concrete. Prestressing steel installed and tensioned in members after placing and curing of the concrete and grouted within the time limit specified below will not require the use of a corrosion inhibitor described herein and rust which may form during the interval between tendon installation and grouting will not be cause for rejection of the steel. The permissible interval between tendon installation and grouting without use of a corrosion inhibitor for various exposure conditions shall be as follows: Very Damp Atmosphere or over Saltwater (Humidity > 70%) 7 days Moderate Atmosphere (Humidity from 40% to 70%) 15 days Very Dry Atmosphere (Humidity < 40%) 20 days After tendons are placed in ducts, the openings at the ends of the ducts shall be sealed to prevent entry of moisture. When steam curing is used, steel for post-tensioning shall not be installed until the steam curing is completed. Whenever electric welding is performed on or near members containing prestressing steel, the welding ground shall be attached directly to the steel being welded. All prestressing steel and hardware shall be protected from weld spatter or other damage. 10.4.3 Placement of Anchorage Hardware The constructor is responsible for the proper placement of all materials according to the design documents of the engineer of record and the requirements stipulated by the anchorage device supplier. The Contractor shall exercise all due care and attention in the placement of anchorage hardware, reinforcement, concrete, and consolidation of concrete in anchorage zones. Modifications to the local zone details verified under provisions of Article 9.21.7.3. in Division I and Article 10.3.2.3 in Division 11 shall be approved by both the engineer of record and the anchorage device supplier. 10.5 IDENTIFICATION AND TESTING All wire, strand, or bars to be shipped to the site shall be assigned a lot number and tagged for identification purposes. Anchorage assemblies to be shipped shall be likewise identified. Each lot of wire or bars and each reel of strand reinforcement shall be accompanied by a manufacturer's certificate of compliance, a mill certificate, and a test report.

The mill certificate and test report shall include the chemical composition (not required for strand), cross-sectional area, yield and ultimate strengths, elongation at rupture, modulus of elasticity, and the stress strain curve for the actual prestressing steel intended for use. All values certified shall be based on test values and nominal sectional areas of the material being certified. The Contractor shall furnish to the Engineer for verification testing the samples described in the following subarticles selected from each lot. If ordered by the Engineer, 558 HIGHWAY BRIDGES the selection of samples shall be made at the manufacturer's plant by the Inspector. All samples submitted shall be representativeof the lot to be furnished and, in the case of wire or strand, shall be taken from the same master roll. The actual strength of the prestressing steel shall not be less than specified by the applicable ASTM Standard, and shall be determined by tests of representative samples of the tendon material in conformance with ASTM Standards. of the 'pecified for testing be furnished free of cost and shall be delivered in time for tests to be made well in advance of anticipated time of use. 10.5.1 Pretensioning Method Tendons For pretensioned strands, one sample at least 7 feet long shall be furnished in accordance with the requirements of paragraph 9.1 of AASHTO M 203. 10.5.2 Post-Tensioning Method Tendons ne fOfolwinglengths be furnished for each 20 ton, or portion thereof, lot of material used in the work. (a) For wires requiring heading-5 feet. (b) For wires not requiring heading-sufficient length to make up one parallel-lay cable 5 feet long consisting of the same number of wires as the cable to be furnished. (c) For strand to be furnished with fittings-5 feet between near ends of fittings. (d) For bars to be furnished with threaded ends and nuts-5 feet between threads at ends. 10.5.3 Anchorage Assemblies and Couplers The Contractor shall furnish for testing, one specimen of each size of prestressing tendon, including couplings, of the selected type, with end fittings and anchorage assembly attached, for strength tests only. These specimens shall be 5 feet in clear length, measured between ends of fittings. If the results of the test indicate the necessity of check tests, additional specimens shall be furnished without cost. When dynamic testing is required, the Contractor shall perform the testing and shall furnish certified copies of test results which indicate conformance with the specified requirements prior to installation of anchorages or couplers. For prestressing systems previously tested and approved on projects having the same tendon configuration, the Engineer may not require complete tendon samples 10.5 provided there is no change in the material, design, or details previously approved. Shop drawings or prestressing details shall identify the project on which approval was obtained, otherwise testing shall be conducted. PROTECTION OF STEEL All prestressing steel shall be protected against physical damage and rust or other results of corrosion at all times from manufacture to grouting. Prestressing steel shall also be free of deleterious such as grease, oil, wm, or paint. Prestressing steel that has sustained physical damage at any time shall be rejected. The development of pitting or other results of corrosion, other than rust stain~shallbe cause for rejection. Prestressing steel shall be packaged in containers or shipping forms for the protection of the strand against physical damage and corrosion during shipping and storage. A corrosion inhibitor which prevents rust or other results of corrosion shall be placed in the package or form, or shall be incorporated in a corrosion inhibitor carrier type packaging material, or when permitted by the Engineer, may be applied directly to the steel. The corrosion inhibitor shall have no deleterious effect on the steel or concrete or bond strength of steel to concrete or grout. Packaging or forms damaged from any cause shall be immediately replaced or restored to original condition. The shipping package or form shall be clearly marked with a statement that the package contains high-strength prestressing steel, and the type of corrosion inhibitor used, including the date packaged. All anchorages, end fittings, couplers, and exposed tendons, which will not be encased in concrete or grout in the completed work, shall be permanently protected against corrosion. 10.7 CORROSION INHIBITOR Corrosion inhibitor shall consist of a vapor phase inhibitor (VPI) powder conforming to the provisions of Federal Specification MIL-P-3420 or as otherwise approved by the Engineer. When approved, water soluble oil may be used on tendons as a corrosion inhibitor. 10.8 DUCTS Ducts used to provide holes or voids in the concrete for the placement of post-tensioned bonded tendons may be either formed with removable cores or may consist of rigid or semi-rigid ducts which are cast into the concrete. 10.8 DIVISION 11--CONSTRUCTION Ducts formed with removable cores shall be formed with no constrictions which would tend to block the passage of grout. All coring materials shall be removed. Ducts formed by sheath left in place shall be a type that will not permit the intrusion of cement paste. They shall transfer bond stresses as required and shall retain shape under the weight of the concrete and shall have sufficient strength to maintain their correct alignment without visible wobble during placement of concrete. 10.8.1 Metal Ducts Sheathing for ducts shall be metal, except as provided herein. Such ducts shall be galvanized ferrous metal and shall be fabricated with either welded or interlocked seams. Galvanizing of welded seams will not be required. Rigid ducts shall have smooth inner walls and shall be capable of being curved to the proper configuration without crimping or flattening. Semi-rigid ducts shall be corrugated and when tendons are to be inserted after the concrete has been placed their minimum wall thickness shall be as follows: 26 gauge for ducts less than or equal to 2%-inch diameter, 24 gauge for ducts greater than 2%-inch diameter. When bar tendons are preassembled with such ducts, the duct thickness shall not be less than 31 gauge. 10.8.2 Polyethylene Duct As an alternative to metal ducts, ducts for transverse tendons in deck slabs and at other locations where shown or approved may be of high density polyethylene, conforming to the material requirements of ASTM D 3350. Polyethyleneduct shall not be used when the radius of curvature of the tendon is less than 30 feet. Semi-rigid polyethylene ducts for use where completely embedded in concrete shall be corrugated with minimum material thickness of 0.050+0.010 inch. Such ducts shall have a white coating on the outside, or shall be of white material with ultraviolet stabilizers added. Rigid polyethyleneducts for use where the tendon is not embedded in concrete shall be rigid pipe manufactured in accordance with ASTM D 2447, Grades P33 or P34; F714 or D3350 with a cell classification of PE345433C. For external applications, such duct shall have an external diameter to wall thickness ratio of 21 or less. For applications where polyethylene duct is exposed to sunlight or ultraviolet light, carbon black shall be incorporated into the polyethylene pipe resin in such amount to provide resistanct to ultraviolet degradation in accordance with ASTM D 1248. 559 10.8.3 Duct Area The inside diameter of ducts shall be at least Y4 inch larger than the nominal diameter of single wire, bar, or strand tendons, or in the case of multiple wire, bar or strand tendons, the inside cross-sectional area of the sheathing shall be at least two times the net area of the prestressing steel. When tendons are to be placed by the pull through method, the duct area shall be at least 2% times the net area of the prestressing steel.

10.8.4 Duct Fittings Coupling and transition fittings for ducts formed by sheathing shall be of either ferrous metal or polyethylene, and shall be cement paste intrusion proof and of sufficient strength to prevent distortion or displacement of the ducts during concrete placement. All ducts or anchorage assemblies shall be provided with pipes or other suitable connections at each end of the duct for the injection of grout after prestressing. As specified in Article 10.4.1.1, ducts shall also be provided with ports for venting or grouting at high points and for draining at intermediate low points. Vent and drain pipes shall be %-inch minimum diameter standard pipe or suitable plastic pipe. Connection to ducts shall be made with metallic or plastic structural fasteners. The vents and drains shall be mortar tight, taped as necessary, and shall provide means for injection of grout through the vents and for sealing to prevent leakage of grout. 10.9 GROUT Materials for use in making grout which is to be placed in the ducts after tendons are post-tensioned shall conform to the following. 10.9.1 Portland Cement Portland cement shall conform to one of the following: Specifications for Portland Cement-AASHTO M 85 (ASTM C 150),Types I, 11, or III.Cement used for grouting shall be fresh and shall not contain any lumps or other indication of hydration or "pack set." 10.9.2 Water The water used in the grout shall be potable, clean, and free of injurious quantities of substances known to be harmful to Portland cement or prestressing steel. 560 HIGHWAY BRIDGES 10.9.3 Admixtures Admixtures, if used, shall impart the properties of lowwater content, good flowability, minimum bleed, and expansion if desired. They shall contain no chemicals in quantities that may have harmful effect on the prestressing steel or cement. Admixtures which, at the dosage used, contain chlorides in excess of 0.005% of the weight of the cement used or contain any fluorides, sulphites, and nitrates shall not be used. When a grout expanding admixture is required, or is used at the Contractor's option, it shall be well dispersed through the other admixtures and shall produce a 2% to 6% unrestrained expansion of the grout. Amount of admixture to obtain a desired amount of expansion shall be determined by tests. If the source of manufacture or brand of either admixture or cement changes after testing, new tests shall be conducted to determine proper proportions. All admixtures shall be used in accordance with the instructions of the manufacturer. 10.10 TENSIONING . 10.10.1 General Tensioning Requirements Prestressing steel shall be tensioned by hydraulic jacks so as to produce the forces shown on the plans or on the approved working drawing with appropriate allowances for all losses. Losses to be provided for shall be as specified in Division I, Article 9.16. For post-tensioned work the losses shall also include the anchor set loss appropriate for the anchorage system employed. For pretensioned members, the strand stress prior to seating (jacking stress) shall not exceed 80% of the minimum ultimate tensile strength of the prestressing steel (0.80 f,'). This allowable stress, which slightly exceeds the values allowed in Division I, Article 9.15.1, may be permitted to offset seating losses and to accommodate compensation for temperature differences specified in Article 10.5.2. For post-tensioned members, the strand stress prior to seating (jacking stress) and the stress in the steel immediately after seating shall not exceed the values allowed in Division I, Article 9.15.1. The method of tensioning employed shall be one of the following as specified or approved: (1) Pretensioning; in which the prestressing strand or tendons are stressed prior to being embedded in the concrete placed for the member. After the concrete has attained the required strength, the prestressing force is 10.9.3 released from the external anchorages and transferred, by bond, into the concrete. (2) Post-tensioning; in which the reinforcing tendons are installed in voids or ducts within the concrete and are stressed and anchored against the concrete after the development of the required concrete strength.

As a final operation under this method, the voids or ducts are pressure-grouted. (3) Combined Method; in which part of the reinforcement is pretensioned and part post-tensioned. Under this method all applicable requirements for pretensioning and for post-tensioning shall apply to the respective reinforcing elements using these methods. During stressing of strand, individual wire failures may be accepted by the Engineer, provided not more than one wire in any strand is broken and the area of broken wires does not exceed 2% of the total area of the prestressing steel in the member. 10.10.1.1 Concrete Strength Prestressing forces shall not be applied or transferred to the concrete until the concrete has attained the strength specified for initial stressing. In addition, cast-in-place concrete for other than segmentally constructed bridges shall not be post-tensioned until at least 10 days after the last concrete has been placed in the member to be posttensioned. 10.10.1.2 Prestressing Equipment Hydraulic jacks used to stress tendons shall be capable of providing and sustaining the necessary forces and shall be equipped with either a pressure gauge or a load cell for determining the jacking stress. The jacking system shall provide an independent means by which the tendon elongation can be measured. The pressure gauge shall have an accurately reading dial at least 6 inches in diameter or a digital display, and each jack and its gauge shall be calibrated as a unit with the cylinder extension in the approximate position that it will be at final jacking force, and shall be accompanied by a certified calibration chart or curve. The load cell shall be calibrated and shall be provided with an indicator by means of which the prestressing force in the tendon may be determined. The range of the load cell shall be such that the lower 10% of the manufacturer's rated capacity will not be used in determining the jacking stress. When approved by the Engineer, calibrated proving rings may be used in lieu of load cells. Recalibration of gauges shall be repeated at least annually and whenever gauge pressures and elongations indicate materially different stresses. DIVISION 11--C(INSTRUCTION 10.10.1.2 Only oxygen flame or mechanicalcutting devices shall be used to cut strand after installation in the member or after stressing. Electric arc welders shall not be used. 10.10.1.3 Sequence of Stressing When the sequence of stressing individual tendons is not otherwise specified, the stressing of post-tensioning tendons and the release of pretensioned tendons shall be done in a sequence that produces a minimum of eccentric force in the member. 10.10.1.4 Measurement of Stress Arecord of gauge pressures and tendon elongations for each tendon shall be provided by the Contractor for review and approval by the Engineer. Elongations shall be measured to an accuracy of +%6 inch. Stressing tails of post-tensioned tendons shall not be cut off until the stressing records have been approved. The stress in tendons during tensioning shall be determined by the gauge or load cell readings and shall be verified with the measured elongations. Calculations of anticipated elongations shall utilize the modulus of elasticity, based on nominal area, as furnished by the manufacturer for the lot of steel being tensioned, or as determined by a bench test of strands used in the work. All tendons shall be tensioned to a preliminary force as necessary to eliminate any take-up in the tensioning system before elongation readings are started. This preliminary force shall be between 5% and 25% of the final jacking force. The initial force shall be measured by a dynamometer or by other approved method, so that its amount can be used as a check against elongation as computed and as measured. Each strand shall be marked prior to final stressing to permit measurement of elongation and to insure that all anchor wedges set properly. It is anticipated that there may be discrepancy in indicated stress between jack gauge pressure and elongation. In such event, the load used as indicated by the gauge pressure, shall produce a slight over-stress rather than under-stress.When a discrepancy between gauge pressure and elongation of more than 5% in tendons over 50 feet long or 7% in tendons of 50 feet or less in length occurs, the entire operation shall be carefully checked and the source of error determined and corrected before proceeding further. When provisional ducts are provided for addition of prestressing force in event of an apparent force deficiency in tendons over 50 feet long, the discrepancy between the force indicated by gauge pressure and elongation may be increased to 7% before investigation into the source of the error. 10.10.2 561 Pretensioning Method Requirements Stressing shall be accomplished by either single strand stressing or multiple strand stressing. The amount of stress to be given each strand shall be as shown in the plans or the approved working drawings. All strand to be stressed in a group (multiple strand stressing) shall be brought to a uniform initial tension prior to being given their full pretensioning. The amount of the initial tensioning force shall be within the range specified in Article 10.5.1 and shall be the minimum required to eliminate all slack and to equalize the stresses in the tendons as determined by the Engineer. The amount of this force will be influenced by the length of the casting bed and the size and number of tendons in the group to be tensioned. Draped pretensioned tendons shall either be tensioned partially by jacking at the end of the bed and partially by uplifting or depressing tendons, or they shall be tensioned entirely by jacking, with the tendons being held in their draped positions by means of rollers, pins, or other approved methods during the jacking operation. Approved low-friction devices shall be used at all points of change in slope of tendon trajectory when tensioning draped pretensioned strands, regardless of the tensioning method used.

If the load for a draped strand, as determined by elongation measurements,is more than 5% less than that indicated by the jack gauges, the strand shall be tensioned from both ends of the bed and the load as computed from the sum of elongation at both ends shall agree within 5% of that indicated by the jack gauges. When ordered by the Engineer, prestressing steel strands in pretensioned members, if tensioned individually, shall be checked by the Contractor for loss of prestress not more than 3 hours prior to placing concrete for the members. The method and equipment for checking the loss of prestress shall be subject to approval by the Engineer. All strands that show a loss of prestress in excess of 3% shall be retensioned to the original computed jacking stress.

Stress on all strands shall be maintained between anchorages until the concrete has reached the compressive strength required at time of transfer of stress to concrete. When prestressing steel in pretensioned members is tensioned at a temperature more than 25°F lower than the estimated temperatureof the concrete and the prestressing steel at the time of initial set of the concrete, the calculated elongation of the prestressing steel shall be increased to compensate for the loss in stress, due to the change in temperature, but in no case shall the jacking stress exceed 80% of the specified minimum ultimate tensile strength of the prestressing steel.

562 HIGHWAY BRIDGES Strand splicing methods and devices shall be approved by the Engineer. When single strand jacking is used, only one splice per strand will be permitted. When multi-strand jacking is used, either all strands shall be spliced or no more than 10% of the strands shall be spliced. Spliced strands shall be similar in physical properties, from the same source, and shall have the same "twist" or "lay." All splices shall be located outside of the prestressed units. Side and flange forms that restrain deflection shall be removed before release of pretensioning reinforcement. Except when otherwise shown on the plans, all pretensioned-prestressing strands shall be cut off flush with the end of the member and the exposed ends of the strand and a 1-inch strip of adjoining concrete shall be cleaned and painted. Cleaning shall be by wire brushing or abrasive blast cleaning to remove all dirt and residue that is not firmly bonded to the metal or concrete surfaces. The surfaces shall be coated with one thick coat of zinc-rich paint conforming to the requirements of Federal Specification IT-P-641. The paint shall be thoroughly mixed at the time of application, and shall be worked into any voids in the strands. 10.10.3 Post-TensioningMethod Requirements Prior to post-tensioning any member, the Contractor shall demonstrate to the satisfaction of the Engineer that the prestressing steel is free and unbonded in the duct. All strands in each tendon, except for thosein flat ducts with not more than four strands, shall be stressed simultaneously with a multi-strand jack. Tensioning shall be accomplished so as to provide the forces and elongations specified in Article 10.5.1. Except as provided herein or when shown on the plans or on the approved working drawings, tendons in continuous post-tensioned members shall be tensioned by jacking at each end of the tendon. For straight tendons and when one end stressing is shown on the plans, tensioning may be performed by jacking from one end or both ends of the tendon at the option of the Contractor. 10.11 GROUTING 10.11.1 General When the post-tensioning method is used, the prestressing steel shall be provided with permanent protection and shall be bonded to the concrete by completely filling the void space between the duct and the tendon with grout. 10.10.2 10.11.2 Preparation of Ducts All ducts shall be clean and free of deleterious materials that would impair bonding or interfere with grouting procedures. Ducts with concrete walls (cored ducts) shall be flushed to ensure that the concrete is thoroughly wetted. Metal ducts shall be flushed if necessary to remove deleterious material. Water used for flushing ducts may contain slack lime (calcium hydroxide) or quicklime (calcium oxide) in the amount of 0.1 lb per gallon. After flushing, all water shall be blown out of the duct with oil-free compressed air.

10.11.3 Equipment The grouting equipment shall include a mixer capable of continuous mechanical mixing which will produce a grout free of lumps and undispersed cement, a grout pump and standby flushing equipment with water supply. The equipment shall be able to pump the mixed grout in a manner which will comply with all requirements. Accessory equipment which will provide for accurate solid and liquid measures shall be provided to batch all materials. The pump shall be a positive displacement type and be able to produce an outlet pressure of at least 150 psi. The pump should have seals adequate to prevent introduction of oil, air, or other foreign substance into the grout, and to prevent loss of grout or water. A pressure gauge having a full-scale reading of no greater than 300 psi shall be placed at some point in the grout line between the pump outlet and the duct inlet. The grouting equipment shall contain a screen having clear openings of 0.125-inch maximum size to screen the grout prior to its introduction into the grout pump. If a grout with a thixotropic additive is used, a screen opening of 3/16 inch is satisfactory. This screen shall be easily accessible for inspection and cleaning. The grouting equipment shall utilize gravity feed to the pump inlet from a hopper attached to and directly over it. The hopper must be kept at least partially full of grout at all times during the pumping operation to prevent air from being drawn into the post-tensioning duct. Under normal conditions, the grouting equipment shall be capable of continuously grouting the largest tendon on the project in no more than 20 minutes. 10.11.4 Miving of Grout Water shall be added to the mixer first, followed by Portland cement and admixture, or as required by the admixture manufacturer. 10.11.4 DIVISION 11--CONSTRUCTION Mixing shall be of such duration as to obtain a uniform, thoroughly blended grout, without excessive temperature increase or loss of expansive properties of the admixture. The grout shall be continuously agitated until it is pumped. Water shall not be added to increase grout flowability which has been decreased by delayed use of the grout. Proportions of materials shall be based on tests made on the grout before grouting is begun, or may be selected based on prior documented experience with similar materials and equipment and under comparable field conditions (weather, temperature, etc.). The water content shall be the minimum necessary for proper placement, and when 'Qpe I or I1 cement is used shall not exceed a watercement ratio of 0.45 or approximately 5 gallons of water per sack (94 lb) of cement. The water content required for Type III cement shall be established for a particular brand based on tests. The pumpability of the grout may be determined by the Engineer in accordance with the U.S. Corps of Engineers Method CRD-C79. When this method is used, the efflux time of the grout sample immediately after mixing shall not be less than 11 seconds. fieflow cone test does not apply to grout which incorporates a thixotropic additive. 563 and the pumping pressure allowed to build to a minimum of 75 psi before the inlet vent is closed. Plugs, caps, or valves thus required shall not be removed or opened until the grout has set. 10.11.6 Temperature Considerations When temperatures are below 32OF, ducts shall be kept free of water to avoid damage due to freezing. The temperatureof the concrete shall be 35°F or higher from the time of grouting until job cured 2-inch cubes of grout reach a minimum compressive strength of 800 psi. Grout shall not be above 90°F during mixing or pumping. If necessary, the mixing water shall be cooled. 10.12 MEASUREMENT AND PAYMENT Measurement The prestressing of cast-in-place concrete will be measured by the lump sum for each item or location listed in the bid items. 10.11.5 Injection of Grout 10.12.2 Payment All grout and high-point vent openings shall be open when grouting starts. Grout shall be allowed to flow from the first vent after the inlet pipe until any residual flushing water or entrapped air has been removed, at which time the vent should be capped or otherwise closed. Remaining vents shall be closed in sequence in the same manner. The pumping pressure at the tendon inlet shall not exceed 250 psi. If the actual grouting pressure exceeds the maximum recommended pumping pressure, grout may be injected at any vent which has been, or is ready to be capped as long as a one-way flow of grout is maintained. If this procedure is used, the vent which is to be used for injection shall be fitted with a positive shutoff. When one-way flow of grout cannot be maintained, the grout shall be immediately flushed out of the duct with water. Grout shall be pumped through the duct and continuously wasted at the outlet pipe until no visible slugs of water or air are ejected and the efflux time of the ejected grout, as measured by a flow cone test, if used, is not less than that of the injected grout. To ensure that the tendon remains filled with grout, the outlet shall then be closed No separate payment will be made for prestressing precast concrete members. Payment for prestressing precast concrete members shall be considered as included in the contract price paid for the precast members as provided for in Section 8, "Concrete Structures." The contract lump sum price paid for prestressing castin-place concrete shall include full compensation for furnishing all labor, materials, tools, equipment and incidentals, and for doing all work involved in furnishing, placing, and tensioning the prestressing steel in cast-inplace concrete structures, complete in place, as shown on the plans, as specified in these Specifications and the special provisions, and as directed by the Engineer. Full compensation for furnishing and placing additional concrete and deformed bar reinforcing steel required by the particular system used, ducts, anchoring devices, distribution plates or assemblies and incidental parts, for furnishing samples for testing, working drawings, and for pressure grouting ducts shall be considered as included in the contract lump sum price paid for prestressing cast-in-place concrete or in the contract price for furnishing precast members, and no additional compensation will be allowed therefore. Section 11 STEEL STRUCTURES This work shall consist of furnishing, fabricating, and erecting steel structures and structural steel portions of other structures in accordance with these Specifications, the Special Provisions and the details shown on the plans. The structural steel fabricating plant shall be certified under the AISC Quality Certification Program, Category I.

The fabrication of fracture critical members shall be Category III. Details of design which are permitted to be selected by the Contractor shall conform to Division I of these Specifications.

Painting shall conform to the provisions of Section 13, "Painting," of these Specifications. Falsework used in the erection of structural steel shall conform to the provisions of Section 3, "Temporary Works," of these Specifications. Structural components designated on the plans or in the special provisions as "fracture critical" shall conform to the provisions of Chapter 12 of the ANSI/'SHTO/ AWS D1.5 Bridge Welding Code. Welding and weld qualification tests shall conform to the provisions of the current ANSI/AASHTO/AWS D1.5 Bridge Welding Code. mill orders and certified mill test reports. Mill test reports shall show the chemical analysis and physical test results for each heat of steel used in the work. With the approval of the Engineer, certificates of compliance shall be furnished in lieu of mill test reports for material that normally is not supplied with mill test reports, and for items such as fills, minor gusset plates and similar material when quantities are small and the material is taken from stock. Certified mill test reports for steels with specified impact values shall include, in addition to other test results, the results of Charpy V-notch impact tests. When fine grain practice is specified, the test report shall confirm that the material was so produced. Copies of mill orders shall be furnished at the time orders are placed with the manufacturer. Certified mill test reports and Certificates of Compliance shall be furnished prior to the start of fabrication of material covered by these reports. The Certificate of Compliance shall be signed by the manufacturer and shall certify that the material is in conformance with the specifications to which it has been manufactured. Material to be used shall be made available to the Engineer so that each piece can be examined.

The Engineer shall have free access at all times to any portion of the fabrication site where the material is stored or where work on the material is being performed. 11.1.2 Notice of Beginning of Work 11.1.4 Inspector's Authority The Contractor shall give the Engineer ample notice of the beginning of work at the mill or in the shop, so that inspection may be provided. The term "mill" means any rolling mill or foundry where material for the work is to be manufactured. No material shall be manufactured, or work done in the shop, before the Engineer has been so notified. The Inspector shall have the authority to reject materials or workmanship which do not fulfill the requirements of these Specifications. In cases of dispute, the Contractor may appeal to the Engineer, whose decision shall be final. Inspection at the mill and shop is intended as a means of facilitating the work and avoiding errors, and it is expressly understood that it will not relieve the Contractor of any responsibility in regard to defective material or workmanship and the necessity for replacing the same. The acceptance of any material or finished members by the Inspector shall not be a bar to their subsequent rejection, if found defective. Rejected materials and workman- 11.1 GENERAL 11.1.1 Description 11.1.3 Inspection Structural steel will be inspected at the fabrication site. The Contractor shall furnish to the Engineer a copy of all 566 HIGHWAY BRIDGES ship shall be replaced as soon as practical or corrected by the Contractor.

11.2 WORKING DRAWINGS The Contractor shall expressly understand that the Engineer's approval of the working drawings submitted by the Contractor covers the requirements for "strength and detail," and that the Engineer assumes no responsibility for errors in dimensions. Working drawings must be approved by the Engineer prior to performance of the work involved and such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work. 11.2.1 Shop Drawings The Contractor shall submit copies of the detailed shop drawings to the Engineer for approval. Working drawings shall be submitted sufficientlyin advance of the start of the affected work to allow time for review by the Engineer and corrections by the Contractor without delaying the work. Working drawings for steel structures shall give full detailed dimensions and sizes of component parts of the structure and details of all miscellaneous parts, such as pins, nuts, bolts, drains, etc. Where specific orientation of plates is required, the direction of rolling of plates shall be shown. Working drawings shall specifically identify each piece that is to be made of steel which is to be other than AASHTO M 270 (ASTM A 709) Grade 36 steel. 11.2.2 Erection Drawings The Contractor shall submit drawings illustrating fully his or her proposed method of erection. The drawings shall show details of all falsework bents, bracing, guys, dead-men, lifting devices, and attachments to the bridge members: sequence of erection, location of cranes and barges, crane capacities, location of lifting points on the bridge members, and weights of the members. The plan and drawings shall be complete in detail for all anticipated phases and conditions during erection. Calculations may be required to demonstrate that allowable stresses are not exceeded and that member capacities and final geometry will be correct. 11.2.3 Camber Diagram A camber diagram shall be furnished to the Engineer by the Fabricator, showing the camber at each panel point 11.1.4 in the cases of trusses or arch ribs, and at the location of field splices and fractions of span length ('hpoints minimum) in the cases of continuous beam and girders or rigid frames. The camber diagram shall show calculated cambers to be used in preassembly of the structure in accordance with Article 11.5.3. 11.3 MATERIALS 11.3.1 Structural Steel 11.3.1.1 General Steel shall be furnished according to the following specifications.The grade or grades of steel to be furnished shall be as shown on the plans or specified. All steel for use in main load-carrying member components subject to tensile stresses shall conform to the applicable Charpy V-notch Impact Test requirements of AASHTO M 270 (ASTM A 709). Welded girders made of ASTM A 709, Grade HPS70W steels shall be fabricated in accordance with the AASHTO Guide Specijications for Highway Bridge Fabrication with HPS70W Steel, which supplements the ANSVAASHTO/AWS D1.5 Bridge Welding Code. 11.3.1.2 Carbon Steel Unless otherwise specified, structural carbon steel for bolted or welded construction shall conform to: Structural Steel for Bridges, AASHTO M 270 (ASTM A 709) Grade 36. 11.3.1.3 High-Strength LOW-AUO~ Structural Steel High-strength low-alloy steel shall conform to: Structural Steel for Bridges, AASJ3TO M 270 (ASTM A 709) Grades 50 or 50W. 11.3.1.4 High-Strength Low-Alloy, Quenched and Tempered Structural Steel Plate High-strength, low-alloy quenched and tempered steel plate shall conform to AASHTO M 270 (ASTM A 709) Grade 70W, or Grade HPS70W. 11.3.1.5 High-Yield Strength, Quenched and Tempered Alloy Steel Plate High-yield strength, quenched, and tempered alloy steel plate shall conform to: 11.3.1.5 DIVISION II-CONSTRUCTION (a) Structural Steel for Bridges, AASHTO M 270 (ASTM A 709) Grades 100 or 100W. (b) Quenched and tempered alloy steel structural shapes and seamless mechanical tubing, meeting all of the mechanical and chemical requirements of AASHTO M 270 (ASTM A 709) Grades 100 or lOOW steel, except that the specified maximum tensile strength may be 140,000 psi for structural shapes and 145,000 psi for seamless mechanical tubing, shall be considered as AASHTO M 270 (ASTM A 709) Grades 100 and lOOW steel. 11.3.1.6 Eyebars Steel for eyebars shall be of a weldable grade.

These grades include structural steel conforming to: (a) Structural Steel for Bridges, AASHTO M 270 (ASTM A 709) Grade 36. (b) Structural Steel for Bridges, AASHTO M 270 (ASTM A 709) Grades 50 and 50W. 567 (ASTM A 563) Grades DH, DH3, C, C3, and D. Nuts for AASHTO M 253 (ASTM A 490) bolts shall conform to the requirements of AASHTO M 291 (ASTM A 563) Grades DH and DH3. Nuts to be galvanized (hot-dip or mechanically galvanized) shall be heat treated Grade DH or DH3.

Plain (ungalvanized) nuts shall have a minimum hardness of 89 HRB. Nuts to be used with AASHTO M 164 (ASTM A 325) Type 3 bolts shall be of Grade C3 or DH3. Nuts to be used with AASHTO M 253 (ASTM A 490) bolts shall be of Grade DH3. All galvanized nuts shall be lubricated with a lubricant containing a visible dye. Black bolts must be oily to touch when delivered and installed. Washers shall be hardened steel washers conforming to the requirements of AASHTO M 293 (ASTM F 436) and Article 11.5.6.4.3. 11.3.1.7 Structural 'Ihbing 11.3.2.2 Identifying Marks Structural tubing shall be either cold-formed welded or seamless tubing conforming to ASTM 500, Grade B or hot-formed welded or seamless tubing conforming to ASTM 501. 11.3.2 High-Strength Fasteners 11.3.2.1 Material High-strength bolts for structural steel joints shall conform to either AASHTO M 164 (ASTM A 325) or AASHTO M 253 (ASTM A 490). When high-strength bolts are used with unpainted weathering grades of steel, the bolts shall be Type 3. The supplier shall provide a lot number appearing on the shipping package and a certification noting when and where all testing was done, including rotational capacity tests, and zinc thickness when galvanized bolts and nuts are used. The maximum hardness for AASHTO M 164 (ASTM A 325) b

11.4.8 Bolt Holes 11.4.8.1 Holes for High-Strength Bolts and Unfinished Bolts* 11.4.8.1.1 General All holes for bolts shall be either punched or drilled except as noted herein. Material fonning parts of a member *See Article 11.5.5 for bolts included in designation "Unfinished Bolts." 572 HIGHWAY BRIDGES composed of not more than five thicknesses of metal may be punched 1/16 inch larger than the nominal diameter of the bolts whenever the thickness of the material is not greater than 3/4 inch for structural steel, S/s inch for highstrength steel or 1/2 inch for quenched and tempered alloy steel, unless subpunching and reaming are required under Article 11.4.8.5. When material is thicker than 3/4 inch for structural steel, 5h inch for high-strength steel, or 1/2 inch for quenched and tempered alloy steel, all holes shall either be subdrilled and reamed or drilled full size. Also, when more than five thicknesses are joined, or as required by Article 11.4.8.5, material shall be subdrilled and reamed or drilled full size while in assembly. When required, all holes shall beeither subpunched or subdrilled (subdrilled if thickness limitation governs) 3/16 inch smaller and, after assembling, reamed 1/16 inch larger or drilled full size to l/16 inch larger than the nominal diameter of the bolts. When shown on the plans, enlarged or slotted holes are allowed with high-strength bolts. 11.4.8.1.2 Punched Holes The diameter of the die shall not exceed the diameter of the punch by more than 1/16 inch. If any holes must be enlarged to admit the bolts, such holes shall be reamed. Holes must be clean cut without tom or ragged edges. The slightly conical hole that naturally results from punching operations is considered acceptable. 11.4.8.1.3 Reamed or Drilled Holes Reamed or drilled holes shall be cylindrical, perpendicular to the member, and shall comply with the requirements of Article 11.4.8.1.1 as to size. Where practical, reamers shall be directed by mechanical means. Burrs on the outside surfaces shall be removed. Reaming and drilling shall be done with twist drills, twist reamers or rotobroach cutters. Onnecting parts requiring reamed or drilled holes shall be assembled and securely held while being reamed or drilled and shall be match marked before disassembling. 11.4.8.1.4 Accuracy of Holes Holes not more than 1/32 inch larger in diameter than the true decimal equivalent of the nominal diameter that may result from a drill or reamer of the nominal diameter are considered acceptable. The width of slotted holes which are produced by flame cutting or a combination of drilling or punching and flame cutting shall generally be not more than 1/32inchgreater than the nominal width. The flame cut surface shall be ground smooth. 11.4.8.1.1 11.4.8.2 Accuracy of Hole Group 11.4.8.2.1 Accuracy- Before - ReaminnAll holes punched full size, subpunched, or subdrilled shall be so accurately punched that after assembling (before any reaming is done) a cylindrical pin '/a inch smaller in diameter than the nominal size of the punched hole may be entered perpendicular to the face of the member, without drifting, in at least 75% of the contiguous holes in the same plane. If the requirement is not fulfilled, the badly punched pieces will be rejected. If any hole will not pass a pin 3/16 inch smaller in diameter than the nominal size of the punched hole, this will be cause for rejection. 11.4.8.2.2 Accuracy After Reaming When holes are reamed or drilled, 85% of the holes in any contiguous group shall, after reaming or drilling, show no offset greater than 1/32 inch between adjacent thicknesses of metal. AU steel templates shall have hardened steel bushings in holes accurately dimensioned from the center lines of the connection as inscribed on the template. The center lines shall be used in locating accurately the template from the milled or scribed ends of the members. 11.4.8.3 Numerically Controlled Drilled Field Connections In lieu of subsized holes and reaming while assembled, or drilling holes full-size while assembled, the Contractor shall have the option to drill or punch bolt holes full-size in unassembled pieces andlor connections including templates for use with matching subsized and reamed holes, by means of suitable numerically controlled (NIC) drilling or punching equipment. Full-sized punched holes shall meet the requirements of Article 11.4.8.1. If NIC drilling or punching equipment is used, the Contractor, by means of check assemblies, will be required to demonstrate the accuracy of this drilling or punching procedure in accordance with the provisions of Article 11.5.3.3. Holes drilled or punched by NIC equipment shall be drilled or punched to appropriate size either through individual pieces, or drilled through any combination of pieces held tightly together. 11.4.8.4 Holes for Ribbed Bolts, Turned Bolts, or Other Approved Bearing Type Bolts All holes for ribbed bolts, turned bolts, or other approved bearing-type bolts shall be subpunched or subdrilled %6 inch smaller than the nominal diameter of the DMSION 11--C(INSTRUCTION bolt and reamed when assembled, or drilled to a steel template or, after assembling, drilled from the solid at the option of the Fabricator. In any case the finished holes shall provide a driving fit as specified on the plans or in the special provisions.

11.4.8.5 Preparation of Field Connections Holes in all field connections and field splices of main member of trusses, arches, continuous beam spans, bents, towers (each face), plate girders, and rigid frames shall be subpunched or subdrilled and subsequently reamed while assembled or drilled full size through a steel template while assembled. Holes for field splices of rolled beam stringers continuous over floor beams or cross frames may be drilled full size unassembled to a steel template. All holes for floor beams or cross frames may be drilled full size unassembled to a steel template, except that all holes for floor beam and stringer field end connections shall be subpunched and reamed while assembled or drilled full size to a steel template. Reaming or drilling full size of field connection holes through a steel template shall be done after the template has been located with utmost care as to position and angle and f d y bolted in place. Templates used for reaming matching members, or the opposite faces of a single member shall be exact duplicates. Templates used for connections on like parts or members shall be so accurately located that the parts or members are duplicates and require no match-marking. For any connection, in lieu of subpunching and reaming or subdrilling and reaming, the fabricator may, at his option, drill holes full size with all thicknesses or material assembled in proper position. 11.4.9 Pins and Rollers 11.4.9.1 General Pins and rollers shall be accurately turned to the dimensions shown on the drawings and shall be straight, smooth, and fiee from flaws. Pins and rollers more than 9 inches in diameter shall be forged and annealed.

Pins and rollers 9 inches or less in diameter may be either forged and annealed or cold-finished carbon-steel shafting. In pins larger than 9 inches in diameter, a hole not less than 2 inches in diameter shall be bored full length along the axis after the forging has been allowed to cool to a temperature below the critical range, under suitable conditions to prevent injury by too rapid cooling, and before being annealed. 11.4.9.2 Boring Pin Holes Pin holes shall be bored true to the specified diameter, smooth and straight, at right angles with the axis of the member and parallel with each other unless otherwise required. The final surface shall be produced by a finishing cut.

The diameter of the pin hole shall not exceed that of the pin by more than 1/50 inch for pins 5 inches or less in diameter, or by 1/32 inch for larger pins. The distance outside to outside of end holes in tension members and inside to inside of end holes in compression members shall not vary from that specified more than 1/32 inch.

Boring of pin holes in built-up members shall be done after the member has been assembled. 11.4.9.3 Threads for Bolts and Pins Threads for all bolts and pins for structural steel construction shall conform to the United Standard Series UNC ANSI B1.1, Class 2A for external threads and Class 2B for internal threads, except that pin ends having a diameter of 1% inches or more shall be threaded six threads to the inch. 11.4.10 Eyebars Pin holes may be flame cut at least 2 inches smaller in diameter than the finished pin diameter.

All eyebars that are to be placed side by side in the structure shall be securely fastened together in the order that they will be placed on the pin and bored at both ends while so clamped. Eyebars shall be packed and match-marked for shipment and erection. All identifying marks shall be stamped with steel stencils on the edge of one head of each member after fabrication is completed so as to be visible when the bars are nested in place on the structure.

Steel die stamps shall be low stress type. No welding is allowed on eyebars or to secure adjacent eyebars. The eyebars shall be straight and free from twists and the pin holes shall be accurately located on the center line of the bar. The inclination of any bar to the plane of the truss shall not exceed 1/16 inch to a foot.

The edges of eyebars that lie between the transverse center line of their pin holes shall be cut simultaneously with two mechanically operated torches abreast of each other, guided by a substantial template, in such a manner as to prevent distortion of the plates. 11.4.11 Annealing and Stress Relieving Structural members which are indicated in the contract to be annealed or normalized shall have finished machin- 574 HIGHWAY BRIDGES ing, boring, and straightening done subsequent to heat treatment. Normalizing and annealing (full annealing) shall be as specified in ASTM E 44. The temperatures shall be maintained uniformly throughout the furnace during the heating and cooling so that the temperature at no two points on the member will differ by more than 100°F at any one time.

Members of AASHTO M 270 (ASTM A 709) Grades 100/100W or Grade 70W steels shall not be annealed or normalized and shall be stress relieved only with the approval of the Engineer. A record of each furnace charge shall identify the pieces in the charge and show the temperatures and schedule actually used.

Proper instruments, including recording pyrometers, shall be provided for determining at any time the temperatures of members in the furnace. The records of the treatment operation shall be available to and meet the approval of the Engineer. The holding temperature for stress relieving Grades HPS70W and 100/100Wshall not exceed l,lOO°F and for Grade 70W shall not exceed 1,050°F. Members, such as bridge shoes, pedestals, or other parts that are built up by welding sections of plate together shall be stress relieved in accordance with the procedure of Section 4.4 of the current ANSVAASHTO/AWS D1.5 Bridge Welding Code, when required by the plans, specifications, or special provisions governing the contract.

11.4.12 Curved Girders 11.4.12.1 General Flanges of curved, welded girders may be cut to the radii shown on the plans or curved by applying heat as specified in the succeeding articles providing the radii is not less than allowed by Article 10.15.2 of Division I. 11.4.12.2 Heat Curving Rolled Beams and Welded Girders 11.4.12.2.1 Materials Except for Grade HPS70W steel, steels that are manufactured to a specified minimum yield point greater than 50,000 psi shall not be heat curved. 11.4.12.2.2 Type of Heating Beams and girders may be curved by either continuous or V-type heating as approved by the Engineer. For the continuous method, a strip or intermittent strips along the edge of the top and bottom flange shall be heated approximately simultaneously depending on flange widths and thicknesses; the strip shall be of sufficient width and tem- 11.4.11 perature to obtain the required curvature. For the V-type heating, the top and bottom flanges shall be heated in truncated triangular or wedge-shaped areas having their base along the flange edge and spaced at regular intervals along each flange; the spacing and temperature shall be as required to obtain the required curvature, and heating shall progress along the top and bottom flange at approximately the same rate. For the V-type heating, the apex of the truncated triangular area applied to the inside flange surface shall terminate just before the juncture of the web and the flange is reached. To avoid unnecessary web distortion, special care shall be taken when heating the inside flange surfaces (the surfaces that intersect the web) so that heat is not applied directly to the web. When the radius of curvature is 1,000 feet or more, the apex of the truncated triangular heating pattern applied to the outside flange surface shall extend to the juncture of the flange and web. When the radius of curvature is less than 1,000 feet, the apex of the truncated triangular heating pattern applied to the outside flange surface shall extend past the web for a distance equal to one-eighth of the flange width or 3 inches, whichever is less. The truncated triangular pattern shall have an included angle of approximately 15 to 30°, but the base of the triangle shall not exceed 10 inches. Variations in the patterns prescribed above may be made with the approval of the Engineer. For both types of heating, the flange edges to be heated are those that will be on the inside of the horizontal curve after cooling. Heating both inside and outside flange surfaces is only mandatory when the flange thickness is 1 l/4 inches or greater, in which case, the two surfaces shall be heated concurrently. The maximum temperature shall be prescribed as follows. 11.4.12.2.3 Temperature The heat-curving operation shall be conducted in such a manner that the temperature of the steel does not exceed 1,200°Ffor Grades 36,50 and 50W, l,lOO°F for Grades HPS70W and 100/100W;and 1,050°F for Grade 70W as measured by temperature indicating crayons or other suitable means. The girder shall not be artificially cooled until after naturally cooling to 600°F. The method of artificial cooling is subject to the approval of the Engineer. 11.4.12.2.4 Position for Heating The girder may be heat-curved with the web in either a vertical or a horizontal position. When curved in the vertical position, the girder must be braced or supported in such a manner that the tendency of the girder to deflect laterally during the heat-curving process will not cause the girder to overturn. 11.4.12.2.4 575 DIVISION 11--CONSTRUCTION When curved in the horizontal position, the girder must be supported near its ends and at intermediate points, if required, to obtain a uniform curvature; the bending stress in the flanges due to the dead weight of the girder and externally applied loads must not exceed the usual allowable design stress. When the girder is positioned horizontally for heating, intermediate safety catch blocks must be maintained at the mid-length of the girder within 2 inches of the flanges at all times during the heating process to guard against a sudden sag due to plastic flange buckling. 11.4.12.2.5 Sequence of Operations The girder shall be heat-curved in the fabrication shop before it is painted. The heat curving operation may be conducted either before or after all the required welding of transverse intermediate stiffeners is completed. However, unless provisions are made for girder shrinkage, connection plates and bearing stiffeners shall be located and attached after heat curving. If longitudinal stiffeners are required, they shall be heat-curved or oxygen-cut separately and then welded to the curved girder.

When cover plates are to be attached to rolled beams, they may be attached before heat curving if the total thickness of one flange and cover plate is less than 2% inches and the radius of curvature is greater than 1,000 feet. For other rolled beams with cover plates, the beams must be heat-curved before the cover plates are attached; cover plates must be either heat curved or oxygen-cut separately and then welded to the curved beam. 11.4.12.2.6 Camber Girders shall be cambered before heat curving. Camber for rolled beams may be obtained by heatcambering methods approved by the Engineer. For plate girders, the web shall be cut to the prescribed camber with suitable allowance for shrinkage due to cutting, welding, and heat curving. However, subject to the approval of the Engineer, moderate deviations from specified camber may be corrected by a carefully supervised application of heat. 11.4.13 Orthotropic-Deck Superstructures 11.4.13.1 General Dimensional tolerance limits for orthotropic-deck bridge members shall be applied to each completed but unloaded member and shall be as specified in Article 3.5 of the current ANSI/AASHTO/AWS D1.5 Bridge Welding Code except as follows. The deviation from detailed flatness, straightness, or curvature at any point shall be the perpendicular distance from that point to a template edge which has the detailed straightness or curvature and which is in contact with the element at two other points. The term element as used herein refers to individual panels, stiffeners, flanges, or other pieces. The template edge may have any length not exceeding the greatest dimension of the element being examined and, for any panel, not exceeding 1.5 times the least dimension of the panel; it may be placed anywhere within the boundaries of the element.

The deviation shall be measured between adjacent points of contact of the template edge with the element; the distance between these adjacent points of contact shall be used in the formulas to establish the tolerance limits for the segment being measured whenever this distance is less than the applicable dimension of the element specified for the formula. 11.4.13.2 Flatness of Panels (a) The term "panel" as used in this article means a clear area of steel plate surface bounded by stiffeners, webs, flanges, or plate edges and not further subdivided by any such elements. The provisions of this article apply to all panels in the bridge; for plates stiffened on one side only such as orthotropic-deck plates or flanges of box girders, this includes the total clear width on the side without stiffeners as well as the panels between stiffeners on the side with stiffeners. (b) The maximum deviation from detailed flatness or curvature of a panel shall not exceed the greater of: 3/16 inch or 11.4.12.2.7 Measurement of Curvature and Camber Horizontal curvature and vertical camber shall be measured for final acceptance after all welding and heating operations are completed and the flanges have cooled to a uniform temperature. Horizontal curvature shall be checked with the girder in the vertical position. - 1 4 y f i inch where, D = the least dimension in inches along the bound- T = the minimum thickness in inches of the plate ary of the panel comprising the panel.

576 HIGHWAY BRIDGES 11.4.13.3 Straightness of Longitudinal Stiffeners Subject to Calculated Compressive Stress, Including Orthotropic-Deck Ribs The maximum deviation from detailed straightness or curvature in any direction perpendicular to its length of a longitudinal web stiffener or other stiffener subject to calculated compressive stress shall not exceed: where L = the length of the stiffener or rib between cross members, webs, or flanges, in inches. excessively stressed, deformed, or otherwise damaged. Bolts, nuts and washers (where required) from each rotational-capacity lot shall be shipped in the same container. If there is only one production lot number for each size of nut and washer, the nuts and washers may be shipped in separate containers. Pins, small parts and packages of bolts, washers, and nuts shall be shipped in boxes, crates, kegs, or barrels, but the gross weight of any package shall not exceed 300 pounds. A list and description of the contained materials shall be plainly marked on the outside of each shipping container. 11.5 ASSEMBLY 11.5.1 Bolting 11.4.13.4 Straightness of Transverse Web Stiffeners and Other Stiffeners not Subject to Calculated Compressive Stress The maximum deviation from detailed straightness or curvature in any direction perpendicular to its length of a transverse web stiffener or other stiffener not subject to calculated compressive stress shall not exceed: L where L = the length of the stiffener between cross members, webs, or flanges, in inches. 11.4.14 Full-Sized Tests When full-sized tests of fabricated structural members or eyebars are required by the contract, the Contractor shall provide suitable facilities, material, supervision, and labor necessary for making and recording the required tests. The members tested in accordance with the contract shall be paid for in accordance with Article 11.7.2. 11.4.15 Marking and Shipping Each member shall be painted or marked with an erection mark for identification and an erection diagram showing these marks shall be furnished to the Engineer. The Contractor shall furnish to the Engineer as many copies of material orders, shipping statements, and erection diagrams as the Engineer may direct.

The weights of the individual members shall be shown on the statements. Members weighing more than 3 tons shall have the weights marked thereon.

Structural members shall be loaded on trucks or cars in such a manner that they may be transported and unloaded at their destination without being Surfaces of metal in contact shall be cleaned before assembling. The parts of a member shall be assembled, well pinned, and firmly drawn together before drilling, reaming, or bolting is commenced. Assembled pieces shall be taken apart, if necessary, for the removal of burrs and shavings produced by the operation. The member shall be free from twists, bends, and other deformation. The drifting done during assembling shall be only such as to bring the parts into position and not sufficient to enlarge the holes or distort the metal. Surfaces and edges to be welded shall be smooth, uniform, clean and free of defects which would adversely affect the quality of the weld. Edge preparation shall be done in accordance with the current ANSVAASHTO/AWS D1.5 Bridge Welding Code. 11.5.3 Preassembly of Field Connections 11.5.3.1 General Field connections of main members of trusses, arches, continuous beams, plate girders, bents, towers and rigid frames shall be preassembled prior to erection as necessary to verify the geometry of the completed structure or unit and to verify or prepare field splices.

Attaining accurate geometry is the responsibility of the Contractor who shall propose an appropriate method of preassembly for approval by the Engineer. The method and details of preassembly shall be consistent with the erection procedure shown on the erection plans and camber diagrams prepared by the Contractor and approved by the Engineer. As a minimum, the preassembly procedure shall consist of assembling three contiguous panels accurately 11.5.3.1 DIVISION11--CONSTRUCTION adjusted for line and camber. Successive assemblies shall consist of at least one section or panel of the previous assembly (repositioned if necessary and adequately pinned to assure accurate alignment) plus two or more sections or panels added at the advancing end. In the case of structures longer than 150 feet, each assembly shall be not less than 150 feet long regardless of the length of individual continuous panels or sections. At the option of the fabricator, sequence of assembly may start from any location in the structure and proceed in one or both directions so long as the preceding requirements are satisfied. 11.5.3.2 Bolted Connections For bolted connections holes shall be prepared as outlined in Article 11.4.8. Where applicable, major components shall be assembled with millkd ends ofcbmpression members in full bearing and then shall have their subsized holes reamed to the specified size while the connections are assembled. 11.5.3.3 Check Assembly-Numerically Controlled Drilling When the contractor elects to use numerically controlled drilling, a check assembly shall be required for each major structural type of each project, unless otherwise designated on the plans or in the special provisions, and shall consist of at least three contiguous shop sections or, in a truss, all members in at least three contiguous panels but not less than the number of panels associated with three contiguous chord lengths (i.e., length between field splices). Check assemblies should be based on the proposed order of erection, joints in bearings, special complex points, and similar considerations. Special points could be the portals of skewed trusses, for example. The check assemblies shall preferably be the first sections of each major structural type to be fabricated. Shop assemblies other than the check assemblies will not be required. If the check assembly fails in some specific manner to demonstrate that the required accuracy is being obtained, further check assemblies may be required by the Engineer for which there shall be no additional cost to the Department. Each assembly, including camber, alignment, accuracy of holes, and fit of milled joints, shall be approved by the Engineer before reaming is commenced or before an NIC drilled check assembly is dismantled. 11.5.3.4 Field Welded Connections 577 For field welded connections the fit of members including the proper space between abutting flanges shall be prepared or verified with the segment preassembled in accordance with Article 11.5.3.1. 11.5.4 Match Marking Connecting parts preassembled in the shop to assure proper fit in the field shall be match-marked, and a diagram showing such marks shall be furnished to the Engineer. 11.5.5 Connections Using Unfinished, lhrned or Ribbed Bolts 11.5.5.1 General When unfinished bolts are specified, the bolts shall be unfinished, turned, or ribbed bolts conforming to the requirements for Grade A Bolts of Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength, ASTM A 307. Bolts shall have single self-locking nuts or double nuts unless otherwise shown on the plans or in the special provisions. Beveled washers shall be used where bearing faces have a slope of more than 1:20 with respect to a plane normal to the bolt axis. The specifications of this article do not pertain to the use of high-strength bolts. Bolted connections fabricated with high-strength bolts shall conform to Article 11.5.6. 11.5.5.2 Turned Bolts The surface of the body of turned bolts shall meet the ANSI roughness rating value of 125.

Heads and nuts shall be hexagonal with standard dimensions for bolts of the nominal size specified or the next larger nominal size.

Diameter of threads shall be equal to the body of the bolt or the nominal diameter of the bolt specified. Holes for turned bolts shall be carefully reamed with bolts furnished to provide for a light driving fit. Threads shall be entirely outside of the holes. A washer shall be provided under the nut. 11.5.5.3 Ribbed Bolts The body of ribbed bolts shall be of an approved form with continuous longitudinal ribs.

The diameter of the body measured on a circle through the points of the ribs shall be 5/64 inch greater than the nominal diameter specified for the bolts. Ribbed bolts shall be furnished with round heads conforming to ANSI B 18.5 unless otherwise specified. Nuts shall be hexagonal, either recessed or with a washer of suitable thickness. Ribbed bolts shall make a driving fit 578 HIGHWAY BRIDGES with the holes. The hardness of the ribs shall be such that the ribs do not mash down enough to permit the bolts to turn in the holes during tightening. If for any reason the bolt twists before drawing tight, the hole shall be carefully reamed and an oversized bolt used as a replacement. 11.5.6 Connections Using High-Strength Bolts 11.5.6.1 General This article covers the assembly of structural joints using AASHTO M 164 (ASTM A 325) or AASHTO M 253 (ASTM A 490) high-strength bolts, or equivalent fasteners, installed so as to develop the minimum required bolt tension specified in Table 11.5A. The bolts are used in holes conforming to the requirements of Article 11.4.8. 11.5.6.2 Bolted Parts All material within the grip of the bolt shall be steel, there shall be no compressible material such as gaskets or insulation within the grip.

Bolted steel parts shall fit solidly together after the bolts are snugged, and may be coated or uncoated. The slope of the surfaces of parts in contact with the bolt head or nut shall not exceed 1:20 with respect to a plane normal to the bolt axis. 11.5.6.3 Surface Conditions At the t i e of assembly, all joint surfaces, including surfaces adjacent to the bolt head and nut, shall be free of scale, except tight mill scale, and shall be free of dirt or other foreign material. Burrs that would prevent solid seating of the connected parts in the snug condition shall be removed. TABLE 11.5A Required Fastener Tension Minimum Bolt Tension in Pounds* Bolt Size Inches AASHTO M 164 ASTM A 325 AASHTO M 253 ASTM A 490 * The minimum bolt tension shall be taken as 70% of specified minimum tensile strength of bolts (as specified in ASTM Specifications for tests of full-size A 325 and A 490 bolts with UNC threads loaded in axial tension) rounded to the nearest kip.

11.5.5.3 Paint is permitted on the faying surface including slip critical joints when designed in accordance with Articles 10.32.3, or 10.56.1.3, Division I. The faying surfaces of slip-critical comections shall meet the requirements of the following paragraphs, as applicable: (1) In noncoated joints, paint, including any inadvertent overspray, shall be excluded from areas closer than one-bolt diameter, but not less than 1 inch, from the edge of any hole and all areas within the bolt pattern. (2) Joints specified to have painted faying surfaces shall be blast cleaned and coated with a paint which has been qualified in accordance with requirements of Articles 10.32.3.2.3 or 10.57.3.3, Division I. (3) Coated joints shall not be assembled before the coating has cured for the minimum time used in the quallfying test. (4) Faying surfaces specified to be galvanized shall be hot-dip galvanized in accordance with AASHTO M 111 (ASTM A 123), and shall subsequently be roughened by means of hand wire brushing. Power wire brushing is not permitted. 11.5.6.4 Installation 115.6.4.1 General Fastener components shall be assigned lot numbers (including rotational-capacity lot numbers) prior to shipping, and components shall be assembled when installed. Such components shall be protected from dirt and moisture at the job site.

Remove from protective storage only the number of anticipated components to be installed during a work shift. Components not used shall be returned to protected storage at the end of the shift. Components shall not be cleaned of lubricant that is required to be present in asdelivered condition. Assemblies for slip-critical connections which accumulate rust or dirt resulting from job site conditions shall be cleaned, relubricated and tested for rotational-capacity prior to installation. All galvanized nuts shall be lubricated with a lubricant containing a visible dye. Plain bolts must be "oily" to touch when delivered and installed. Lubricant on exposed surfaces shall be removed prior to painting. A bolt tension measuring device (a Skidmore-Wilhelm Calibrator or other acceptable bolt tension indicating device) shall be at all job sites where high-strength bolts are being installed and tensioned. The tension measuring device shall be used to perform the rotational-capacity test and to confirm (1) the suitability to satisfy the requirements of Table 11.5A of the complete fastener assembly, including lubrication if required to be used in the work, (2) calibration of the wrenches, if applicable, and (3) the understanding and proper use by the bolting crew of the 11.5.6.4.1 DIVISION 11-CONSTRUCTION installation method. To perform the calibrated wrench verification test for short grip bolts, direct tension indicators (DTI) with solid plates may be used in lieu of a tension measuring device. The DTI lot shall be first verified with a longer grip bolt in the Skidmore-WilhelmCalibrator or an acceptable equivalent device. The frequency of confirmation testing, the number of tests to be performed, and the test procedure shall be as specified in Articles 11 56.4.4 through 11 56.4.7, as applicable. The accuracy of the tension measuring device shall be confirmed by an approved testing agency at least annually. Bolts and nuts together with washers of size and quality specified, located as required below, shall be installed in properly aligned holes and tensioned and inspected by any of the installation methods described in Articles 11 56.4.4 through 11.5.6.4.7 to at least the minimum tension specified in Table 11.5A.Tensioning may be done by turning the bolt while the nut is prevented from rotating when it is impractical to turn the nut. Impact wrenches, if used, shall be of adequate capacity and sufficiently supplied with air to tension each bolt in approximately 10 seconds.

A 490) fasteners and galvaAASHTO M 253 nized AASHTO M 164 (ASTM A 325) fasteners shall not be reused. Other AASHTO M 164 (ASTM A 325) bolts may be reused if approved by the Engineer. Touching up or retensioning previously tensioned bolts which may have been loosened by the tensioning of adjacent bolts shall not be considered as reuse provided the tensioning continues from the initial position and does not require greater rotation, including the tolerance, than that required by Table 11.5B.

Bolts shall be installed in all holes of the connection and the connection brought to a snug condition. Snug is defined as having all plies of the connection in firm contact. Snugging shall progress systematically from the most rigid part of the connection to the free edges. The snugging sequence shall be repeated until the full connection is in a snug condition. IASTM 11.5.6.4.2 Rotational-Capacity Tests Rotational-capacity testing is required for all fastener assemblies. Galvanized assemblies shall be tested galvanized. Washers are required as part of the test even though they may not be required as part of the installation procedure. The following shall apply: (a) Except as modified herein, the rotational-capacity test shall be performed in accordance with the requirements of AASHTO M 164 (ASTM A 325). (b) Each combination of bolt production lot, nut lot and washer lot shall be tested as an assembly. Where washers are not required by the installation procedures, they need not be included in the lot identification. 579 TABLE 115B Nut Rotation from the Snug-right C~ndition~Geometry of Outer Faces of Bolted Parts One Face Both Faces Normal to Sloped Not Bolt Axis More Than and Other Bolt Length Face Sloped 1:20 From Normal To Measured Not More Bolt Axis, Than 1:20, From Bevel Underside of Both Faces Bevel Head to End of Normal to Washer Not Washers Not Used Bolt Bolt Axis Used Up to and including 4 diameters Over 4 d i e t e r s but not exceeding 8 diameters Over 8 diameters but not exceeding 12 diametersc 113 turn 112 turn 2 3 turn 112 turn 2/3 turn 516 turn 2/3 turn 516 turn laun Nut r o t a h is relative to bolt, regardless of the element (nut or bolt) being turned. For bdts installed by 1R bun and less, the tolerance should be plus or minus 30 degrees; hr bolts installedby 213 turn and more, the tderance should be plus or minus 45 degrees.

Applicable oaly to cunnectioas in whicb dl material within grip of the bolt is steel cNoresearcbworkhaskenperfomedby theResearchCouncilRiveted and Bolted !Structural Joints to establish the kun4-11utprocedure when bolt lengths umd 12 diameters. Therefwe, the required rotation must be determined by actual tests in a suitable tension device simulating the actual conditim (c) A rotational-capacity lot number shall have been assigned to each combination of lots tested. (d) The minimum frequency of testing shall be two assemblies per rotational-capacity lot. (e) For bolts that are long enough to fit in a SkidmoreWilhelm Calibrator, the bolt, nut and washer assembly shall be assembled in a Skidmore-Wilhelm Calibrator or an acceptable equivalent device. (f) Bolts that are too short to be tested in a SkidmoreWilhelm Calibrator may be tested in a steel joint.

The tension requirement, in (g) below, need not apply. The maximum torque requirement, torque < 0.25 PD, shall be computed using a value of P equal to the turn test tension taken as 1.15 times the bolt tension in Table 11.5A.

(g) The tension reached at the below rotation (i.e., turntest tension) shall be equal to or greater than 1 .I5 times the required fastener tension (i.e., installation tension) shown in Table 11.5A. 580 HIGHWAY BRIDGES (h) The minimum rotation from an initial tension of 10% of the minimum required tension (snug condition) shall be two times the required number of turns indicated in Table 11.5B without stripping or failure. (i) After the required installation tension listed above has been exceeded, one reading of tension and torque shall be taken and recorded. The torque value shall conform to the following: Torque 5 0.25 PD Where: Torque = measured torque (foot-pounds) P = measured bolt tension (pounds) D = bolt diameter (feet). 11.5.6.4.3 Requirement for Washers Where the outer face of the bolted parts has a slope greater than 1:20 with respect to a plane normal to the bolt axis, a hardened beveled washer shall be used to compensate for the lack of parallelism. Hardened beveled washers for American Standard Beams and Channels shall be required and shall be square or rectangular, shall confor& to the requirements of AASHTO M 293 (ASTM F 436), and shall taper in thickness. Where necessary, washers may be clipped on one side to a point not closer than YEof the bolt diameter from the center of the washer. Hardened washers are not required for connections using AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490) bolts except as follows: Hardened washers shall be used under the turned element when tensioning is to be performed by calibrated wrench method. Irrespective of the tensioning method, hardened washers shall be used under both the head and the nut when AASHTO M 253 (ASTM A 490) bolts are to be installed in material having a specified yield point less than 40 ksi. However, when DTIs are used they may replace a hardened washer provided a standard hole is used.

Where AASHTO M 164 (ASTM A 325) bolts of any diameter or AASHTO M 253 (ASTM A 490) bolts equal to or less than 1 inch in diameter are to be installed in oversize or short-slotted holes in an outer ply, a hardened washer conforming to AASHTO M 293 (ASTM F 436) shall be used. When AASHTO M 253 (ASTM A 490) bolts over 1 inch in diameter are to be installed in an oversized or short-slotted hole in an outer ply, hardened washers conforming to AASHTO M 293 (ASTM F 436) except with 5/16 inch minimum thickness shall be 11 56.4.2 used under both the head and the nut in lieu of standard thickness hardened washers. Multiple hardened washers with combined thickness equal to or greater than 5/16 inch do not satisfy this requirement. Where AASHTO M 164 (ASTM A 325) bolts of any diameter or AASHTO M 253 (ASTM A 490) bolts equal to or less than 1 inch in diameter are to be installed in a long slotted hole in an outer ply, a plate washer or continuous bar of at least 7 1 6 inch thickness with standard holes shall be provided. These washers or bars shall have a size sufficient to completelycover the slot after installation and shall be i f structural grade material, but need not be hardened except as follows. When AASHTO M 253 (ASTM A 490) bolts over 1 inch in diameter are to be used in long slotted holes in external plies, a single hardened washer conforming to AASHTO M 293 (ASTM F 436) but with 5/16 inch minimum thickness shall be used in lieu of washers or bars of structural grade material. Multiple hardened washers with combined thickness equal to or greater than 5/16 inch do not satisfy this requirement. Alternate design fasteners meeting the requirements of Article 11.3.2.6 with a geometry which provides a bearing circle on the head or nut with a diameter equal to or greater then the diameter of hardened washers meeting the requirements of AASHTO M 293 (ASTM F 436) satisfy the requirementsfor washers specified herein and may be used without washers. 11.5.6.4.4 Turn-of-Nut Installation Method When the turn-of-nut installation method is used, hardened washers are not required except as may be specified in Article 11 56.4.3. Verification testing using a representative sample of not less than three fastener assemblies of each diameter, length and grade to be used in the work shall be performed at the start of work in a device capable of indicating bolt tension. This verification test shall demonstrate that the method used to develop a snug condition and control the turns from snug by the bolting crew develops a tension not less than 5% greater than the tension required by Table 11.5A. Periodic retesting shall be performed when ordered by the Engineer. After snugging, the applicable amount of rotation specified in Table 11.5B shall be achieved. During the tensioning operation there shall be no rotation of the part not turned by the wrench. Tensioningshall progress systematically from the most rigid part of the joint to its free edges. 11.5.6.4.5 Calibrated Wrench Installatioa Method The calibrated wrench method may be used only when wrenches are calibrated on a daily basis and when a hardened washer is used under the turned element. Standard 11.5.6.4.5 DIVISION II-C(assumed to relate torque to tension shall not be acceptable. When calibrated wrenches are used for installation, they shall be set to deliver a torque which has been calibrated to develop a tension not less than 5% in excess of the minimum tension specified in Table 11.5A.The installation procedures shall be calibrated by verification testing at least once each working day for each fastener assembly lot that is being installed that day in the work. This verification testing shall be accomplished in a tension measuring device capable of indicatingactual bolt tension by testing three typical fastener assemblies from each lot. Bolts, nuts and washers under the turned element shall be sampled from production lots. Wrenches shall be recalibrated when significant difference is noted in the surface condition of the bolts, threads, nuts or washers. It shall be verified during actual installation in the assembled steel work that the wrench adjustment selected by the calibration does not produce a nut or bolt head rotation from a snug condition greater than that permitted in Table 11.5B. If manual torque wrenches are used, nuts shall be torqued in the tensioning direction when torque is measured. When calibrated wrenches are used to install and tension bolts in a connection, bolts shall be installed with hardened washers under the turned element. Following snugging, the connection shall be tensioned using the calibrated wrench. Tensioning shall progress systematically from the most rigid part of the joint to its free edges. The wrench shall be returned to "touch up" previously tensioned bolts which may have been relaxed as a result of the subsequent tensioning of adjacent bolts until all bolts are tensioned to the prescribed amount. bolts shall be installed in all holes of the connection and initially snugged sufficiently to bring all plies of the joint intofirm contact but without yielding or fracturing the control or indicator element of the fasteners.Allfasteners shall then be further tensioned, progressing systematically from the most rigid part of the connection to the free edges in a manner that will minimize relaxation of previously tensioned bolts. In some cases, proper tensioning of the bolts may require more than a single cycle of systematic partial tensioning prior to final yielding or fracturing of the control or indicator element of individual fasteners. If yielding or twist-off occurs prior to the final tensioning cycle, the fastener assembly shall be replaced with a new one.

11 5 6 . 4 . 7 Direct Tension Indicator Installation Method When Direct Tension Indicators (DTIs) meeting the requirements of Article 11.3.2.6 are to be used with highstrength bolts to indicate bolt tension, they shall be subjected to the verification testing described below and installed in accordance with the method specified below. Unless otherwise approved by the engineer-of-record, the DTIs shall be installed under the head of the bolt and the nut turned to tension the bolt. The Manufacturer's recommendations shall be followed for the proper orientation of the DTI and additional washers, if any, required for the correct use of the DTI. Installation of a DTI under the turned element may be permitted if a washer separates the turned element from the DTI. 11 5 6 . 4 . 7 ~ Verification 11.5.6.4.6 Alternative Design Bolts Installation Method When fasteners which incorporate a design feature intended to indirectly indicate that the applied torque develops the required tension or to automatically develop the tension required by Table 11.5A and which have been qualified under Article 11.3.2.5 are to be installed, verification testing using a representative sample of not less than three fastener assemblies of each diameter, length and grade to be used in the work shall be performed at the job site in a device capable of indicating bolt tension. The test assembly shall include flat-hardened washers, if required in the actual connection, arranged as in the actual connections to be tensioned. The verification test shall demonstrate that each bolt develops a tension not less than 5% greater than the tension required by Table 11.5A. Manufacturer's installation procedure shall be followed for installation of bolts in the calibration device and in all connections. Periodic retesting shall be performed when ordered by the Engineer. When alternate design fasteners which are intended to control or indicate bolt tension of the fasteners are used, Verification testing shall be performed in a calibrated bolt tension measuring device. A special flat insert shall be used in place of the normal bolt head holding insert.

Three verification tests are required for each combination of fastener assembly rotational-capacity lot, DTI lot, and DTI position relative to the turned element (bolt head or nut) to be used on the project. The fastener assembly shall be installed in the tension measuring device with the DTI located in the same position as in the work. The element not turned (bolt or nut) shall be restrained from rotation.

The purpose of verification testing is to ensure that the fastener will be at or above the desired installation tension when the requisite number of spaces between the protrusions have a gap of 0.005 inches or less and that the bolt will not have excessive

their full cross-sectional area, or otherwise connected to provide full continuity in accordance with the design. 588 HIGHWAY BRTDGES 12.4 PROVISION FOR CAMBER Unless otherwise provided on the plans, provision for camber shall be made as follows: Steel units so rigid that they will not readily follow the camber required shall be cambered in the shop. For grid flooring types other than those employing a field placed full depth concrete filling attached to the deck with welded shear connectors, the stringers shall be canted or provided with shop-welded beveled bearing bars to provide a bearing surface parallel to the crown of the roadway. If beveled bars are used, they shall be continuous and fillet welded along the center line of the stringer flange; in which case, the design span length shall be governed by the width of the bearing- bar instead of the width of the stringer flange. Longitudinal stringers, except as provided in the following paragraph, shall be mill cambered or provided with b e a n g strips so that the completed floor after dead load deflection will conform to the longitudinal camber shown on the plans.
Vertical adjustment of full-depth-filled grid floors, which are to be connected to supporting members with shear connectors, may be accomplished by use of adjusting bolts operating through nuts welded to the grid and bearing on the top flange of framing members.
Alternatively, shims may be used, and shims must be used if construction vehicles are to be allowed on the floor prior to final attachment. 12.5 FIELD ASSEMBLY Areas of considerable size shall be placed and, if necessary, adjusted to proper fit before thefloor is connected to its supports. Care shall be taken during lifting and placing to avoid overstressing the grid units. The main elements shall be made continuous as specified in Article 12.3, and sections shall be connected together along their edges by welding or bolting in accordance with the plans or the approved working drawings. 12.6 CONNECTION TO SUPPORTS Except when other connection methods are specified or approved, the floor shall be connected to its steel supports by welding every fourth main element to the supporting member; however, welds shall be spaced no greater than 15 inches on centers. Before any welding is done, the floor shall either be temporarily loaded or it shall be clamped down to make a tight joint with full bearing. To minimize the stresses induced through clamping down, any differential elevation of % inch or more over a 4-foot support- 12.4 ing member shall be shimmed before welding the shim, the grid, and the supporting member. The location, length, and size of the welds shall be subject to the approval of the Engineer. Around the perimeter of continuous units of grid flooring, the ends of all the main steel members of the flooring shall be securely fastened together by means of steel plates or angles welded to the ends of the main members, or by thoroughly encasing the ends with concrete. When specified or approved, methods other than welding may be used for attaching steel grid floors (both open and concrete filled types) to framing members. In such cases, welded headed shear connectors can be employed for concrete filled grids and open steel grids can be connected to framing members by bolting. 12.7 WELDING All shop and field welding shall be done in accordance welding Code wi thmsuaasHTO, AWS 12.8 REPAIRING DAMAGED GALVANIZED COATINGS Galvanized surfaces that are abraded or damaged at any time after the application of the zinc coating shall be repaired by thoroughly wire brushing the damaged areas and removing all loose and cracked coating, after which the cleaned areas shall be painted with two applications of unthinned commercial quality zinc-rich primer (organic vehicle type). Spray cans shall not be used. 12.9 PLACEMENT OF CONCRETE FILLER 12.9.1 Forms Concrete filled types of flooring with bottom flanges not in contact with each other shall be provided with bottom forms of metal or wood to retain the concrete filler without excessive leakage. Forms shall be removed after the concrete has been cured except that metal forms conforming to the following paragraph may be left in place. If metal form strips are used they shall fit tightly on the bottom flanges or protrusions of the grid members and be placed in noncontinuous lengths so as to extend not more than 1 inch onto the edge of each support, but in all cases the forms shall be such as will result in adequate bearing of slab on the support. If metal forms are to be left in place, they shall either be galvanized or protective treated by the same method that is required for the grid flooring. 12.9.2 DIVISION II-CONSTRUCTION 12.9.2 Placement i When the plans indicate that the concrete filling does not extend to the bottom of the steel grid, the concrete, except concrete for cells in which shear connectors are to be installed, may be placed with the grid in an inverted position prior to installation, or the portion of the grid to remain unfilled may be blocked out by the use of a temporary inert filling material, such as sand or polystyrene board filler which is later removed, or by the use of metal lath form strips or other approved methods. The method used shall permit full embedment of the tertiary bars and the shear connector studs, if used.
When the plans or specifications indicate that filled or partially filled grids or reinforced concrete slabs incorporating steel grids are to act compositely with their supporting members, all shear connecting studs shall be fully encased in concrete and the entire area between the top flange of the supporting member and the bottom of the grid filling shall be filled with concrete.
589 The concrete for filled grid floors shall be mixed, placed, and cured in accordance with the requirements of Section 8. The concrete shall be thoroughly compacted by vibrating the steel grid floor. The vibrating device and the manner of operating it shall be subject to the approval of the Engineer. 12.10 MEASUREMENT AND PAYMENT Steel grid flooring will be measured by the square foot. The number of square feet will be based on the dimensions of the flooring in place and approved by the Engineer in the completed work. Steel grid flooring will be paid for at the contract price per square foot. Such payment for steel grid floor, open or concrete filled types, shall be considered to be full compensation for the cost of furnishing of all materials, equipment, tools, and labor necessary for the satisfactory completion of the work. Section 13 PAINTING 13.1 GENERAL 13.1.1 Description This work shall consist of the painting of surfaces shown on the plans or otherwise specified to be painted. The work includes, but is not limited to, the preparation of surfaces to be painted, application and curing of the paint, protection of the work, protection of existing facilities, vehicles and the public from damage due to this work, and the furnishing of all labor, equipment, and materials needed to perform the work.
13.1.2 Protection of Public and Property The Contractor shall comply with all applicable environmental protection and occupational safety and health standards, rules, regulations, and orders. Failure to comply with these standards, rules, regulations, and orders will be sufficient cause for suspension or disqualification. All reasonable precautions shall be taken to contain waste materials (used blasting material and old paint) classified as hazardous. Disposal of hazardous waste material shall be performed in accordance with all applicable federal, state, and local laws. The Contractor shall provide protective devices such as tarps, screens or covers as necessary to prevent damage to the work and to other property or persons from all cleaning and painting operations. Paint or paint stains that result in an unsightly appearance on surfaces not designated to be painted shall be removed or obliterated by the Contractor at own expense. 13.1.3 Protection of the Work All painted surfaces that are marred or damaged as a result of operations of the Contractor shall be repaired by the Contractor, at own expense, with materials and to a condition equal to that of the coating specified herein. If tr c causes an objectionable amount of dust, the Contractor, when directed by the Engineer, shall sprinkle the adjacent roadbed and shoulders with water or dust palliative for a sufficient distance on each side of the location where painting is being done. Upon completion of all painting operations and of any other work that would cause dust, grease, or other foreign materials to be deposited on the painted surfaces, the painted surfaces shall be thoroughly cleaned. At the time of opening structures to public traffic, the painting shall be completed, and the surfaces shall be undamaged and clean. 13.1.4 Color If not otherwiseshown or specified, the color of the top or finish coat of paint shall be as directed by the Engineer. 13.2 PAINTING METAL STRUCTURES 13.2.1 Coating Systems and Paints The coating system and paints to be applied shall consist of the system in Table 13.2.1 which is specified for use or modified by the special provisions. 13.2.2 Weather Conditions Paint shall be applied only on thoroughly dry surfaces. Painting will not be permitted when the atmospheric temperature, paint, or the surface to be painted is at or below 40°F or above 100°F,or when metal surfaces are less than 5°F above the dew point, or when the humidity exceeds 85% at the site of the work, or when freshly painted surfaces may become damaged by rain, fog, or dust, or when it can be anticipated that the atmospheric temperature will drop below 40°F during the drying period, except as provided herein for painting in enclosures. Metal surfaces which are hot enough to cause the paint to blister, to produce a porous paint film, or to cause the vehicle to separate from the pigment shall not be painted. Subject to approval of the Engineer, the Contractor may provide a suitable enclosure to pennit painting during inclement weather. Provisions shall be made to artifi- 592 HIGHWAY BRIDGES 13.2.2 TABLE 13.2.1 Primer Intermediate Coat Top Coat Total System High Pollution and Coastal Mild Climate Mild Climate or Maintenance Repainting Inorganic Zinc-3 mils. Epoxy 2 mils or Vinyl Wash Primer 0.3-0.5 mils. Epoxy, Vinyl, or Urethane-2 mils. 5.3-7 mils. Organic Zinc-3 mils. Epoxy 2 mils.
or Vinyl Wash Primer 0.3-0.5 mils. Epoxy, Vinyl, or Urethane-2 mils. 5.3-7 mils. OiUAlkyd-2 OiUAlkyd-2 mils. mils. OiUAlkyd-2 mils. 6 mils. Notes: (1) Except for vinyl wash primer, the coating and system thicknesses shown are minimums. (2) Coating systems shown for severe areas are satisfactory in less severe areas.
(3) Coastal-within 1,000 feet of ocean or tidal water. High Pollution-air pollution environment such as industrial areas. Mild-othsr than coastal area not in air pollution environment. (4) Inorganic zinc paint shall meet the requirements of Military Specification WD-P-23236A (SH). (5) Organic zinc paint shall meet the requirements of Military Specification DOD-P-21035A. (6) mnyl Wash Primer shall meet the requirements of Military Specification WD-P-15328D. (7) V i l top coat paint shall meet the requirements of the Steel Structures Painting Council, SSPC-Paint 9. (8) Epoxy paint shall meet the requirements of the Steel Structures Painting Council, SSPC-22. (9) OiVAlkyd primer and intermediate coat paint shall meet the requirements of the Steel Stnrctures Painting Council, SSPC-Paint 25.
(10) OiAlkyd top coat paint shall meet the requirements of the Steel Structures Painting Council, SSPC-Paint 104. (11) Urethane top coat paint shall meet the recommendations of the Steel Structures Painting Council, SSPC-PS Guide 1700.
(12) Paints are hazardous because of their flammability and potential toxicity. Safe handling practices are required and should include, but not be limited to, the provisions of SSPC-PA Guide 3, "A Guide to Safety in Paint Application." cially control atmospheric conditions inside the enclosure within limits suitable for painting throughout the painting operation. Surfaces painted under cover in damp or cold weather shall remain under cover until the paint dries or weather conditions permit open exposure. Full compensation for providing and maintaining such enclosures shall be considered as included in the prices paid for the various contract items of work involving painting and no additional compensation will be allowed therefore. All blast cleaning, except that performed within closed buildings, and all painting shall be performed during daylight hours unless otherwise provided by the contract documents. 13.2.3 Surface Preparation All exposed surfaces of structural steel, except galvanized or metalized surfaces, shall be cleaned and painted. All surfaces of new structural steel shall be cleaned by the blast-cleaning method unless otherwise specified in the special provisions, or approved in writing by the Engineer. In repainting existing steel structures the method of cleaning shall be as specified in the special provisions.
Any damage to sound paint, on areas not designated for treatment, resulting from the Contractor's operations shall be repaired by the Contractor at own expense to the satisfaction of the Engineer. The methods used in the cleaning of metal surfaces shall conform to the following. 13.2.3.1 Blast Cleaning Abrasives used for blast cleaning shall be either clean dry sand, mineral grit, steel shot, or steel grit, at the option of the Contractor, and shall have a suitable grading to produce satisfactory results.
The use of other abrasives will not be permitted unless approved in writing by the Engineer. Unwashed beach sand containing salt or excessive amounts of silt will not be allowed. All dirt, mill scale, rust, paint, and other foreign material shall be removed from exposed steel surfaces in accordance with the requirements of the Steel Structures Painting Council Surface Preparation Specification No. 10, SSPC-SP10-Near-White Blast Cleaning. Blast cleaning shall leave all surfaces with a dense and uniform anchor pattern of not less than 1 nor more than 3 mils. as measured with an approved surface profile comparator. When blast cleaning is being performed near machinery, all journals, bearings, motors, and moving parts shall be sealed against entry of abrasive dust before blast cleaning begins. Blast cleaned surfaces shall be primed or treated the same day blast cleaning is done, unless otherwise authorized by the Engineer. If cleaned surfaces rust or are con- 13.2.3.1 DIVISION11--CONSTRUCTION taminated with foreign material before painting is accomplished, they shall be reblast cleaned by the Contractor at own expense. 13.2.3.2 Steam Cleaning All dirt, grease, loose chalky paint, or other foreign material which has accumulated on the previously painted or galvanized surfaces shall be removed with a steam cleaning apparatus which shall precede all other phases of cleaning. It is not intended that sound paint be removed by this process. Any paint which becomes loose, curled, lifted, or loses its bond with the preceding coat or coats after steam cleaning shall be removed as directed by the Engineer to sound paint or metal surface by the Contractor at own expense.
A biodegradable detergent shall be added to the feed water of the steam generator or applied to the surface to be cleaned. The detergent shall be of such composition and shall be added or applied in such quantity that the cleaning as described in the above paragraph is accomplished. Any residue, detergent, or other foreign material which may accumulate on cleaned surfaces shall be removed by flushing with fresh water. Steam cleaning shall not be performed more than 2 weeks prior to painting or other phases of cleaning. Subsequent painting shall not be performed until the cleaned surfaces are thoroughly dry and in no case in less than 24 hours after cleaning and flushing. 13.2.3.3 Solvent Cleaning Unless otherwise prohibited by the special provisions, solvents shall be used to remove oil, grease, and other soluble contaminants in accordance with the requirements of SSPC-SP1, Solvent Cleaning. Solvent cleaning shall be performed prior to blast cleaning. If contamination remains after blasting, the area shall be recleaned with solvent. 13.2.3.4 Hand Cleaning Wire brushes, either hand or powered, hand scraping tools, power grinders, or sandpaper shall be used to remove all dirt, loose rust and mill scale, or paint which is not firmly bonded to the metal surfaces. Pneumatic chipping hammers shall not be used unless authorized in writing by the Engineer. 13.2.4 Application of Paints The Contractor shall notify the Engirieer, in writing, at least 1 week in advance of the date that cleaning and painting operations are to begin. 593 Painting shall be done in a neat and workmanlike manner. Unless otherwise specified, paint shall be applied by brush, spray, or roller, or any combination thereof peculiar to the paint being applied.
Each application of paint shall be thoroughly cured and any skips, holidays, thin areas, or other deficiencies corrected before the succeeding application. The surface of the paint being covered shall be free from moisture, dust, grease, or any other deleterious materials that would prevent the bond of the succeeding applications. In spot painting, old paint which lifts after the first application shall be removed by scraping and the area repainted before the next application. Paints specified are formulated ready for application and no thinning will be allowed unless otherwise provided in the applicable materials specification for the paint being used. Brushes, when used, shall have sufficient body and length of bristle to spread the paint in a uniform film. Round, oval-shaped brushes, or flat brushes not wider than 4%inches shall be used. Paint shall be evenly spread and thoroughly brushed out. On all surfaces that are inaccessible for painting by regular means, the paint shall be applied by sheepskin daubers, bottle brushes, or by any other means approved by the Engineer.
Rollers, when used, shall be of a type that do not leave a stippled texture in the paint film. Rollers shall be used only on flat, even surfaces to produce a paint film of even thickness with no skips, runs, sags, or thin areas. Paint may be applied with airless or conventional spray equipment. Suitable traps or separators acceptable to the Engineer shall be furnished and installed in the airline to each spray pot to exclude oil and water from the air. Any spray method which produces excessive paint build-up, runs, sags, or thin areas in the paint film, or skips and holidays, will be considered unsatisfactory and the Engineer may require modification of the spray method or prohibit its use and require brushing instead. Mechanical mixers shall be used to mix paint.
Prior to application, paint shall be mixed a sufficient length of time to thoroughly mix the pigment and vehicle together, and shall be kept thoroughly mixed during its application. The dry film thickness of the paint will be measured in place with a calibrated magnetic film thickness gage according to Steel Structures Painting Council SSPC-PA2. The thickness of each application shall be limited to that which will result in uniform drying throughout the paint film. Succeeding applications of paint shall be of such shade as to contrast with the paint being covered. HIGHWAY BRIDGES Structures shall be blast cleaned and painted with the total thickness of undercoats before erection. After erection and before applying subsequent paint, all areas where paint has been damaged or has deteriorated and all exposed un~aintedsurfaces shall be thoroughly cleaned and spot painted with the specified undercoats to the specified thickness. Surfaces exposed to the atmosphere and which would be inaccessiblefor painting after erection shall be painted the full number of applications prior to erection. Vinyl wash primer, if required, shall not be applied more than 12 hours before application of the succeeding coat of paint. The vinyl wash primer shall be applied by spraying to produce a uniform wet film on the surface. The dry film thickness shall be between 0.3 and 0.5 mils. The painting of areas under joint connection and splice plates shall conform to Article 11.5.6.3. 13.2.4.1 Application of Zinc-Rich Primers Zinc-rich primers, which include organic and inorganic zinc primers, shall be applied by spray methods. On areas inaccessible to spray application, the paint may be applied by brush or daubers. Mechanical mixers shall be used in mixing the primer.
After mixing, zinc-rich primers shall be strained through a metal 30-60 mesh screen or a double layer of cheesecloth immediately prior to or during pouring into the spray pot. An agitating spray pot shall be used in all spray application of zinc-rich primers. The agitator or stirring rod shall reach to within 2 inches of the bottom of the spray pot and shall be in motion at all times during primer application. Such motion shall be sufficient to keep the primer well mixed. Spray equipment shall provide the proper pot pressure and atomization pressure to produce a coating the composition of which shall comply in all respects to the specifications for zinc paint. The hose from pot to nozzle shall not be more than 75 feet long, nor be used more than 15 feet above or below the pot. Cured, zinc-rich primer shall be free from dust, dirt, salt, or other deleterious deposits and thoroughly dry before applying vinyl wash primer. In addition, the application of inorganic zinc paints shall conform to the following paragraphs. Succeeding applications of inorganic zinc paints shall be applied within 24 hours, but not less than 30 minutes after prior application of such paint. In areas where mud-cracking occurs in the inorganic zinc paint, it shall be blast cleaned back to soundly bonded paint, and recoated to the same thickness by the same methods specified for the original coat.
13.2.4 Paint shall be cured for 48 hours at a relative humidity of at least 45% before the application of vinyl wash primer. The cured inorganic zinc paint shall be hosed down with water and be in a surface dry condition before the application of vinyl wash primer if the vinyl wash primer is not applied within 3 weeks after the inorganic zinc paint is applied, or when there is evidence of dust, dirt, salt, or other deleterious deposits on the inorganic zinc paint.
13.2.5 Measurement and Payment Cleaning and painting structural steel will be paid for on the basis of lump sum prices, unless otherwise specified in the special provisions. The lump sum prices paid for clean structural steel and for paint structural steel or the lump sum price paid for clean and paint structural steel shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in cleaning and painting structural steel as shown on the plans, and as specified in these specifications and the special provisions, and as directed by the Engineer. 13.3 PAINTING GALVANIZED SURFACES All galvanized surfaces that are to be painted shall first be cleaned by washing with mineral spirit solvent sufficient to remove any oil, grease, or other materials foreign to the galvanized coating. After cleaning, vinyl wash primer shall be applied to such surfaces.
The vinyl wash primer shall be applied by spraying to produce a uniform wet film on the surface. The dry film thickness shall be between 0.3 and 0.5 mils. Finish paint to be applied to primed galvanized surfaces shall be as shown on the plans or otherwise specified. If not shown or otherwise specified, the finish paint shall be the same as that used on adjacent metal work or shall be as directed by the Engineer.
No separate payment will be made for preparing and painting galvanized surfaces and full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in preparing and painting galvanized surfaces as shown on the plans, and as specified in these specifications and the special provisions, and as directed by the Engineer will be considered as included in the prices paid for the various contract items of work involving the galvanized surfaces. 13.4 DIVISION 11--CONSTRUCTION 595 13.4 PAINTING TIMBER 13.4.5 Painting Treated Timber 13.4.1 General Unless otherwise shown on the plans or specified in the special provisions, all new timber requiring painting shall be painted with three applications of paint. The paint used for various applications will be as specified in these specifications or as shown on the plans or specified in the special provisions. The painting of previously painted surfaces shall be as required by the plans and specifications.
Timber treated with creosote or oil-borne, pentachlorophenol preservatives shall normally not be painted. Timber treated with water-borne preservatives shall be clean and be reduced to no more than 20% moisture content before it is painted. Any visible salt crystals on the wood surface shall be washed and brushed away, and the moisture content reduced again to the specified level before painting.
Stored timber awaiting painting shall be covered and stacked with spreaders to ensure air circulation. 13.4.2 Preparation of Surfaces 13.4.6 Payment All cracked or peeled paint, loose challcy paint, dirt and other foreign material shall be removed by wire brushing, scraping or other means immediately prior to painting. The moisture content of the timber shall not be more than 20% at the time of the first application. No separate payment will be made for preparing surfaces and for painting new timber. The painting of existing timber will be paid for on the basis of lump sum prices. Full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in preparing surfaces and painting timber as shown on the plans, and as specified in these specifications and the special provisions, and as directed by the Engineer will be considered as included in the prices paid for the various contract items of work involving new timber or the prices paid for painting existing timber. 13.4.3 Paint Paint for timber structures, except as otherwise provided herein, shall conform to the Specification for White and Tinted Ready-Mixed Paint, AASHTO M 70. The paint as specified is intended for use in covering previously painted surfaces. When it is applied to unpainted timber, turpentine and linseed oil shall be added as required by the character of the surface in an amount not to exceed 1 pint per gallon of the paint as specified. The paint shall be either white or tinted as directed by the Engineer. If a black finish paint is specified, the first or prime coat shall be as specified above. Black paint shall conform to the Specifications for Black Paint, AASHTO M 68. 13.4.4 Application When permitted in writing by the Engineer, the first application of paint may be applied prior to erection. After the first application has dried and the timber is in place, all cracks, checks, nail holes, or other depressions shall be puttied flush with the surface and allowed to dry before the second application of paint.
Paint shall be applied by brush, air spray, or roller, spread evenly, and worked thoroughly into all seasoning cracks, comers, and recesses. No later coat shall be applied until the full thickness of the previous coat has dried. Final brush strokes with aluminum paint shall be made in the same direction to ensure that powder particles "leaf' evenly. 13.5 PAINTING CONCRETE 13.5.1 Surface Preparation Prior to painting concrete surfaces, laitance and curing compounds shall be removed from the surface by abrasive blast cleaning in accordance with Article 13.2.3.1. Concrete surfaces shall be thoroughly dry and free of dust at the time the paint is to be applied. Any artificial drying procedures and methods shall be subject to approval by the Engineer. 13.5.2 Paint Unless otherwise specified in the special provisions, paint to be applied to concrete surfaces shall be acrylic emulsion and shall comply in all respects to Federal Specification TT-P-19 (latest revision), Paint, Acrylic Emulsion, Exterior. This paint may be tinted by using "universal" or "all purpose" concentrates. 13.5.3 Application Acrylic emulsion paint shall be applied in not less than two applications to produce a uniform appearance. 596 HIGHWAY BRIDGES The paint shall be applied only when the ambient temperature is 50°F, or above. Painting will not be permitted when it can be anticipated that the ambient temperature will drop below 50°F during the application and drying of the paint. 13.5.4 Measurement and Payment Preparing and painting concrete will be measured either by the lump sum or by the square foot as listed in the 13.5.3 schedule of bid items. When measured by the square foot, measurement will be determined along the surface of the actual area painted. The contract price paid per lump sum or square foot for prepare-and-paintconcrete shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in preparing the concrete and applying the paint to concrete surfaces, as shown on the plans, and as specified in these specifications and the special provisions,and as directed by the Engineer. Section 14 STONE MASONRY 14.1 DESCRIPTION 14.2.1.2 Ashlar Stone This work shall consist of the construction of stone masonry structures and the stone masonry portions of composite structures in accordance with these Specifications and in reasonably close conformity with the lines and grades shown on the plans or established by the Engineer. Stone for ashlar masonry shall be reasonably fine grained and uniform in color. Preferably, stone shall be from a quarry, the product of which is known to be of satisfactory quality. Stone shall be of such character that it can be brought to such lines and surfaces, whether curved or plane, as may be required.
Any stone having defects that have been repaired with cement or other materials shall be rejected. 14.1.1 Rubble Masonry Rubble masonry, as here specified, shall include various classes of roughly squared and dressed stone laid in cement mortar. 14.2.2 Shipment and Storage of Stone Quarry operations and delivery of stone to the point of use shall be organized to insure deliveries well ahead of masonry operations. A sufficiently large stock of the specified stone shall be kept on the site at all times, to permit adequate selection of stone by the masons. The stone shall be kept free from dirt, oil, or any other injurious material which may prevent the proper adhesion of the mortar or detract from the appearance of the exposed surfaces. 14.1.2 Ashlar Masonry Ashlar masonry shall consist of first-class cut stone masonry laid in regular courses and shall include all work in which, as distinguished from rubble masonry, the individual stones are dressed or tooled to exact dimensions. 14.2 MATERIALS 14.2.3 Mortar 14.2.1 Stone for masonry shall be tough, dense, sound and durable and free of seams, cracks, inclusions or other structural defects. Stone shall be of the type and quality shown on the plans or otherwise specified, Prior to shipment of stone to the job site, the Contractor shall obtain approval of the proposed source and shall submit a representative sample of stone to the Engineer for inspection and, if necessary, testing. The sample shall be dressed and finished as specified for use in the work and shall not be less than 6 inches in any dimension. All stone used in the work shall be of a quality comparable to that of the sample submitted. The ingredients used in making mortar shall conform to the following requirements: Portland Cement, Admixtures and Water; Section 8 Masonry Cement; ASTM C 91 Hydrated Lime; ASTM C 207 Quick Lime used to make lime putty; ASTM C 5 Sand Aggregate; AASHTO M 45 (ASTM C 144) The proportions of materials shall be such that the volume of sand in a damp, loose condition is between 2Y4 and 3 times the volume of the cementitious materials. The cementitious materials shall consist of either one part of portland cement to between Y4 and % parts of hydrated lime or lime putty, or one part of portland cement to between one and two parts of masonry cement. Premixed materials conforming to these requirements may be used. 14.2.1.1 Rubble Stone Stone for mortar rubble masonry shall be free from rounded, worn, or weathered surfaces. All weathered stone shall be rejected.
597 598 HIGHWAY BRIDGES Admixtures shall be used only when specified or approved by the Engineer. 14.3 MANUF'ACTUREOF STONE FOR MASONRY 14.3.1 General Each stone shall be free from depressions and projections that might weaken it or prevent it from being properly bedded, and shall be of a shape to meet the requirements for the class of masonry specified. When no dimensions are shown on the plans, the stones shall be furnished in the sizes and face areas necessary to produce the general characteristics and appearance as indicated on the plans. The thickness of courses, if varied, shall diminish regularly from bottom to top of wall.
The size of ring stones in arches shall be as shown on the plans. When headers are required, their lengths shall be not less than the width of bed of the widest adjacent stretcher plus 12 inches. 14.2.3 line, shall be used at all angles and ends of walls. If specified, all comers or angles in exterior surfaces shall be finished with a chisel draft. Bed surfaces of face stones shall be normal to the faces of the stones for about 3 inches and from this point may depart from normal not more than 2 inches in 12 inches. Joint surfaces of face stones shall form an angle with the bed surfaces of not less than 45". All shaping or dressing of stone shall be done before the stone is laid in the wall, and no dressing or hammering which will loosen the stone will be permitted after it is placed. 14.3.3.3 Dressing Stone shall be dressed to remove any thin or weak portions. Face stones shall be dressed to provide bed and joint lines with a maximum variation from true line of 1% inches unless otherwise indicated on the plans or in the special provisions. 14.3.4 Ashlar Masonry 14.3.2 Surface Finishes of Stone 14.3.4.1 Size For the purpose of this specification the surface finishes of stone are defined as follows: Smooth-jinished: Having a surface in which the variations from the pitch line do not exceed %6 inch. Fine-finished: Having a surface in which the variations from the pitch line do not exceed Y4 inch. Rough-finished: Having a surface in which the variations from the pitch line do not exceed % inch. Scabbled: Having a surface in which the variations from the pitch line do not exceed 3/4 inch. Rock-faced: Having an irregular projecting face without indications of tool marks. The projections beyond the pitch line shall not exceed 3 inches and no part of the face shall recede back of the pitch line. 14.3.3 Rubble Masonry 14.3.3.1 Size Individual stones shall have a thickness of not less than 8 inches and a width of not less than 1%times the thickness. No stones, except headers, shall have a length less than 1% times their width. 14.3.3.2 Shape The stones shall be roughly squared on joints, beds, and faces. Selected stone, roughly squared and pitched to The individual stones shall be large and well proportioned. They shall not be less than 12 inches nor more than 30 inches in thickness. 14.3.4.2 Dressing Stones shall be dressed to exact sizes and shapes before being laid and shall be cut to lie on their natural beds with top and bottom truly parallel. Hollow beds will not be permitted. The bottom bed shall be the full size of the stone and no stone shall have an over-hanging top. In rock-face construction the face side of any stone shall not present an undercut contour adjacent to its bottom arris giving a top-heavy, unstable appearance when laid. Beds of face stone shall be fine-finished for a depth of not less than 12 inches. Vertical joints of face stone shall be fine-finished and full to the square for a depth of not less than 9 inches. Exposed surfaces of the face stone shall be given the surface finish indicated on the plans, with edges pitched to true lines and exact batter. Chisel drafts 1%inches wide shall be cut at all exterior comers.
Face stone forming the starling or nosing of piers shall be rough-finished unless otherwise specified. Holes for stone hooks shall not be permitted to show in exposed surfaces. 14.3.4.3 DIVISION II-CONSTRUCTION 599 Stretchers shall have a width of bed of not less than 1% times their thickness. They shall have a length of bed not less than twice nor more than 3% times their thickness, and not less than 3 feet. chine-mixed mortar shall be prepared in an approved mixer and shall be mixed not less than 3 minutes nor more than 10 minutes. Mortar shall be used within 1%hours after mixing and before final set begins. Retempering of mortar shall be done as necessaty to maintain proper consistency during placement. 14.3.5 Arch Ring Stones 14.4.3 Selection and Placing of Stone 14.3.4.3 Stretchers Arch ring stone joint surfaces shall be radial and at right angles to the front faces of the stones. They shall be dressed for a distance of at least 3 inches from the front faces and the soffits, from which points they may depart from a plane normal to the face not to exceed % inches to 12 inches. The back surface in contact with the concrete of the arch barrel shall be parallel to the front face and shall be dressed for a distance of 6 inches from the intrados. The top shall be cut perpendicular to the front face and shall be dressed for a distance of at least 3 inches from the front. When concrete is to be placed after the masonry has been constructed, adjacent ring stones shall vary at least 6 inches in depth. Stratification in arch ring stones shall be parallel to the radial joints and in other stones shall be parallel to the beds.
When specified in the special provisions, a full-sized template of the arch ring shall be laid out near the quarry site, showing face dimensions of each ring stone and thickness of joints. The template shall be approved by the Engineer before the shaping of any ring stone is started, and no ring stone shall be placed in the structure until all ring stones have been shaped, dressed, and approved by the Engineer. 14.4 CONSTRUCTION 14.4.1 Weather Conditions Stone masonry shall not be constructed in freezing weather or when the stone contains frost, except by written permission of the Engineer and subject to such conditians as he or she may require. 14.4.2 Mixing Mortar The mortar shall be hand or machine mixed, as may be required by the Engineer. In the preparation of handmixed mortar, the sand and cement shall be thoroughly mixed together in a clean, tight mortar box until the mixture is of uniform color, after which clean water shall be added in such quantity as to form a stiff plastic mass. Ma- 14.4.3.1 General When masonry is placed on a prepared foundation bed, the bed shall be firm and normal to, or in steps normal to, the face of the wall, and approved by the Engineer before any stone is placed. When it is placed on foundation masonry, the bearing surface of the foundation masonry shall be cleaned thoroughly and in a saturated-surfacedry condition when the mortar bed is spread. All masonry shall be constructed by experienced workmen. Face stones shall be set in random bond to produce the effect shown on the plans. Care shall be taken to prevent the bunching of small stones or stones of the same size. When weathered or colored stones, or stones of varying texture, are being used, care shall be exercised to distribute the various kinds of stones uniformly throughout the exposed faces of the work. Large stones shall be used for the bottom courses and large, selected stones shall be used in the comers. In general, the stones shall decrease in size from the bottom to the top of work. Each stone shall be cleaned and thoroughly saturated with water before being set and the bed which is to receive it shall be clean and well moistened. All stones shall be well bedded in freshly made mortar. The mortar joints shall be full and the stones carefully settled in place before the mortar has set. No spalls will be permitted in the beds. No pinning up of stones with spalls will be permitted in beds. Stone shall not be dropped upon, or slid over the wall, nor will hammering, rolling, or turning of stones on the wall be allowed. They shall be carefully set without jamng the stone already laid and they shall be handled with a lewis or other appliance that will not cause disfigurement. In case any stone is moved or the joint broken, the stone shall be taken up, the mortar thoroughly cleaned from bed and joints, and the stone reset in fresh mortar.
14.4.3.2 Rubble Masonry Rubble masonry shall be laid to line and in courses roughly leveled up.
The bottom or foundation courses shall be composed of large, selected stones and all courses shall be laid with bearing beds parallel to the natural bed HIGHWAY BRIDGES of the material. The vertical joints in each course of rubble masonry shall break with those in adjoining courses at least 6 inches. In no case shall a vertical joint be so located as to occur directly above or below a header. 14.4.3.3 Ashlar Masonry The stones in any one course of ashlar masonry shall be placed so as to form bonds of not less than 12 inches with the stones of adjoining courses. Headers shall be placed over stretchers and, in general, the headers of each course shall equally divide the spaces between the headers of adjoining courses, but no header shall be placed over a joint and no joint shall be made over a header. 14.4.4 Beds and Joints Beds and joints in rubble masonry shall have an average thickness of not more than 1 inch. Beds and joints in ashlar masonry shall be not less than 3/8 inch nor more than % inch in thickness and the thickness of the joint or bed shall be uniform throughout. The thickness of beds in ashlar masonry may vary as shown from the bottom to the top of the work.
However, in each course the beds shall be of uniform thickness throughout. Beds shall not extend in an unbroken line through more than five stones. Joints in ashlar masonry shall be vertical. In all other masonry, joints may be at angles with the vertical from 0" to 45O. Each face stone shall bond with all contiguous face stones at least 6 inches longitudinally and 2 inches vertically. Ring stone joints on the faces and soffits shall be not less than % inch nor more than 1% inches in thickness. Cross beds for vertical walls shall be level and for battered walls may vary from level to normal to the batter line of the face of the wall. All joints shall be completely filled with mortar.
14.4.5 Headers Headers shall hold in the heart of the wall the same size shown in the face and shall extend not less than 12 inches into the core or backing. They shall occupy not less than one-fifth of the face area of the wall and shall be evenly distributed. Headers in rubble masonry walls 2 feet or less in thickness shall extend entirely through the wall. Headers in ashlar masonry shall be placed in each course and shall have a width of not less than 1% times their thickness. In walls having a thickness of 4 feet or less, the headers shall extend entirely through the wall.
In walls of greater thickness, the length of headers shall be not less than 2% times their thickness when the course is 18 inches or less in height, and not less than 4 feet in courses of greater height. Headers shall be spaced not further apart than 8 feet center to center. There shall be at least one header to every two stretchers. 14.4.6 Cores and Backing 14.4.6.1 General Cores and backing shall consist either of roughly bedded and jointed headers and stretchers, as specified above, or of Class B or C concrete, as may be specified. The headers and stretchers in walls having a thickness of 3 feet or less shall have a width or length equal to the full thickness of the wall. No backing will be allowed. 14.4.6.2 Stone When stone is used for cores or backing, at least onehalf of the stone shall be of the same size and character as the face stone, and with parallel ends. No course shall be less than 8 inches thick. Stone backing shall be laid in the same manner as specified above for face stone, with headers interlocking with face headers when the thickness of the wall will permit.
Backing shall be laid to break joints with the face stone.
Stone cores shall be laid in full mortar beds so as to bond not less than 12 inches with face and backing stone and with each other. Bed joints in cores and backing shall not exceed 1 inch and vertical joints shall not exceed 4 inches in thickness. 14.4.6.3 Concrete Concrete used for cores and backing shall conform to the requirements specified in Section 8, "Concrete Structures." The operations involved in the handling and placing of concrete used in cores and backing shall conform to the requirements specified in Section 8. However, the puddling and compacting of concrete adjacent to the ashlar masonry facing shall be done in a manner that will insure the filling of all spaces around the stones and secure full contact and efficient bond with all stone surfaces. 14.4.6.4 Leveling Courses Stone cores and backing shall be carried up to the approximate level of the face course before the succeeding course is started. 14.4.6.4 DIVISION 11-CONSTRUCTION The construction joints produced in concrete cores or backing by the intermittent placing of concrete shall be located, in general, not less than 6 inches below the top bed of any course of masonry. 14.4.7 Facing for Concrete Unless otherwise specified in the Special Provisions, the stone masonry shall be constructed before placing concrete. Steel anchors as shown on the plans or specified in the Special Provisions shall be used. To improve the bond between the stone masonry and the concrete backing, the back of the masonry shall be made as uneven as the stones will permit. After the stone facing has been laid and the mortar has attained sufficient strength, all surfaces against which concrete is to be placed shall be cleaned carefully and all dirt, loose material, and accumulations of mortar droppings removed. When placing concrete all interstices of the masonry shall be filled and the concrete thoroughly spaded and worked until it is brought into intimate contact with every part of the back of the masonry.
601 ified shall be of Class A concrete which shall conform to the requirements of Section 8, "Concrete Structures." Concrete copings shall be made in sections extending the full width of the wall, not less than 12 inches in thickness, and from 5 to 10 feet long. The sections may be cast in place or precast and set in place in full mortar beds. 14.4.9 Dowels and Cramps Where required, coping stone, stone in the wings of abutments, and stone in piers shall be secured with wrought-iron cramps or dowels as indicated on the plans. Dowel holes shall be drilled through each stone before the stone is placed and, after it is in place, such dowel holes shall be extended by drilling into the underlying course not less than 6 inches.
Cramps shall be of the shapes and dimensions shown on the plans or approved by the Engineer. They shall be inset in the stone so as to be flush with the surfaces. Cramps and dowels shall be set in lead, care being taken to completely fill the surrounding spaces with the molten

Structural lumber and timber, solid sawn or glued laminated, in exposed permanent structures, other than running planks on decks, shall be treated in conformance with the requirements of Section 17, "Preserved Treatment of Wood." Temporary structures or lumber and timber of certain species with adequate heartwood requirements, as listed in AASHTO M 168, when permitted by the plans or specifications, do not require preservative treatment. When the special provisions require certification of quality for timber or lumber, the Contractor shall furnish the following certificates of compliance to the Engineer, as appropriate, upon delivery of the materials to the job site: For timber and lumber, a certification by an agency certified by the American Lumber Standards Committee that the timber or lumber conforms to the grade, species, and any other specified requirements. For glued laminated timber, a certification by a qualified inspection and testing agency that the glued laminated timber complies with the grade, species, and other requirements outlined in ANSIIAITC A190.1. If the wood is to be treated with a preservative, a certificate of compliance, as specified in Article 17.3.3, shall be furnished. 16.2.2 Steel Components Rods, plates, eyebars, and shapes shall conform to the requirements of AASHTO M 270 (ASTM A 709) Grade 36 unless otherwise specified.
608 HIGHWAY BRIDGES 16.2.3 Castings Castings shall be cast steel or gray-iron, as specified, conforming to the requirements of Articles 11.3.5 or 11.3.6. 16.2.4 Hardware Bolts, nuts, drift-bolts, and dowels may be of mild steel. Washers may be cast iron ogee or malleable iron castings, or they may be cut from mild steel plate, as specified. Bolts shall have either standard square, hex or dome heads, or economy type (washer) heads. Nails shall be cut or round wire of standard form. Spikes shall be cut or wire spikes, or boat spikes, as specified. Unless otherwise specified, bolts shall comply with ASTM A 307, and shall have coarse threads, Class 2 tolerance conforming to ANSI Standard Specifications. All fasteners, including nails, spikes, bolts, dowels, washers, and lag screws shall be galvanized, unless otherwise specified or permitted. 16.2.5 Galvanizing 16.2.5.1 Unless otherwise specified, all hardware for timber structures shall be galvanized in accordance with AASHTO M 232 (ASTM A 153) or cadmium plated in accordance with AASHTO M 299 (ASTM B 696). All steel components, timber connectors, and castings, other than malleable iron, shall be galvanized in accordance with AASHTO M 111 (ASTM A 123). 16.2.6 Timber Connectors 16.2.6.1 Dimensions The various types of timber connectors shall generally conform to the dimensions shown in Table 16.1 and to the dimensions specified in this Article 16.2.6. 16.2.6.2 Split Ring Connectors Split rings of 2!4-inch inside diameter and 4-inch inside diameter shall be manufactured from hot-rolled carbon steel conforming to the Society of Automotive Engineers Specification SAE-1010.
Each ring shall form a closed true circle with the principal axis of the cross section of the ring metal parallel to the geometric axis of the ring. The metal section shall be beveled from the central portion toward the edges to a thickness less than the rnidsection. It shall be cut through in one place in its circumference to form a tongue and slot. 16.2.3 16.2.6.3 Shear-Plate Connectors Pressed steel shear-plates of 2%-inch diameter shall be manufactured from hot-rolled carbon steel conforming to the Society of Automotive Engineers Specification SAE1010. Each plate shall be a true circle with a flange around the edge, extending at right angles to the face of the plate and extending from one face only, the plate portion having a central bolt hole and two small perforations on opposite sides of the hole and midway from the center and circumference. Malleable iron shear-plates of 4-inch diameter shall be manufactured according to ASTM A 47, Grade 32510, for malleable iron casting. Each casting shall consist of a perforated round plate with a flange around the edge extending at right angles to the face of the plate and projecting from one face only, the plate portion having a central bolt hole reamed to size with an integral hub concentric to the bolt hole and extending from the same face as the flange. 16.2.6.4 Spike-Grid Connectors Spike-gria timber connectors shall be manufactured according to ASTM A 47, Grade 32510, for malleableiron casting. TABLE 16.1 Typical Dimensions of Timber Connectors (dimensions in inches) Split Rings 2-112 in.
Split Ring: Inside Diameter at Center When Closed Thickness of Metal at Center Depth of Metal (Width of Ring) Groove: Inside Diameter Width Depth Bolt Diameter: Hole Diameter Washers, Standard: Round,Cast or Malleable Iron, Diameter Round, Mild Steel, Diameter Thickness Square Plate, Mild Steel, Length of Side Thickness 4 in. DIVISION II-CONSTRUCTION TABLE 16.1 (Continued) Shear Plates (dimensions in inches) 2-98 Shear Plate: Material Diameterofplate DiameterofBoltHole Thickness of Plate Depth of Flange 2-518 4 Pressed Light gage steel Malleable Malleable 16.3 FABRICATION AND CONSTRUCTION 2.62 0.81 0.172 0.42 4.03 0.81 0.20 0.64 4.03 0.94. 0.20 0.64 16.3.1 Workmanship Steel straps or shapes for use with shear plates shall be designed in accordance with accepted engineering practices. Hole Diameter in Straps or Shapes for Bolts 13/16 13/16 13/16 15/16 2.63 4.03 1.55 0.81 0.97 0.27 0.64 0.22 0.50 3.49 4.03 1.55 0.94 0.97 0.27 0.64 0.22 0.50 3.49 Circular DapDimensions: Workmanship shall be first class throughout, and all framing shall be true and exact. Unless otherwise specified, nails and spikes shall be driven with just sufficient force to set the heads flush with the surface of the wood. Deep hammer marks in wood surfaces shall be considered evidence of poor workmanship and sufficient cause for removal of the workman causing them. 16.3.2 Storage of Material D - E G 0.19 0.45 0.25 H I 2.25 2.63 1.07 0.81 0.65 0.13 0.38 0.14 0.34 2.37 13/16 13/16 13/16 15/16 3 3 3-112 2 5/32 2 5/32 2-114 11/64 3 114 3 114 3 114 A B C F 0.81 - Bolt Hole-Diameter in T i r a central circular fillet which forms a bolt hole opening of 1% inch. Fillets in cross section shall be diamond shaped except that the inner circular fillet may be flattened on one side to provide for manufacturer identification. 4 2.62 0.81 0.12 0.35 609 Washers, Standard: Round, Cast or Malleable Iron Diameter 3 Round, Medium Steel Minimum: 2 Diameter Thickness 5/32 Square Plate: Length of Side 3 Thickness 114 Lumber and timber stored at the construction site shall be kept in orderly piles or stacks. Untreated material shall be open-stacked on supports at least 12 inches above the ground surface to avoid absorption of ground moisture and permit air circulation and it shall be so stacked and stickered as to permit free circulation of air between the tiers and courses. In particular cases required by the Engineer, the Contractor shall provide protection from the weather by a suitable covering. The ground underneath and in the vicinity of the timber shall be cleared of weeds and rubbish. The storage area shall be chosen or constructed so that water will not collect under or near the stored timber. 16.3.3 Treated Timber Square grids shall consist of four rows of opposing spikes forming a 4%-inch square grid with 16 teeth that are held in place by fillets. Fillets for the flat grid in cross section shall be diamond shaped. Fillets for the single curve grids shall be increased in depth to allow for curvature and shall maintain a thickness between the sloping faces of the fillets equal to the width of the fillet. Circular grids of 3Y4-inch diameter shall consist of eight opposing spikes equally spaced around the outer circumference and held in place by connecting fillets around the outer diameter and radial fillets projecting to 16.3.3.1 Handling Treated timber shall be carefully handled without sudden dropping, breaking of outer fibers, bruising, or penetrating the surface with tools. It shall be handled with web slings. Cant hooks, peaveys, pikes, or hooks shall not be used. When metal bands are used to bundle members, corner protectors shall be provided to prevent damage to the treated timber. 16.3.3.2 Framing and Boring All cutting, framing, and boring of treated timbers shall be done before treatment insofar as is practicable. When treated timbers are to be placed in waters infested by marine borers, untreated cuts, borings, or otherjoint framings below high-water elevation shall be avoided. 610 HIGHWAY BRIDGES 16.3.3.3 Cuts and Abrasions . All cuts and all recesses formed by countersinking in creosote treated piles or timbers, and all abrasions, after having been carefully trimmed, shall be field treated as specified either in this paragraph or the following paragraph. Cuts and recesses shall be covered with two applications of a mixture of 60% creosote oil and 40% roofing pitch or brush coated with at least two applications of hot creosote oil and covered with hot roofing pitch. Recesses likely to collect injurious materials shall be filled with hot roofing pitch. Unless specified otherwise, hot preservatives shall be heated to a temperature between 150" and 200°F.
Where particularly heavy coatings are required, a suitable plastic compound can be prepared by mixing 10% to 20% of creosote and 80% to 90% of coal-tar roofing pitch. For timbers originally treated with pentachlorophenol, creosote, creosote solutions or water-borne preservatives, all cuts, abrasions and recesses which occur after treatment shall be field treated by two liberal applications of a compatible preservative in accordance with the requirements of the American Wood Preservers Association Standard M 4 entitled, "Standard for the Care of Pressure Treated Wood Products." 16.3.3.4 Bored Holes All holes bored after treatment shall be treated by filling the holes with the preservative used for field treatment. After treatment, any holes not filled with bolts or other items shall be plugged with preservative treated plugs.
16.3.3.5 Temporary Attachment Whenever, with the approval of the Engineer, forms or temporary braces are attached to treated timber with nails or spikes, the resulting holes shall be treated as required for bored holes and shall be filled by driving galvanized nails, spikes, or preservative-treated plugs flush with the surface.
16.3.4 Installation of Connectors Timber connectors shall be one of the following types, as specified on the plans: the split ring, the shear plate, or the spike grid. The split ring and the shear plate types shall be installed in precut grooves of dimensions as given herein or as recommended by the manufacturer. Spike grids shall be forced into the wood so that timbers will be in firm contact. Pressure equipment that does not damage the wood shall be utilized. One acceptable method is to use high-strength bolts or rods fitted with low friction 16.3.3.3 ball-bearing washers made for this purpose. The highstrength bolt will be replaced with specified bolts for the final installation. All connectorsof this type at a joint shall be embedded simultaneously and uniformly. Connector grooves in timber shall be cut concentric with the bolt hole, shall conform to the cross-sectional shape of the rings, and shall provide a snug fit. Inside groove diameter shall be larger than nominal ring diameter in order that the ring will expand slightly during installation. (See Table 16.1.) Fabrication of all structural members using connectors shall be done prior to preservative treatment. When prefabricated from templates or shop details, bolt holes shall not be more than y 1 6 inch from required placement. Bolt holes shall be %6 inch larger than the finished bolt diameter. Bolt holes shall be bored perpendicular to the face of the timber.
Timber after fabrication shall be stored in a manner that will prevent changes in the dimensions of the members before assembly. Timber should be cured before fabrication so that it will remain stable in its dimensions. Timber that shrinks during storage causing predrilled grooves for split rings or plates to become elliptical or causing bolt hole spacing to change will be sufficient reason for rejection.
16.3.5 Holes for Bolts, Dowels, Rods, and Lag Screws Holes for round drift-bolts and dowels shall be bored with a bit %6 inch less in diameter than the bolt or dowel to be used. The diameter of holes for square drift-bolts or dowels shall be equal to the least dimension of the bolt or dowel. Holes for machine bolts shall be bored with a bit the same diameter as the finished bolt, except as otherwise provided for bolts in connectors. Holes for rods shall be bored with a bit %6 inch greater in diameter than the finished rod.
Holes for lag screws shall be bored with a bit not larger than the body of the screw at the base of the thread. To prevent splitting or stripping the threads, the hole for the shank shall be bored the same diameter and to the same depth as the shank. The depth of holes for lag screws shall be approximately 1 inch less than the length under the head. 16.3.6 Bolts and Washers A washer, of the size and type specified, shall be used under all bolt heads (except for timber bolts with economy type heads) and nuts which would otherwise come in contact with wood. 16.3.6 DMSION II-CONSTRUCTION The nuts of all bolts shall be effectually locked after they have been finally tightened. 16.3.7 Countersinking Countersinking shall be done where smooth or flush surfaces are required. All recesses in treated timber, formed for countersinking, shall be treated as specified in Article 16.3.3.3. Recesses likely to collect injurious materials shall be filled with hot roofing pitch. 611 16.3.9.4 Posts Posts shall be fastened to pedestals with dowels of not less than Y4-inch diameter, extending at least 6 inches into the posts, or by other types of connectors as detailed on the plans.
Posts shall be fastened to sills by one of the following methods, as indicated on the plans: All lumber and timber shall be accurately cut and framed to a close fit in such manner that the joints will have even bearing over the entire contact surfaces. Mortises shall be true to size for their full depth and tenons shall fit snugly. No shimming will be permitted in making joints, nor will open joints be accepted. (a) By dowels of not less than Y4-inch diameter, extending at least 6 inches into posts and sills. (b) By drift-bolts of not less than %-inchdiameter driven diagonally through the base of the post and extending at least 9 inches into the sill. Drift bolts shall be driven in holes as required by Article 16.3.5 at a 45' angle and shall enter the post at least 6 inches above the post base. (c) By other types of connectors as detailed on the plans. 16.3.9 Framed Bents 16.3.9.5 Caps 16.3.8 Framing 16.3.9.1 Mud Sills Mud sills shall be firmly and evenly bedded to solid bearing and tamped in place. Mud sills shall be pressure preservative treated for ground contact. Where untreated timber is permitted for mud sills, it shall be of heart cedar, heart cypress, redwood, or other durable timber as approved by the Engineer. 16.3.9.2 Concrete Pedestals Concrete pedestals for the support of framed bents shall be carefully finished so that the sills or posts will take even bearing. Dowels for anchoring sills or posts shall be not less than 7 4 inches in diameter and project at least 6 inches above the tops of the pedestals. These dowels shall be cast in the concrete pedestals. Concrete and reinforcing steel shall conform to the requirements of Sections 8, "Concrete Structures," and 9, "Reinforcing Steel," respectively.
16.3.9.3 Sills Sills shall have true and even bearing on mud sills, piles, or pedestals. They shall be drift-bolted to mud sills or piles with bolts of not less than %-inch diameter and extending into the mud sills or piles at least 6 inches, or by other types of connectors as detailed on the plans. When possible, all earth shall be removed from contact with sills so that there will be free air circulation around the sills. Timber caps shall be placed, with ends aligned, in a manner to secure an even and uniform bearing over the tops of the supporting posts or piles. All caps shall be secured by drift-bolts of not less than %-inch diameter, extending at least 9 inches into the posts or piles, or by other types of connectors as detailed on the plans. The driftbolts shall be approximately in the center of the post or pile. 16.3.9.6 Bracing Bracing shall be bolted through the pile, post, or cap at the ends and at all intermediate intersections using a bolt of not less than 5/8 inches in diameter. Bracing shall be of sufficient length to provide a minimum distance of 8 inches between the outside bolt and the end of the brace. 16.3.10 Stringers Stringers shall be sized at bearings and shall be placed in position so that knots near edges will be in the top portions of the stringers. Outside stringers may have butt joints with the ends cut on a taper, but interior stringers shall be lapped to take bearing over the full width of the floor beam or cap at each end.
The lapped ends of untreated stringers shall be separated at least % inch for the circulation of air and shall be securely fastened by drift-bolting where specified. When stringers are two panels in length the joints shall be staggered.
612 HIGHWAY BRIDGES Unless otherwisespecified in the contract, cross-bridging or blocking shall be placed at the center of each span. Cross-bridging between stringers shall be neatly and accurately framed and securely toe-nailed with at least two nails in each end. All cross-bridging members shall have full bearing at each end against the sides of stringers. Blocking shall be snug-fit and held in place by either prefabricated galvanized steel beam hangers or by tie-rods as detailed on the plans. 16.3.11 Plank Floors Unless otherwise specified, planks for flooring shall be surfaced four sides (S 4 S).
Single plank floors shall consist of a single thickness of plank supported by stringers or joists. The planks shall be laid heart side down, with %-inch openings between them for seasoned material and with tight joints for unseasoned material. Each plank shall be securely spiked to each joist.
The planks shall be carefully graded as to thickness and so laid that no two adjacent planks shall vary in thickness by more than % inch. Two-ply timber floors shall consist of two layers of flooring supported on stringers or joists. The top course shall be laid either diagonal or parallel to the center line of roadway, as specified, and each floor piece shall be securely fastened to the lower course. Joints shall be staggered at least 3 feet. If the top flooring is placed parallel to the center line of the roadway, special care shall be taken to securely fasten the ends of the flooring. At each end of the bridgk these members shall be beveled. 16.3.12 Nail Laminated or Strip Floors The ' ~ p s be placed On at 'ght the center line of roadway.
Each strip shall be nailed to the as shown in Figure 16'3' The 'pikes shall preceding be of sufficient length to Pass through two strips and at least half-way through the third strip. If timber supports are used, every other strip shall be toe-nailed to every other support. The size of the spikes shall be as shown on the plans. When specified on the plans, the strips shall be securely attached to steel supports by the use of approved galvanized metal clips.
Care shall be taken to have each strip vertical and tight against the preceding strip, and bearing evenly on all the supports. 16.3.13 Glue Laminated Panel Decks Unless otherwise specified, deck panels shall be pressure preservative treated with creosote or pentachlorophenol with Type A, C, or D carrier. When it is not 16.3.10 possible to complete the fabrication and drilling of glulam members for field connections before treating, a preservative treatment shall be applied to cut or drilled areas in the field, in accordance with Articles 16.3.3.3 and 16.3.3.4. Panels shall not be dragged or skidded.
Glue laminated deck panels shall be handled, and transported in a way to prevent bending the panels, especially transverse to the laminated pieces. When lifted, they shall be supported at a sufficient number of points to avoid overstressing, and the edges shall be protected from damage. When dowels are shown on the drawings between deck panels, a template or drilling jig shall be used to ensure that dowel holes are accurately spaced.
The holes shall be drilled to a depth Y4 inch greater than one-half the dowel length and of the same diameter as the dowel unless otherwise shown on the drawings. A temporary dowel shall be used as a check for snug fit prior to production drilling. The dowels shall be of the size shown on the drawings with the tips slightly tapered or rounded. A lubricant may be used to facilitate the connection process. The tips of the dowels shall be partially and equally started into the holes of the two panels being joined. The panels shall be drawn together keeping the edges parallel, until the panels abut tightly. Each panel shall be securely fastened to each stringer or girder as shown on the drawings. 16.3.14 Composite Wood-Concrete Decks Shear connectors needed to resist shear and provide hold-down capacity between timber and concrete elements which are designed for composite action shall be furnished and installed in conformance with the details shown on the plans or specified in the prov~sions. is deIf no such details are provided and the the Contractor rib d on the plans as being sc e details and deshall worhg drawings for vices for approval by the Engineer before the subject work is begun. 16.3.15 Wheel Guards and Railing Wheel guards and railing shall be accurately framed in accordance with the plans and erected true to line and grade. Unless otherwise specified, wheel guards, rails, and rail posts shall be surfaced four-sides (S 4 S). Wheel guards shall be laid in sections not less than 12 feet long, except where necessary to match expansion joints or end joints. Railings shall conform to the requirements in Section 20, "Railings." 16.3.16 DIVISION II-CONSTRUCTION 16.3.16 Trusses Trusses, when completed, shall show no irregularities of line.
Chords shall be straight and true from end to end in horizontal projection and, in vertical projection, shall show a smooth curve through panel points conforming to the correct camber. All bearing surfaces shall fit accurately. Uneven or rough cuts at the points of bearing shall be cause for rejection of the piece containing the defect. 16.4 PAINTING Rails and rail posts of timber and any other parts designated on the plan or in the special provisions to be painted shall be painted with three coats of specification paint. Paint and its application shall conform to the requirements in Section 13, "Painting." Metal parts, except for hardware, galvanized or cadmium plated metal, and malleable iron, shall be given one coat of shop paint and, after erection, two coats of field paint as specified in Section 13, "Painting." 16.5 MEASUREMENT The quantities to be paid for will be the number of thousand feet board measure (Mbm)of each species and grade of lumber and timber listed in the schedule of bid items, complete in place and accepted.
Measurements of lumber and timber will be computed from the nominal dimensions and actual lengths.
The cross-sectional dimensions on the plans will be interpreted as standard sizes. The standard cross-sectional dimensions will be used in the computations even though the actual size is less in the dimension specified. Timber in wheel guards will be included. Timber in piling, railing, and other items for which separate payment is provided will not be included. Measurements for glued laminated girders and beams will be computed from the applicable finished dimensions and actual lengths. Quantities for glue laminated girders and beams to be paid for will be the linear feet for each size and stress combination. The measurement of lumber and timber and of glued laminated girders and beams will include only such material as is a part of the completed and accepted work, and will not include materials used for erection purposes, such as falsework, bracing, sheeting, etc. 16.6 PAYMENT Payment for timber, lumber, and glued laminated girders and beams shall be considered to be full compensation for all costs of furnishing of materials, including hardware and timber connectors, preservative treatment, equipment, tools, and labor for the fabrication, erection, and painting necessary to complete all of the work in compliance with the plans and specifications in a satisfactory manner. Metal parts, other than hardware and timber connectors, will be measured and paid for as provided in Section 23, "Miscellaneous Metal." Railings and concrete will be measured and paid for as provided in Sections 20, "Railings" and 8, "Concrete Structures," respectively.
h h r r 9" 9' Nominal 2-inch thick lumber lamination indicates nails in first lamination x indicates nails in second lamination + indicates nails In third lamination -1' 613 for 4- mminal lamination depth 1-l/rfor 6" or greater nominal lamination depth FIGURE 16.3 Nail Placement Pattern Section 17 PRESERVATIVE TREATMENT OF WOOD 17.1 GENERAL Unless otherwise specified in the Special Provisions or shown on the design drawings, timber railings and posts and timber that are to be painted shall be treated with pentachlorophenol with a Type C solvent or with a water-borne preservative of either Q p e CCA or ACZA. This work shall consist of treating wood, including lumber, timber, piles and poles, with designated preservatives in accordance with these Specifications. It shall include furnishing all materials, preparing, treating, and performing all work to complete treating the wood products required for the project. The type of preservative treatment required shall be as specified in the special provisions or as noted on the plans. When a specific type of preservative is not called for, the kind of preservative to be used shall be adopted for its suitability to the conditionsof exposure to which it will be subjected and shall be subject to approval of the Engineer. The handling and care of treated woods shall conform to the requirements of Sections 4, "Driven Foundation Piles," and 16, "Timber Structures." 17.2.3 Coal-tar Roofing Cement For purposes of these specifications pitch, coal-tar pitch, coal-tar roofing pitch, or coal-tar roofing compound shall mean coal-tar roofing cement wherever the terms are used. Coal-tar roofing cement is a residue of the manufacturing of coke and creosote from bituminous coal. It shall be a thick, heavy-bodied, and paste-like material. When called for, it can be mixed with creosote. It may or may not contain fibrous material.
17.2 MATERIALS 17.2.1 Wood 17.3 IDENTIFICATION AND INSPECTION Piling shall conform to the requirements of Section 4, "Driven Foundation Piles." Timber and lumber shall conform to the requirements of Section 16, "Timber Structures." 17.3.1 Branding and Job Site Inspection Each piece of treated timber shall bear a legible brand, mark, or tag indicating the name of the treater and the specification symbol or specification requirements to which the treatment conforms. Treated wood products bearing the quality mark of the American Wood Preservers Bureau (AWPB) will be acceptable. The Engineer shall be provided adequate facilities and free access to the necessary parts of the treating plant for inspection of material and workmanship to determine that the contract requirements are met. The Engineer reserves the right to retest all materials after delivery to the job site and to reject all materials which do not meet the requirements of the contract; provided that, at the job site reinspection, conformance within 5% of contract requirements shall be acceptable. Reinspection at the job site may include assay to determine retention of preservatives and extraction and analysis of preservative to determine its quality. 17.2.2 Preservatives and Treatments Timber preservatives and treatment methods shall conform to AASHTO M 133. The type of preservative furnished shall be in accordance with that specified or as noted on the plans. It should be noted that AASHTO M 133 designates the preservatives and retentions recommended for Coastal Waters and in marine structures and further that timber for use in "ground or water contact" has requirements that differ from timbers for use "not in ground or water contact." In some instances there is a range of retentions offered which provides for different degrees of exposure based on climate or degree of insect infestation. Unless the higher retentions are specified, not less than the minimum retention is required. 615 616 HIGHWAY BRIDGES 17.3.2 17.3.2 Inspection at Treatment Plant 17.3.3 Certificate of Compliance Unless otherwise specified, inspection of materials and preservative treatment shall be the responsibility of the Contractor and the supplier of treated wood products. Inspections shall be conducted in accordance with AASHTO Specification M 133 (AWPA Standards) by the treater or an independent commercial inspection agency approved by the American Wood Preservers Bureau (AWPB) and the Engineer. The inspection agency shall be engaged by the Contractor directly or through his or her supplier. No direct compensation will be made for these inspection costs, it being understood that the costs of inspection are included in the contract bid prices for treated wood products or construction items of work. Whenever specified or requested by the Engineer, a certificate of compliance with copies of the inspection reports attached shall be furnished to the Engineer with each shipment of material. Such certificates shall identify the type of preservative used and the quantity in pounds per cubic foot (assay method) and shall be signed by the . treater or the qualified independent inspection agency. 17.4 MEASUREMENT AND PAYMENT No separate measurement and payment will be made for preservative treatment as such work is,a part of the work included in furnishing preservative treated materials. Section 18 BEARINGS 18.1 SCOPE 18.2.2 ASTM Standards The following ASTM Standards are relevant to this This section covers the construction and installation of structural bearings that consist of one or more of the following component types: metal rocker and roller bearings, PTFE sliding bearings (flat and curved), plain elastomeric pads, fiberglass reinforced elastomeric pads, cotton duck reinforced pads, steel reinforced elastomeric bearings, pot bearings, disc bearings, and bronze and copper alloy bearings (flat and curved). At the discretion of the Engineer, other component types may be used, but the construction, installation and testing requirements must then be agreed by the Engineer before the start of fabrication. The section also covers ancillary items such as masonry, sole and load distribution plates, bedding materials, anchor bolts, lubricants and adhesives. ASTM A 167 ASTM A 240 ASTM A 307 ASTMA781 ASTM A 788 18.2 APPLICABLE DOCUMENTS ASTM A 802 18.2.1 AASHTO Standards ASTM B 22 The following AASHTO Standards are relevant to this section.
ASTM B 29 ASTM B 36 AASHTO M 102 Steel Forgings, Carbon and Alloy for General Use (ASTM A 668) AASHTO M 107 Bronze Castings for Bridges and Turntables (ASTM B 22) AASHTO M 108 Rolled Copper-Alloy Bearing and Expansion Plates and Sheets for Bridges and Other Industrial Uses (ASTM B100) AASHTO M 164 High-Strength Bolts for Structural Steel Joints (ASTM A 325) AASHTO M 251 Specifications for Plain and Steel Laminated Bearings for Bridges (ASTM D 4014) AASHTO M 253 Heat-Treated Steel Structural Bolts 150 Ksi Minimum Tensile Strength (ASTM A 490) AASHTO M 270 Structural Steel for Bridges (ASTM A 709) ASTM B 100 ASTM B 103 ASTM B 438 ASTM D 395 ASTM D 412 ASTM D 429 ASTM D 518 ASTM D 573 617 Specification for Stainless and HeatResisting Chromium-Nickel Steel Plate Sheet, and Strip. Specification for Heat-Resisting Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels Specification for Carbon Steel Externally Threaded Standard Fasteners Standard Specification for Castings, Steel and Alloy, Common Requirements, for General Industrial Use Specification for Steel Forgings, General Requirements Practice for Steel Castings, Textures and Discontinuities, Evaluation and Specifying by Visual Examination Bronze Castings for Bridges and Turntables (AASHTO M 107) Specification for Pig Lead Specification for Brass Plate, Sheet, Strip, and Rolled Bar Specification for Rolled Copper Alloy Bearing and Expansion Plates for Bridge and Other Structural Uses (AASHTO M 108) Specification for Phosphor-Bronze Plate, Sheet, Strip and Rolled Bar Specification for Sintered Bronze Bearings (Oil Impregnated) Test Methods for Rubber PropertyCompression Set Test Methods for Rubber PropertyTension Test Test Methods for Rubber PropertyPeel Test Test Method for Rubber Deterioration-Surface Cracking Test Method for Rubber-Deterioration in an Air Oven 618 ASTM D 746 ASTM D 792 ASTM D 903 ASTM D 1043 ASTM D 1149 ASTM D 1777 ASTM D 2000 ASTM D 2240 ASTM D 2256 ASTM D 3293 ASTM D 4014 ASTM D 4894 HIGHWAY BRIDGES Test Method for Brittleness Temperature of Plastics and Elastomers by Impact Test Method for Specific Gravity (Relative Density) and Density of Plastics by Displacement Test Method for Peel or Stripping Strength of Adhesive Bonds Stiffness Properties of Plastics as a Function of Temperature by Means of a Torsion Test Test Method for Rubber Deterioration--Surface Ozone Cracking in a Chamber. Method of Measuring Thickness of Textile Materials Classification System for Rubber Products in Automotive Applications Test Method for Rubber PropertyDurometer Hardness Test Method for Breaking Load (Strength) and Elongation of Yarn by the Single-Strand Method Specification for PTFE Resin Molded Sheet Specification for Plain and SteelLaminated Elastomeric Bearings for Bridges (AASHTO M 251) Specification for Polytetrafluoroethylene (PTFE) Granular Molding and Extrusion Materials Specification for Polytetrafluoroeth'lene (PTm) Resin Produced from Dispersion am ASTM D 4895 18.2.3 Other Standards ANSI/AASHTO/ Bridge Welding Code AWS D l .5 M1L-S-8660C Grease for pot bearing elements Epoxy (Federal specification) MMM-A-134 Brass (Federal specification) QQ-B-626 TT-S-230 Caulk (Federal specification) 18.3 GENERAL REQUIREMENTS Bearings shall be constructed in accordance with the details shown on the plans and specifications.When complete details are not provided, bearings shall be furnished that conform to the limited details shown on the plans and shall provide the performance characteristics specified. 18.2.2 18.4 MATERIALS 18.4.1 General 18.4.1.1 Steel 18.4.1.1.1 Rolled steel shall be of the type required on the plans and shall satisfy the testing requirements of the standard to which it conforms.
Unless otherwise specified, it shall conform to AASHTO M 270 (ASTM A 709) Grade 36 and shall cause no adverse electrolyticor chemical reaction with other components of the bearing. It shall be free of all rust and mill scale. 18.4.1.1.2 Unless otherwise specified by the Engineer, steel laminates in steel reinforced elastomeric bearings shall be made from rolled mild steel conforming to M 270 Grade 36, Grade 50 (ASTM A 36, A 572), or equivalent, and shall have a nominal thickness not less than 16 gage. Holes in laminates, not specified on the plans but used for manufacturing purposes, shall be permitted only with the written approval of the Engineer. 18.4.1.1.3 Cast steel shall satisfy the requirementsof ASTM A 802 and be free of all blow-holes and impurities larger than l/s inch.
The inside wall of the pot in pot bearings and the contact surface of metal rocker or roller bearings shall be free of blow-holes or impurities of any size. 18.4.1.1.4 Forged steel shall satisfy the requirements of ASTM A 788.
18.4.1.1.5 Unless otherwise specified by the Engineer, stainless steel shall conform to ASTM A 167 or A 240 type 304, and have a minimum thickness of 20 gage. Stainless steel in contact with PTFE sheet shall be polished to a #8 mirror finish. 18.4.1.1.6 Steel weld metal shall be chosen to be compatible with the parent materials and the welding process used and shall be approved by the Engineer. Stainless steel weld used for overlays shall be type 309L. Bolts shall conform to AASHTO M 164 (ASTM A 325), AASHTO M 253 (ASTM A 490) or ASTM A 307 unless specified otherwise. 18.4.1.1.7 18.4.2 Special Material Requirements for Metal Rocker and Roller Bearings The steel at the contact surface of a metal bearing may be hardened provided that, after hardening, it satisfies the strength and ductility requirements of the contract plans and material specifications. 18.4.3 DIVISION 11-CONSTRUCTION 18.4.3 Special Material Requirements for PTFE Sliding Surfaces 18.4.3.1 PTFE 619 specification MMM-A-134, FEP film or equal, as approved by the Engineer. 18.4.3.3 Lubricants 18.4.3.1.1 PTFE resin shall be 100% pure new material and shall comply with ASTM D 4894 or D 4895. It shall satisfy the requirements of Table 18.4.3.1-1. No reclaimed material shall be used. Finished PTFE sheet, strip and fabric shall be resistant to acids, alkalis, and petroleum products, stable at temperatures from -360°F to +500°E, nonflammable, and nonabsorbing of water. Lubricant, if used, shall consist of a combination of solids which does not react chemically or electrolytically with the PTFE and its mating surface and shall remain stable in the environmental conditions expected at the bridge site. 18.4.3.1.2 Filler material, when used in PTFE, shall be milled glass fiber, carbon fiber or other approved fiber.
The filler shall not react chemically with the PTFE but shall adhere to it so that the two act compositely. The phosphor bronze back plate shall conform to AASHTO M 108 (ASTM B 100) and the porous bronze layer shall conform to ASTM B 103. 18.4.3.1.3 Finished PTFE sheet shall be made from virgin PTEE resin or virgin PTEE resin uniformly blended with approved filler. The maximum filler content shall be 15% for fiberglass and 25% for carbon fibers. The maximum filler content for other materials shall be determined by the Engineer. The PTEE sheet shall satisfy the requirements of Table 18.4.3.1-1. Values for intermediate filler contents may be obtained by interpolation. 18.4.3.1.4 Woven fabric PTFE shall be made from oriented multi-filament PTFE fibers or from a mixture of PTFE fibers made from twisted, slit PTFE tape and other fibers. It shall conform to the requirements of Table 18.4.3.1-1. 18.4.3.2 Adhesives Adhesive used for bonding sheet PTFE shall be an epoxy material satisfying the requirements of federal 18.4.3.4 Interlocked Bronze and Filled PTFE Structures 18.4.4 Special Material Requirements for Pot Bearings 18.4.4.1 The rotational element of the pot bearing shall be made from an elastomeric compound, with a hardness of 50 10 on the Shore A scale. It shall be made from all new material. The raw polymer on which it is based shall be either polychloroprene(neoprene) or polyisoprene (natural rubber). The compound shall satisfy the physical property requirements for a 50 hardness material as specified in Tables 18.4.5.1-1Aor -1B. + 18.4.4.2 The elastomer may be lubricated with a silicone grease which does not react chemically with the elastomer and which does not alter its properties within the range of environmental conditions expected at the bridge site. 18.4.4.3 The sealing rings shall be made of brass conforming to ASTM B 36 (half hard) for rings of rectangular cross-section,and to federal specification QQ-B-626, composition 2, for rings of circular cross-section.The Engineer TABLE 18.4.3.1-1 Physical Properties of PTFE Physical Property Specific Gravity Melting point ("F) Tensile Strength (psi) Elongation at Break (%) ASTM Test Method Sheet (Unfilled) Sheet with 15% glass fibers Sheet with 25% carbon fibers Woven fabric D 4894, D 4895, or D 5977 D 4894, D 4895, or D 5977 D 4894, D 4895, or D 5977 D 4894, D 4895, or D 5977 2.16 t 0.03 2.20 + 0.03 2.10 t 0.03 - 623 + 2 621 + 18 621 + 18 - 28001 20002 13002 24,000 2001 1502 7S2 35l Using Test Method ASTM D 2256 Using Test Method ASTM D 638 620 HIGHWAY BRIDGES 18.4.3.1.1 may, at own discretion, approve other sealing ring materials on the basis of test evidence which demonstrates adequate sealing properties and durability of the material.
laminated bearings shall develop a minimum peel strength of 40 lblin. Peel strength tests shall be performed by ASTM D 429 Method B. 18.4.5 Special Material Requirements for Steel Keinforced Elastomeric Bearings and Elastomeric Pads 18.4.6 Special Material Requirements for Bronze or Copper Alloy Sliding Surfaces 18.4.5.1 Elastomer The raw elastomer shall be either virgin neoprene (polychloroprene) or virgin natural rubber (polyisoprene). The elastomer compound shall be classified as being of low temperature grade 0, 2, 3 , 4 or 5. The grades are defined by the testing requirements in Tables 18.4.5.1-1A and -lB. A higher grade of elastomer may be substituted for a lower one.
In the absence of more specific information, bearings shall be Grade 3,60 durometer elastomer. The elastomer compound shall meet the minimum requirements of Tables 18.4.5.1-1A and -lB except as otherwise specified by the Engineer. The nominal hardness of the compound shall lie between 50 and 60 for reinforced bearings and between 50 and 70 for plain pads. Test requirements may be interpolated for intermediate hardness. If the material is specified by its shear modulus, its measured shear modulus shall lie within 15% of the specified value. A consistent value of hardness shall also be supplied for the purpose of defining limits for the tests in Tables 18.4.5.1-lA and - 1B. If the hardness is specified, the measured shear modulus must fall within the range of Table 14.6.5.2.1 in Article 14.6.5.2 of Division I. When test specimens are cut from the finished product, the physical properties shall be permitted to vary from those specified in Tables 18.4.5.1-1A and -lB by 10%. All material tests shall be carried out at 73" + 4°F unless otherwise noted. Shear modulus tests shall be carried out using the apparatus and procedure described in annex A of ASTM D 4014, amended where necessary by the requirements of Tables 18.4.5.1-1A or -lB. 18.4.5.2 Fabric Reinforcement Fabric reinforcement shall be woven from 100% glass fibers of "E" type yarn with continuous fibers. The minimum thread count in either direction shall be 25 threads per inch. The fabric shall have either a crowfoot or an 8 Harness Satin weave. Each ply of fabric shall have a minimum breaking strength of 800 lblin. of width in each thread direction. 18.4.5.3 Bond The vulcanized bond between fabric and reinforcement shall have a minimum peel strength of 30 lblin. Steel 18.4.6.1 Bronze and Copper Alloys 18.4.6.1.1 Bronze Bronze components shall conform to the requirements of AASHTO M 107 (ASTM B 22) alloy C90500, C91100 or C86300. Alloy C91100 shall be furnished unless otherwise specified. Components may be cast, rolled or forged. Castings shall be free of blow-holes larger than l/s inch and contact surfaces shall be free of all blow-holes of any size. 18.4.6.1.2 Rolled Copper-Alloy Rolled copper-alloy bearing and expansion plates shall conform to the Specification for Rolled Copper-Alloy Bearing and Expansion Plates and

Only new materials shall be used, and no material that has been previously tested shall be used. The surfaces shall first be thoroughly cleaned with a degreasing solvent. No lubrication other than that specified for the prototype bearings shall be used. The mating surfaces for the test pieces shall have a common area no less than the smaller of the bearing area or 7 in2.
The test pieces shall be loaded in compression to a stress corresponding to their maximum service dead plus live Ioad, which shall be held constant for 1 hour prior to and throughout the duration of the sliding test.
At least 100 cycles of sliding, each consisting of at least + 1 inch of movement, shall then be applied at a temperature of 68°F ? 2°F. The uniform sliding speed shall be 2.5 incheslminute. The breakaway friction coefficient shall be computed for each direction of each cycle, and its mean and standard deviation shall be computed for the sixth through twelfth cycles. The initial static breakaway coefficient of friction for the fist cycle shall not exceed twice the design coefficient of friction.The maximum coefficient of friction for all subsequent cycles shall not exceed the design coefficient of friction. Failure of a single sample shall result in rejection of the entire lot. Following the 100 cycles of testing, the breakaway coefficient of friction shall be determined again and shall not exceed the initial value. The bearing or specimen shall show no appreciable sign of wear, bond failure or other defects. 18.7.2.3 Dimensional Check The dimensions of the bearing shall be checked. l h o types of dimensions, standard and critical, shall be measured. For each component type, the standard and critical dimensions are defined in the appropriate Article 18.7.3. The values of the critical dimensions shall be recorded and provided by the manufacturer to the Engineer. Failure of a critical dimension to satisfy its tolerance shall constitute absolute cause for rejection. Failure of a standard measurement to satisfy its tolerance shall, at the discretion of the Engineer, constitute cause for rejection.
18.7.2.2 Flatness shall be checked by placing a precision straightedge on the surface to be checked and by inserting feeler gages between the two.
The straight-edge shall be placed at different orientations and the worst condition shall be established.
No more than three feeler gages may be stacked on top of one another. The straight-edge shall be as long as the largest dimension of the flat surface. Flatness shall satisfy the requirements of Table 18.5.1.5-1. 18.7.2.4 Clearance Test In a clearance check the components of the bearing shall be moved through their design displacements or rotations in order to ve* that the required clearances exist. If the test is conducted on a rotational component which is not under simultaneous full vertical load, allowance shall be made for the displacements which would be caused by that load. 18.7.2.5 Short-term Compression Proof Load Test The bearing shall be loaded in compression to 150% of its rated service load. If a rotational element exists, a tapered plate shall be introduced in the load train so that the bearing sustains the load at the maximum simultaneous design rotation. The load shall be held for 5 minutes, removed, then reapplied for a second period of 5 minutes. The bearing shall be examined visually while under the second loading. Any defects shall constitute cause for rejection. If the load drops below the required value during either application, the test shall be restarted from the beginning. 18.7.2.6 Long-term Compression Proof Load Test The test shall be conducted in the same way as the short-term proof load test except that the second load shall be maintained for 15 hours. If the load drops below 90% of its target value during this time, the load shall be increased to the target value and the test duration shall be increased by the time for which the load was below the required value.
18.7.2.7 Bearing Friction Test (for sliding surfaces only) The purpose of the Bearing Friction Test is to verify that the friction values achieved in the material friction tests are adequate predictors of the friction in the finished bearing.
18.7.2.7 DIVISION II-CONSTRUCTION No lubrication shall be applied except that used for the whole lot of bearings. The bearing shall be loaded in compression with 100% of the full service dead plus live load, which shall be held constant for one hour prior to and throughout the duration of the sliding test. At least 12 cycles of sliding, each consisting of the smaller of the design displacement and ? 1 inch of movement, shall then be applied. The average sliding speed shall be 2.5 incheslminute. When the test is applied to curved sliding bearings, the design rotation shall be used in place of the displacement. In flat sliding bearings, the breakaway friction coefficient shall be computed for each direction of each cycle, and its mean and standard deviation shall be computed for the sixth through twelfth cycles. Neither the friction coefficient for the first movement nor the mean plus two standard deviations for the sixth through twelfth cycles shall exceed the value used in design, and the mean value for the sixth through the twelfth cycles shall not exceed two thirds of the value used in design. In curved sliding surfaces, the moment corresponding to the design rotation shall be established at each peak movement (positive and negative) during the first and last six full cycles of testing.
The corresponding load eccentricity shall be calculated by dividing the moment by the total compressive load acting. The eccentricity shall be small enough that the allowable stresses on the PTFE used in design are not violated. 18.7.2.8 Long-term Deterioration Test The purpose of the test is to venfy the long-term resistance of the materials to creep, wear and deterioration. The test shall be conducted on samples of the materials used in the bearings, or, at the option of the manufacturer, it may be conducted on a pair of bearings, placed back-toback. The samples shall have an area not less than 7 in2. The test piece shall first be loaded in compression to a stress corresponding to 100% of the maximum dead plus live service load. Flat sliding systems shall then be displaced through at least 1000 cycles with an amplitude of at least ? 1 inch (2 inches peak to peak). Curved sliding systems and rotational systems that depend on deformation of an elastomeric element shall be subjected to displacements corresponding to 5000 cycles of rotation at the design amplitude. The sliding may take place at up to 10 incheslminute, except when readings are taken of the coefficient of friction, when the sliding speed shall be 2.5 incheslminute. The following shall be cause for rejection of the bearing: _+ (1) Damage visible to the naked eye on disassembly of the bearing, such as excessive wear, cracks or splits in the material. 629 (2) A coefficient of friction which exceeds two thirds the value used in design. 18.7.2.9 Bearing Horizontal Force Capacity (Fixed or Guided Bearings Only) The purpose of the test is to verify that the bearing is stable and that the guide or restraint system has adequate strength under the most severe combination of horizontal and vertical loads. One or more loading combinations, consisting of a horizontal and vertical service load which could exist simultaneously in the structure, shall be selected. The vertical load shall be applied first, at 1.0 times its nominal value. The horizontal load shall be applied in stages, up to 1.5 times its nominal value. Failure or excessive deflection of any of the components shall be cause for rejection. 18.7.3 Performance Criteria If one bearing of the sample fails, all the bearings of that lot shall be rejected, unless the manufacturer elects to test each bearing of the lot at own expense.
In lieu of this procedure, the Engineer may require every bearing of the lot to be tested. 18.7.4 Special Testing Requirements 18.7.4.1 Special Test Requirements for Rocker and Roller Bearings Material certification tests shall be performed to establish the material properties of the steel. 18.7.4.2 Special Test Requirementsfor PTFE Sliding Bearings Inspection of the completed bearings or representative samples of bearings with PTFE surfaces in the manufacturer's plant may be required by the Engineer. Inspectors, if appointed, shall be allowed free access to the necessary parts of the manufacturer's plant and test facility. When testing is performed by the manufacturer, copies of the test results shall be submitted to the Engineer. The manufacturer is required to perform material tests on the materials used in the sliding surface in accordance with Article 18.7.2.2. A minimum of one test must be performed for each lot of bearings.
If requested by the Engineer and available test facilities permit, complete bearings shall be tested for complete bearing friction as defined in Article 18.7.2.7. If the test facility does not permit testing complete bearings, at the direction 630 HIGHWAY BRIDGES of the Engineer, extra bearings may be manufactured by the Contractor and samples of at least 100-kips capacity at normal working stresses prepared by sectioning the bearings. As soon as all bearings have been manufactured for a given project, notification shall be given to the Engineer who will select the prescribed test bearings at random from the lot. Manufacturer's certification of the steel, elastomeric pads, preformed fabric pads, PTFE, and other materials used in the construction of the bearings shall be furnished along with notification of fabrication completion. 18.7.4.3 Special Test Requirements for Curved Sliding Bearings Curved PTFE sliding surfaces shall satisfy all of the test requirements specified for PTF% sliding surfaces in Article 18.7.4.2, except that, when the prototype bearing is too large to test, a test bearing may be especially manufactured using materials and fabrication methods that are identical to those used for the prototype, in lieu of sectioning a bearing. Critical dimensions shall include the difference between the average radii of the two elements and the variation of the actual curved surface from the average one. The Engineer may require verification of these critical dimensions through a dimensional check as described in Article 18.7.2.3. 18.7.4.4 Special Test Requirements for Pot Bearings 18.7.4.4.1 Material Certijication Tests The manufacturer shall select, at random, samples for material certification tests as defined in Article 18.7.2.1. The tests shall be performed, and certifications shall be delivered to the Engineer. Certification shall be provided for all elastomeric elements. Their material properties shall satisfy the requirements of the design documents and the tests described in Article 18.7.4.5. Additional tests may be required by the Engineer. 18.7.4.4.2 Testing by the Engineer When quality assurance testing is called for by the special provisions, the manufacturer shall furnish to the Engineer the required number of complete bearings and component samples to perform quality assurance testing. At least one elastomeric element shall be tested per lot of bearings. All exterior surfaces of sampled production bearings shall be smooth and free from irregharities or protrusions that might interfere with testing procedures. 18.7.4.3 For quality assurance testing, the Engineer may select at random the required sample bearing(s) and the material samples from completed lots of bearings or from stock. A minimum of 30 days shall be allowed for inspection, sampling, and quality assurance testing of production bearings and component materials. 18.7.4.4.3 Bearing Tests Critical dimensions shall include the clearance between the piston and pot, and shall be verified by the Clearance Test described in Article 18.7.2.4. A Long-term Deterioration Test as described in Article 18.7.2.8 shall be performed on one bearing of each lot of pot bearings with sealing rings other than rings with rectangular cross-sections satisfying Article 14.6.4.5.1 and circular cross-sections satisfying Article 14.6.4.5.2. The test shall be performed at the maximum design rotation combined with maximum dead plus live load. If size limitations prevent testing of the full size bearing, a special bearing with the same sealing rings, the same rotational capacity and no less than 200 kips compressive load capacity may be tested in its place. A Long-term Compression Proof Load Test as described in Article 18.7.2.6 may be required by the Engineer. 18.7.4.5 Test Requirements for Elastomeric Bearings 18.7.4.5.1 Scope Materials for elastomeric bearings and the finished bearings themselves shall be subjected to the tests described in this section.
Material tests shall be in accordance with the appropriate Table 18.4.5.1-1A or Table 18.4.5.1-1B. 18.7.4.5.2 Frequency of Testing The ambient temperature tests on the elastomer described in Article 18.7.4.5.3 shall be conducted for the materials used in each lot of bearings. In lieu of performing a shear modulus test for each batch of material, the manufacturer may elect to provide certificates from tests performed on identical formulations within the preceding year, unless otherwise specified by the Engineer. Test certificates from the supplier shall be provided for each lot of reinforcement. The three low temperature tests on the elastomer described in Article 18.7.4.5.4 shall be conducted on the material used in each lot of bearings for grades 3,4, and 5 material and the instantaneous thermal stiffening test shall be conducted on material of grades 0 and 2. Low temperature brittleness and crystallization tests are not required 18.7.4.5.2 DIVISION 1 1 4 for grades 0 and 2 materials, unless especially requested by the Engineer. For grade 3 material, in lieu of the low temperature crystallization test, the manufacturer may choose to provide certificates from low-temperature crystallization tests performed on identical material within the last year, unless otherwise specified by the Engineer. Every finished bearing shall be visually inspected in accordance with Article 18.7.4.5.5. Every steel reinforced bearing shall be subjected to the short-term load test described in Article 18.7.4.5.6. From each lot of bearings either designed by the provisions of Article 14.6.5 of Division I of this specification or made from grade 4 or grade 5 elastomer, a random sample shall be subjected to the long-term load test described in Articles 18.7.2.7 and 18.7.4.5.7. The sample shall consist of at least one bearing chosen randomly from each size and material batch and shall comprise at least 10% of the lot. If one bearing of the sample fails, all the bearings of that lot shall be rejected, unless the manufacturer elects to test each bearing of the lot at own expense. In lieu of this procedure, the Engineer may require every bearing of the lot to be tested. The Engineer may require shear stiffness tests on material from a random sample of the finished bearings in accordance with Article 18.7.4.5.8. 18.7.4.5.3 Ambient Temperature Tests on the Elastomer The elastomer used shall at least satisfy the limits prescribed in the appropriate Table 18.4.5.1-1A or -lB for durometer hardness, tensile strength, ultimate elongation, heat resistance, compression set, and ozone resistance.
The bond to the reinforcement, if any, shall also satisfy Article 18.4.5.3. The shear modulus of the material shall be tested at 73OF 2 2°F using the apparatus and procedure described in Annex A of ASTM D 4014, amended where necessary by the requirements of Table 18.4.5.1-1A or -lB. It shall fall within 15% of the specified value, or within the range of its hardness given in Article 14.6.5.2 of Division I if no shear modulus is specified. 18.7.4.5.4 Low TemperatureTests on'the Elastomer The tests shall be performed in accordance with the requirements of Tables 18.4.5.1-1A and -lB and the compound shall satisfy all limits for its grade. The testing frequency shall be in accordance with Article 18.7.4.5.2. 18.7.4.5.5 Visual Inspection of the Finished Bearing Each finished bearing shall be inspected for compliance with dimensional tolerances and for overall quality of manufacture. In steel reinforced bearings, the edges of the steel shall be protected everywhere from corrosion. 18.7.4.5.6 Short-Duration Compression Tests on Bearings Each finished bearing shall be subjected to a short-term compression test as described in Article 18.7.2.5. If the bulging pattern suggests laminate parallelism or a layer thickness that is outside the specified tolerances, or poor laminate bond, the bearing shall be rejected. If there are three or more separate surface cracks that are greater than 0.08 inches wide and 0.08 inches deep, the bearing shall be rejected. 18.7.4.5.7 Long-Duration Compression Tests on Bearings The bearing shall be subject to a long-term compression test as described in Article 18.7.2.6. The bearing shall be examined visually at the end of the test while it is still under load. If the bulging pattern suggests laminate parallelism or a layer thickness that is outside the specified tolerances, or poor laminate bond, the bearing shall be rejected. If there are three or more separate surface cracks that are greater than 0.08 inches wide and 0.08 inches deep, the bearing shall be rejected.
18.7.4.5.8 Shear Modulus Tests on Materials from Bearings The shear modulus of the material in the finished bearing shall be evaluated by testing a specimen cut from it using the apparatus and procedure described in Annex A of ASTM D 1014, amended where necessary by the requirements of Table 18.4.5.1-1A or -1B, or, at the discretion of the Engineer, a comparable nondestructive stiffness test may be conducted on a pair of finished bearings.
The shear modulus shall fall within 15% of the specified value, or within the range for its hardness given in Table 14.6.5.2.1 of Division I if no shear modulus is specified. If the test is conducted on finished bearings, the material shear modulus shall be computed from the measured shear stiffness of the bearings, taking due account of the influence on shear stiffness of bearing geometry and compressive load. 18.7.4.7 Test Requirements for Bronze and Copper Alloy Bearings Material certification tests for the bronze or copper shall be performed to verify the properties of the metal. Bearing friction tests as defined in Article 18.7.2.7 or material friction tests as defined in Article 18.7.2.2 may be required by the Engineer. 632 HIGHWAY BRIDGES 18.7.4.8.2 18.7.4.8 Test Requirements for Disc Bearings 18.7.6 Use of Tested Bearings in the Structure 18.7.4.8.1 Material Certijicatiori Tests Bearings which have been satisfactorily tested in accordance with the requirements of this section may be used in the structure provided that they are equipped with new deformable elements, sliding elements and seals, as required by the Engineer. The manufacturer shall select, at random, samples for material certification tests as defined in Article 18.7.2.1. The tests shall be performed, and certifications shall be delivered to the Engineer. Certification shall be provided for all polyether urethane elements. Their material properties shall satisfy the requirements of the design documents and the tests described in Article 18.4.8.1. Additional tests may be required by the Engineer. 18.7.4.8.2 Testing by the Engineer When quality assurance testing is called for by the special provisions, the manufacturer shall furnish to the Engineer the required number of complete bearings and component samples to perform quality assurance testing. At least one set of material property tests in accordance with Article 18.4.8.1 shall be conducted per lot of bearings. All exterior surfaces of sampled production bearings shall be smooth and free from irregularities or protrusions that might interfere with testing procedures. For quality assurance testing, the Engineer may select at random the required sample bearing(s) and the material samples from completed lots of bearings. Aminimum of 30 days shall be allowed for inspection, sampling, and quality assurance testing of production bearings and component materials. 18.7.4.8.3 Bearing Tests Critical dimensionsshall include the clearance between the upper and lower parts of the steel housing, and shall be verified by the Clearance Test described in Article 18.7.2.4. A Long-term Deterioration Test as described in Article 18.7.2.8 shall be performed on one disc bearing of each lot.
The test shall be performed at the maximum design rotation combined with a maximum dead plus live load.
If size limitations prevent testing of the full size bearing, a special bearing with the same rotational capacity and no less than 200 kips compressive load capacity may be tested in its place. A Long-term Compression Proof Load Test as described in Article 18.7.2.6 may be required by the Engineer. 18.7.5 Cost of Tkansporting The Contractorshall assume the cost of transporting all samples from the place of manufacture to the test site and back, or if applicable, to the project site. 18.8 PACKING, SHIPPING AND STORING For transportation and storage, bearings shall be packaged in a way that prevents relative movement of their components and damage by handling, weather, dust, or other normal hazards. They shall be stored only in a clean, protected environment. When installed, bearings shall be clean and free from all foreign substances. Bearings shall not be opened or dismantled at the site except under the direct supervision of, or with the written approval of, the manufacturer or its assigned agents. 18.9 INSTALLATION 18.9.1 General Installation Requirements Bearings shall be installed by qualified personnel at the locations shown on the plans. Bearings shall be set to the dimensions and offsets prescribed by the manufacturer, the Engineer, and the plans and shall be adjusted as necessary to take into account the temperature and future movements of the bridge due to temperature changes, release of falsework, shortening due to prestressing and other bridge - movements. Each bridge bearing shall be located within Ir: '/a inch of its correct position in the horizontal plane and oriented to within an angular tolerance of 0.02 radians. Guided Bearings shall also satisfy the requirements of Article 18.9.2.3.All bearings except those which are placed in opposing pairs shall be set horizontal to within an angular tolerance of 0.005 radians, and must have full and even contact with load plates, where these exist. The superstructure supported by the bearing shall be set on it so that, under full dead load, its slope lies within an angular tolerance of 0.005 radians of the design value. Any departure from this tolerance shall be corrected by means of a tapered plate or by other means approved by the Engineer. If shim stacks are needed to level the bearing they shall be removed after grouting and before the weight of the superstructure acts on the bearing. Metallic bearing assemblies not embedded in the concrete shall be bedded on the concrete with a filler or fabric material conforming to Article 18.4.9. Bearings seated directly on steel work require the supporting surface to be machined so as to provide a level and planar surface upon which the bearing is placed. 18.9.2.2 DIVISION II-CONSTRUCTION Bearings or masonry plates that rest on a steel support may be installed directly on it, provided that it is flat to within a tolerance of 0.002 times the nominal dimension, and is sufficiently rigid that it will not deform under the specified loads to exceed that flatness tolerance. 18.9.2 Special Installation Requirements 18.9.2.1 Installation of Rocker and Roller Bearings Just before placing roller bearings, the Contractor shall coat all contact surfaces thoroughly with oil and graphite. 18.9.2.2 Installation of Elastomeric Bearings Elastomeric bearings without external load plates may be placed directly on a concrete or steel surface provided that it is flat to within an tolerance of 0.005 of the nominal dimension for steel reinforced bearings and 0.01 of the nominal dimension for others. Bearings shall be placed on surfaces that are horizontal to within 0.01 radians. Any lack of parallelism between the top of the bearing and the underside of the girder that exceeds 0.01 radians shall be corrected by grouting or as otherwisedirected by the Engineer. Exterior plates of the bearing shall not be welded unless at least 1.5 inches of steel exists between the weld and the elastomer.
In no case shall the elastomer or the bond be subjected to temperatures higher than 400" E 18.9.2.3 Installation of Guideways and Restraints Guided bearings and bearings which rotate about only one axis shall be oriented in the direction specified on the contract plans to within an angular tolerance of 0.005 radians. 18.9.2.4 Installation of Anchorages Load plates shall be set level to within an angular tolerance of 0.005 radians and shall have a uniform bearing over their whole area. When plates are to be embedded in concrete, provision shall be made to keep the plates in the correct position while the concrete is being placed. A bedding layer may be used to achieve level, uniform bearing. This may consist of grout or a ductile metal such as a thin lead sheet.
The bedding material shall be able to support the specified vertical and horizontal loads without undergoing displacements or deformations detrimental to the bearing or structure. Anchor bolts embedded in concrete shall either be cast into the concrete or shall be grouted into drill holes.
633 18.10 DOCUMENTATION 18.10.1 Working Drawings The manufacturer shall submit to the Engineer shop drawings and design calculations which are sufficiently detailed to permit proper review of the bearings. The drawings shall show all details of the bearings and of the materials proposed for use and must be approved by the Engineer before fabrication of the bearings is begun. Such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work. The drawings shall include, but not be limited to the following information: (1) . . Plan, elevations and sections including all nominal dimensions and material designations. (2) Vertical and horizontal load capacities, horizontal movement capacities and rotation capacities about two horizontal and one vertical axes. (3) Design calculations for all items not completely covered in Section 14 of Division I of this specification.
(4) Material designations and specifications. (5) A schedule of bearing offsets, if any are required. (6) Shop painting or coating requirements. (7) Any special installation requirements. 18.10.2 Marking Each bearing shall be marked in indelible ink or flexible paint. The marking shall consist of the location, orientation, order number, lot number, bearing identification number, and elastomer type and grade number. Unless otherwise specified in the contract documents, the marking shall be on a face which is visible after erection of the bridge. 18.10.3 Certification The manufacturer shall supply certification data for all materials used. This shall consist of at least test reports for the bearing performance tests and for any forgings, castings or hardened material, mill certificates for all other steels used, and a certificate of compliance for the bearing as a whole and for any anchor bolts, dowels or other accessories. If the manufacturer designed the bearing, he shall cerhfy that each bearing satisfies the Engineer's requirements, given under Division I, Section 14, "Bearings." The manufacturer shall also supply a separate sheet showing the materials, critical dimensions and clearances for each bearing other than elastomeric pads. The precise information to be supplied shall be agreed between the Engineer and the manufacturer prior to starting production. 634 HIGHWAY BRIDGES 18.11 18.11 MEASUREMENT 18.12 PAYMENT Bearing devices will be measured either by the pound as determined from scale weights or by a unit basis for each type of bearing assembly listed in the schedule of bid items. Scale weights are not required when calculated weights are shown on the plans, in which case the weights shown on the plans will be used as the basis of payment. Bearing devices will be paid for at the contract price per pound or per unit. Such payment shall include full compensation for furnishing all labor, materials, tools, equipment and incidentals, and for doing all the work involved in furnishing, testing and installing said bearing devices, complete in place, as shown on the plans, and as specified in these Specifications and the special provisions, and as directed by the Engineer. Section 19 BRIDGE DECK JOINT SEALS 19.1 GENERAL This work shall consist of the furnishing and installing of joint sealing systems in bridge decks of the types used where significant movements are expected across the joint. These include compression seal joints consisting of preformed elastomeric material compressed and installed in specially prepared joints and joint seal assemblies consisting of assemblies of metal and elastomeric materials installed in recesses in the deck surface.
Joint seals described in the plans or the specifications as poured joint seals shall conform to the requirement of Article 8.9, "Expansion and Contraction Joints." The type and dimensions or movement rating for bridge deck joint seals at each location shall be as shown on the plans or ordered by the Engineer. All joint seals shall prevent the intrusion of material and water through the joint system. 19.2 WORKING DRAWINGS If not given on the plans, calculations showing the joint settings for their installation will be required before approval to install joints in any bridge deck can be given. The Contractor will submit working drawings to the Engineer showing the installation procedure and joint assembly for bridge decks using proprietary joint systems. Also, shop drawings shall be submitted to the Engineer for approval for joints having a total movement of more than 1%inches. Working drawings must be approved by the Engineer prior to performance of the work involved and such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work. 19.3 MATERIALS Bridge deck joint seal materials and assemblies shall conform to the following specifications: Preformed elastomeric joint seals of multiple web design shall conform to AASHTO M 220 (ASTM D 2628).
Lubricant-adhesivefor use with preformed elastomeric seals shall conform to ASTM D 4070. Deck joint seal assemblies shall be of an approved type for each size required and shall conform to the specifications provided by the manufacturer at the time of approval. Steel and fabricated steel components shall conform to the requirements of Section 23, "Miscellaneous Metal." 19.4 MANUFACTURE AND FABRICATION 19.4.1 Compression Seal Joints Preformed elastomeric joint seals shall not be field spliced except when specifically permitted by the Engineer. 19.4.2 Joint Seal Assemblies Expansion joint assemblies shall be fabricated by the manufacturer and delivered to the bridge site completely assembled, unless otherwise shown on the plans or specified in the special provisions. 19.5 INSTALLATION 19.5.1 General All joint materials and assemblies, when stored at the job site, shall be protected from damage and assemblies shall be supported so as to maintain their true shape and alignment. Deck joint seals shall be constructed and installed to provide a smooth ride. Bridge deck joints shall be covered over by protective material after installation until final cleanup of the bridge deck. After installation and prior to final acceptance, deck joint seals shall be tested in the presence of the Engineer for leakage of water through the joint. Any leakage of the joint seal will be cause for rejection. 636 HIGHWAY BRIDGES 19.5.2 Compression Seal Joints Joints in the roadway area of bridge decks which are to be sealed with compression seals shall be cast to a narrower width than required for the preformed material. Such joints in curbs and sidewalks may be cast to full width. Prior to installation of compression seals in joints whose width is narrower than needed, a groove of proper width and depth to receive the preformed material shall be saw cut along the top of the joint. When making saw cuts into the bridge deck, spalling shall be minimized. Both sides of a groove shall be cut simultaneously to the proper depth and alignment as shown on the plans. The alignment of the saw shall be controlled at all times by a rigid guide. The width of the groove will depend on the temperature and age of the concrete and shall be as directed by the Engineer. Lip of saw cut should be bevelled to avoid later breakage. After saw cutting, any spalls, popouts or cracks shall be repaired prior to installation of the lubricant sealant. Saw cuts are not required where armor plates are used. At the time of installation the joint shall be clean and dry and free from spalls and irregularities which might impair a proper joint seal. Concrete or metal surfaces shall be clean, free of rust, laitance, oils, dirt, dust, or other deleterious materials. Premolded elastomeric compression joint seals shall be installed without damage to the seal by suitable hand methods or machine tools. The lubricant-adhesiveshall be applied to both faces of the joint prior to installation and in accordance with the manufacturer's instructions. The preformed elastomeric seal shall 19.5.2 be compressed to the thickness specified on the plans or as approved by the Engineer for the rated opening and ambient temperature at the time of installation. Loose fitting or open points between the seal and the deck will not be permitted. 19.5.3 Joint Seal Assemblies Expansion joint seal assemblies shall be constructed to provide absolute freedom of movement through a range consistent with that prescribed by the Engineer or as shown on the design plans. Installation shall be in accordance with the manufacturer's recommendations. Final settings of the deck joint seal assembly at the time of casting in the anchorages of the unit depend on the relationship of the current temperature of the superstructure to its expected mean temperature, and shall be as specified by the manufacturer or Engineer or as shown on the plans. 19.6 MEASUREMENT AND PAYMENT Deck joint seals will be measured by the linear foot of acceptable joint seal completely installed by measurements made along the slope of the centerline of the joint seal. Payment of linear feet of joint seal as measured, for each type of seal for which separate payment is provided, shall include full compensation for the cost of labor, equipment and materials to furnish and install the deck joint seal. Section 20 RAILINGS 20.1 GENERAL 20.1.1 Description This work consists of furnishing all materials and constructing railings on structures. The types of railings included in this work consist of handrailings, pedestrian railings, traffic railings which are sometimes called bamers, and railings for other such purposes. Railings constructed at each location shall conform to the type and details shown on the plans for that location.
The work includes the furnishing and placing of mortar or concrete, anchor bolts, reinforcing steel dowels or other devices used to attach the railing to the structure.
20.1.2 Materials All materials not otherwise specified shall conform to the requirements of the applicable AASHTO Standard Specijications for Transportation Materials. Structures," except that formed sections may be fabricated from mild steel, and pipe sections shall be of standard steel pipe. Nuts and bolts not designated as high strength shall conform to the requirements of ASTM A 307 and steel tubing shall conform to the requirements of ASTM A 500, Grade B. 20.2.1.2 Aluminum Railing For aluminum railings or portions of railings, cast alu- minum posts shall conform to the requirements of AASHTO M 193; and extruded components shall conform to the requirements of ASTM B 221. 20.2.1.3 Metal Beam Railing Metal beam rail, posts and hardware shall conform to the requirements in Section 606 of the AASHTO Guide Specijications for Highway Construction. 20.1.3 Construction 20.2.1.4 Welding Unless otherwise permitted by the Engineer, railing shall not be placed until the centering or falsework for the span has been released, rendering the span self-supporting.
20.1.4 L i e and Grade The line and grade of the railing shall be true to that shown on the plans and may include an allowance for camber in each span but shall not follow any unevenness in the superstructure. Unless otherwise specified or shown on the plans, railings on bridges, whether super-elevated or not, shall be vertical. 20.2 METAL RAILING 20.2.1 Materials and Fabrication 20.2.1.1 Steel Railing Materials and fabrication of steel railings shall conform to the applicable requirements of Section 11, "Steel All exposed welds shall be finished by grinding or filing to give a smooth surface. Welding of aluminum materials shall be done by an inert gas shielded, electric arc welding process using no welding flux. Torch or flame cutting of aluminum will not be permitted. 20.2.2 Installation Metal railings shall be carefully adjusted prior to fixing in place to ensure proper matching at abutting joints, correct alignment, and camber throughout their length. Holes for field connections shall be drilled with the railing in place on the structure at proper grade and alignment. Where aluminum alloys come in contact with other metals or concrete, the contacting surfaces shall be thoroughly coated with a dielectric aluminum-impregnated caulking compound, or a synthetic rubber gasket may be placed between the two surfaces. 638 HIGHWAY BFUDGES 20.2.3 Finish Unless otherwise specified, anchor bolts, nuts and all steel portions of railings shall be galvanized and aluminum portions shall be unpainted.
Galvanizing of rail element shall conform to the requirements of AASHTO M 111 (ASTM A 123) and galvanizing of nuts and bolts shall conform to the requirements of AASHTO M 232 (ASTM A 153). Minor abrasions to galvanized surfaces shall be repaired with zinc rich paint. After erection, all sharp protrusions shall be removed and the railing cleaned of discoloring foreign materials. When painting is specified, the type and coating shall conform to the requirements of Section 13, "Painting," or the special provisions. 20.3 CONCRETE RAILING 20.3.1 Materials and Construction Concrete railings, depending on the design, may be constructed by the cast-in-place, precast or, when approved by the Engineer, the slip form method. All materials and construction shall conform to the requirements in Section 8, "Concrete Structures" and Section 9, "Reinforcing Steel." Unless otherwise specified, concrete shall conform to Class AE except that Class A may be used in areas where freezing seldom occurs. When the minimum thickness of the railing at any point is less than 4 inches, Class C (AE) or, where freezing seldom occurs, Class C concrete may be used. Forms for cast-inplace railing shall not be removed until adequate measures to protect and cure the concrete are in place and the concrete has sufficient strength to prevent surface or other damage caused by form removal. Finish for railings constructed with fixed forms shall be Class 2-Rubbed Finish. Finish for railings constructed with slip forms and for temporary railings shall be Class I-Ordinary Finish. 20.4 TIMBER RAILING Unless otherwise stated in the special provisions, posts, rails, and other timber for wood railings shall be constructed according to the requirements of Section 16, "Timber Structures," and Section 17, "PreservativeTreatment of Wood." When treated wood is called for, the preservative treatment shall conform to the requirements of Section 17, "Preventive Treatment of Wood." The sur- 20.2.3 faces of all elements of treated wooden railings that are located where contact with people could occur shall be sealed with two coats of an acceptable sealer.
Acceptable sealers are urethane, shellac, latex epoxy, enamel and varnish. 20.5 STONE AND BRICK RAILINGS Stone and brick railings shall conform to the requirements of Section 14, "Stone Masonry," and Section 15, "Concrete Block and Brick Masonry." 20.6 TEMPORARY RAILING Temporary railings shall be constructed of materials and to the details shown on the plans or specified. Railings shall be properly joined and aligned at the required locations.
Temporary precast barriers shall be installed on a solid base. The temporary railing shall be maintained in first class condition and shall not be removed until all work requiring the railing has been completed. Previously used units may be employed provided they are in a clean and undamaged condition. After removal, temporary railing shall continue to be the property of the Contractor. 20.7 MEASUREMENTAND PAYMENT 20.7.1 Measurement Railings will be measured by the linear foot between the ends of the railing or the outside ends of end posts, whichever is greater. Measurement will be made along the slope of the railing and no deductions will be made for electrolier or other small openings called for on the plans.
20.7.2 Payment Railings will be paid for by the contract prices per linear foot for the various types listed in the schedule of bid items. Such payment shall include full compensation for furnishing all labor, materials, equipment and incidentals and for doing all work involved in constructing the railings or barriers complete in place, including the furnishing and installation of reinforcing steel and steel dowels or anchor bolts which are either placed or drilled and bonded into the structure for attachment of the railing. Section 21 WATERPROOFING 21.1 GENERAL This work shall consist of furnishing and installing materials to waterproof or dampproof concrete or masonry surfaces.
The surfaces to be waterproofed or dampproofed and the type of system to be installed shall be as shown on the plans or otherwise specified. stances for Use in Waterproofing, AASHTO M 117 (ASTM D 173) or the Specifications for Woven Glass Fabric Treated with Asphalt, ASTM D 1663. The Fabric shall be stored in a dry,protected place. The rolls shall not be stored on end. 21.2.2 Preformed Membrane Waterproofing 21.1.1 Waterproofing Waterproofing shall consist of either a constructed-inplace asphalt membrane system or a preformed membrane system, both of which include appropriate priming materials and, when required, protective coverings. Unless a specific type of waterproofing system is shown on the plans or specified, the type of system to be used will be at the option of the Contractor. 21.1.2 Dampproofing Dampproofing shall consist of a coating of primer and two moppings of waterproofing asphalt. 21.2 MATERIALS 21.2.1 Asphalt Membrane Waterproofing System 21.2.1.1 Asphalt Systems 21.2.2.1 Primer Primer for use with the rubberized asphalt membrane shall be a neoprene based material, and the primer for use with the modified bitumen membrane shall be a resin or based material. Primers be a type recornmended by the manufacturer.
21.2.2.2 Preformed Membrane Sheet Preformed membrane sheet shall be of either the rubberized asphalt type or the modified bitumen type. The rubberized asphalt type shall consist of a rubberized asphalt sheet reinforced with a polyethylene film or mesh. The modified bitumen sheet &pi shall consist of a polymer modified bitumen sheet reinforced with a stitchbonded polyester fabric or a fiberglass mesh. The membrane sheet shall conform to the following requirements: Waterproofing asphalt shall conform to the Specification for Asphalt for Dampproofing and Waterproofing, AASHTO M 115 (ASTM D 312). Type I shall be used below ground and Q p e II used above ground. 21.2.1.2 Primer Primer for use with waterproofing asphalt shall conform to the Specification for Primer for Use With Asphalt in Dampproofing and Waterproofing, AASHTO M 116 (ASTM D 41). 21.2.1.3 Fabric The fabric shall conform to either the Specification for Woven Cotton Fabrics Saturated with Bituminous Sub- For Surfaces Other Than Bridge Decks RS~ property Strength ' % Elongation at value Rubberized Modifii BituAsphalt 5 p e menVpe ASTM D 8822 20 lbslin? ASTM D 882' 150%3 20 l b s ~ i n . ~ ASTM D 146' No cracks No cracks 60 mils 60 mils 165°F 210°F 25%3 B"alP Pliability Thickness (Min.) Softening Point (Min.) AASHTO T 53 (continued on next page) HIGHWAY BRIDGES 640 For Bridge Deck Surfaces Prooertv ~~~t ' Tensile Strength % Elongation at BreaK' Pliability Thickness (Min.) Softening Point Win.) 21.2.5 Inspection and Delivery Value Modified Rubberized Asphalt Qpe Bitumen ASTM D 8822 50 lbdin! 21.2.2.2 ASTM D 882' 40 l b ~ / i n . ~ 10%~ ASTM D 146S No cracks 65 mils AASHTO T 53 165°F No cracks 70 mils 210°F ' Breaking Factor in machine direction. Method A, 1" wide strip with 4"minimum initial grip separation and 4"gage length at 2" per minute. Average 5 samples. "73.4"F 2 3.6"F Machine D i t i o n 180' Bend over a 1-inch mandrel at 10°E 21.2.2.3 Mastic The mastic for use with preformed rubberized sheets shall be a rubberized asphalt cold applied joint sealant. The mastic for use with modified bitumen sheet shall be a blend of bituminous and synthetic resins. 21.2.3 Protective Covers Materials for protective covers shall conform to the following unless another type is shown or specified. For surfaces against which backfill will be placed, the protective cover shall consist of %-inchhardboard or other material that will furnish equivalent protection from damage due to sharp coarse backfill material or from construction equipment. For roadway surfaces of bridge decks, the protective cover shall consist of a layer of special asphalt concrete as specified in the special provisions. For horizontal surfaces above which reinforced concrete structures are to be constructed, the protective cover shall consist of a 2-inch course of concrete mortar conforming to the requirements of Article 8.14 except that the proportions shall consist of 1 part Portland cement to 3 parts of fine aggregate.
This mortar course shall be reinforced midway between its top and bottom surfaces with 6 X 6-W1.4 X W1.4 welded wire fabric, or its equivalent. The top surface shall be finished smooth and true to grade. 21.2.4 Dampproofing The primer and asphalt used for dampproofing shall conform to that specified in Article 21.2.1. All waterproofingand dampproofing materials shall be tested before shipment. Unless otherwise ordered by the Engineer, they shall be tested at the place of manufacture, and, when so tested, a copy of the test results shall be sent to the Engineer by the chemist or inspection bureau which has been designated to make the tests, and each package shall have affixed to it a label, seal, or other mark of identification, showing that it has been tested and found acceptable, and identifying the package with the laboratory tests. Factory inspection is preferred, but, in lieu thereof, the Engineer may order that representative samples, properly identified, be sent to him or her for test prior to shipment of the materials. After delivery

Steel." 22.3.10 Geocomposite Drain Geocomposite drain shall consist of a manufactured core with one or both sides covered with a layer of filter fabric. 22.3.4 Slopes shall be shaped to allow the full thickness of the specified slope protection and any bedding or filter gravel, where required. Slopes shall not be steeper than the natural angle of repose of the slope specified in the contract. Where the slopes cannot be excavated to undisturbed material, the underlying material shall be compacted to 95% standard density per AASHTO T 99. 22.4.2 Bedding When called for on the plans, a layer of filter gravel or filter fabric shall be placed on the slope immediately prior The layer of filto placement of the riprap or slope ter gavel shall be shaped to provide the minimum thickness specified. 22.4.3 Filter Fabric When specified in the contract, filter fabric shall be spread uniformly over the prepared slope or surface. The fabric shall be unrolled directly on the surface to the lines and dimensions shown.
The filter fabric shall be lapped a minimum of 12 inches in each direction and shall be anchored in position with approved anchoring devices. The Contractor shall place the riprap in a manner that will not tear, puncture, or shift the fabric. Tracked or wheeled equipment will not be permitted on the fabric covered slopes. 22.4.4 DIVISION 11--CONSTRUCTION 22.4.4 Geocomposite Drain Geocomposite drains shall be installed at locations shown on the plans, described in the special provisions, and where directed by the Engineer. Collection and discharge systems shall be installed as shown on the plans or as directed by the Engineer. Core material manufactured from impermeable plastic sheeting having connecting corrugations shall be placed with the corrugations approximately perpendicular to the drainage collection system. When only one side of the geocomposite drain is covered with filter fabric, the drain shall be installed with the filter fabric side facing the embankment. The fabric facing the embankment side shall overlap a minimum of 3 inches at all joints and wrap around the exterior edges a minimum of 3 inches beyond the exterior edge. If additional fabric is needed to provide overlap at joints and wrap-around at edges, the added fabric shall overlap the fabric on the geocomposite drain at least 6 inches and be attached thereto. Should the fabric on the geocomposite drain be tom or punctured, the damaged section shall be replaced completely or repaired by placing a piece of fabric that is large enough to cover the damaged area and provide a 6-inch overlap all around the damaged area. 22.4.5 Hand Placing Stones Where hand placing of stones is specified, the larger stones shall be placed first with close joints. The larger stones shall be placed in the footing trench. Stones shall be placed with their longitudinal axis normal to the embankment face and arranged so that each stone above the foundation course has a three-point bearing on the underlying stones. The foundation course is the course placed on the slope in contact with the ground surface. Bearing on smaller stones that may be used for chinking voids will not be acceptable. Placing of stones by dumping will not be permitted. Interstices shall be filled with smaller stones and spalls. 22.4.6 Machine-Placed Stones 22.4.6.1 Dry Placement Machine-placed stones shall be so placed so as to provide a minimum of voids, and the larger stones shall be placed in the toe course and on the outside surface of the slope protection. The stone may be placed by dumping and may be spread in layers by bulldozers or other suitable equipment. At the completion of slope protection 647 work, the footing trench shall be filled with excavated material, and compaction will not be required. 22.4.6.2 Underwater Placement When placed under water, free dumping will not be permitted without written permission of the Engineer. Placement shall be by controlled methods using bottom dump buckets or wire rope baskets lowered through the water to the point of placement. 22.4.7 Wire-Enclosed Riprap (Gabions) 22.4.7.1 Fabrication The wire mesh shall be twisted to form hexagonal openings of uniform size. The maximum linear dimension of the mesh opening shall not exceed 4% inches and the area of the mesh opening shall not exceed 8 square inches. The mesh shall be fabricated in such a manner as to be nonravelling. Nonravelling is defined as the ability to resist pulling apart at any of the twists or connections forming the mesh when a single wire strand in a section is cut. Gabions shall be fabricated so the sides, ends, lid, and diaphragms can be assembled at the construction site into rectangular baskets of the specified size. Gabions shall be of single unit construction-base, lid, ends, and sides shall be either woven into a single unit or one edge of these members connected to the base section of the gabion in a manner that strength and flexibility at the point of connection is at least equal to that of the mesh. Where the length of the gabion exceeds its horizontal width, the gabion shall be equally divided by diaphragms of the same mesh and gauge as the body of the gabions, into cells the length of which does not exceed the horizontal width. The gabion shall be furnished with the necessary diaphragms secured in proper position on the base in a manner that no additional tying at this junction will be necessary.
All perimeter edges of the mesh forming the gabion shall be securely clip bound or selvedged so that the joints formed by tying the selvedges have at least the same strength as the body of the mesh. Selvedge wire used through all the edges (perimeter wire) shall not be less than 0.148-inch diameter (U.S. Wire Gage No. 9) and shall meet the same strength and coating specifications as the wire mesh.
Tie and connection wire shall be supplied in sufficient quantity to securely fasten all edges of the gabion and diaphragms and to provide for at least four cross connecting wires in each cell whose height is equal to the width and at least two cross-connecting wires in each cell whose height is one-half the width of the gabion. Cross-connect- 648 HIGHWAY BRIDGES ing wires will not be required when the height of the cell is one-third the width of the gabion. Tie and connection wire shall meet the same strength and coating specifications as the wire used in the mesh, except that it may be as much as two gages smaller. In lieu of tie wire, two gauge galvanized hog rings may be used to connect adjacent baskets and to secure basket lids. Spacing of the hog rings shall not exceed 6 inches. Vertical joints in the completed work shall be staggered at approximately % or YZthe length of the full baskets. 22.4.7.2 Installation The gabions shall be placed on a smooth foundation. Final line and grade shall be approved by the Engineer. Each gabion unit shall be assembled by binding together all vertical edges with wire ties on approximately 6-inch spacing or by a continuous piece of connecting wire stitched around the vertical edges with a coil about every 4 inches.
Empty gabion units shall be set to line and grade as shown on the plans or as directed by the Engineer. Wire ties, hog rings, or connecting wire shall be used to join the units together in the same manner as described above for assembling. Internal tie wires shall be uniformly spaced and securely fastened in each cell of the structure. A standard fence stretcher, chain fall, or iron rod may be used to stretch the wire baskets and hold alignment.
The gabions shall be filled with stone carefully placed by hand or machine to assure alignment and avoid bulges with a minimum of voids. Alternate placing of rock and connection wires shall be performed until the gabion is filled. After a gabion has been filled, the lid shall be bent over until it meets the sides and edges. The lid shall then be secured to the sides, ends and diaphragms with the wire ties or connecting wire in the manner described above for assembling. 22.4.8 Grouted Riprap Stones shall be placed on the slope as specified in Article 22.4.5 and shall be thoroughly moistened with water after placement. Grout shall be applied while the stone is moist and shall be worked into the interstices to completely fill the voids. Where the depth is in excess of 12 inches, the stone shall be placed in 12-inch lifts and each lift grouted prior to placement of the next lift. Succeeding lifts shall be constructed and grouted before grout in the previous lift has set. Grout shall be placed only when the weather is suitable and shall be protected from freezing for at least 4 days. The surface shall be cured by covering with moist earth, 22.4.7.1 wet rugs or curing blankets for at least 3 days after grout placement. Weep holes shall be provided through the riprap as shown on the plans or as directed by the Engineer. 22.4.9 Sacked Concrete Riprap Sacks shall be filled with approximately 1 cubic foot of concrete, leaving room at the top to fold the sacks and retain the concrete during placement. Immediately after being filled, the sacks shall be placed and lightly trampled to conform with the earth face and with adjacent sacks. The first two courses shall provide a foundation of double thickness. The first foundation course shall consist of a double row of stretchers (long dimension of sack parallel to contour of slope) laid level and adjacent to each other in a neatly trimmed trench.
The trench shall be located as shown on the plans, or as directed by the Engineer, cut to the proper depth and width to accommodate placement of the first two foundation courses, and cut back into the slope a sufficient distance to enable proper subsequent placement of the riprap. The second foundation course shall consist of a row of headers (long dimension at right angles to the stretchers) placed directly above the double row of stretchers.
The remaining courses shall consist of stretchers and shall be placed with staggered joints. Dirt and debris shall be removed from the top of the sacks before the next course is placed. Stretchers shall be placed so that the folded ends are not adjacent. Headers shall be placed with the folds toward the earth face.
Not more than four vertical courses of sacks shall be placed in any tier until initial set has taken place in the first course. When there will not be proper bearing or bond for the concrete because of delays in placing succeeding layers of sacks, a small trench shall be excavated back of the row of sacks and filled with fresh concrete before the next layer of sacks is laid. Header courses may be required at any level to provide additional stability.
Sacked concrete riprap shall be cured with a blanket of wet earth or by sprinkling with a fine spray of water every 2 hours during the daytime for 4 days. Weep holes shall be provided through the riprap as shown on the plans or as directed by the Engineer. 22.4.10 Concrete Slope Paving 22.4.10.1 General This work shall consist of constructing cast-in-place and precast portland cement concrete slope paving. At the option of the Contractor, the cast-in-place slope paving shall be constructed of either Portland cement concrete or 22.4.10.1 DIVISION 11--CONSTRUCTION pneumatically applied mortar. Where specified or permitted by the Engineer, this work shall also include woven fabric forms filled with fine aggregate Portland cement concrete grout. 22.4.10.2 Cast-in-Place Slope Paving Concrete shall be mixed and placed in conformance with the provisions in Section 8, "Concrete Structures," and shall be spread and tamped until it is thoroughly compacted and mortar flushes to the surface. If the slope is too steep to permit the use of concrete sufficiently wet to flush with tamping, the concrete shall be tamped until consolidated and a mortar surface Y4-inch thick, troweled on immediately. The mortar shall consist of one part Portland cement and three parts of fine aggregate. The mortar surface shall be considered as a part of the concrete and no separate payment will be made therefore. After striking off to grade, the concrete shall be hand floated with wooden floats.
The entire surface shall be broomed with a fine texture hair push broom to produce a uniform surface with the broom marks parallel to the edges of the panel. Edges and joints shall be edged with a Y4-inch radius edger prior to the brooming. Pneumatically applied mortar shall be placed and finished in accordance with the provisions in Section 24, "Pneumatically Applied Mortar." Expansion joints shall be installed transversely at intervals of 20 feet. Longitudinal expansion joints shall be installed at the locations shown on the plans. Expansion joints shall be filled with expansion joint filler %-inch thick. Cast-in-place concrete and pneumatically applied mortar shall be cured as provided in Sections 8 and 24, respectively.
Weep holes shall be provided through the slope paving as shown on the plans or as directed by the Engineer. When permitted or specified, the Contractor may use woven fabric forms filled with pumpable fine aggregate Portland cement concrete grout as the slope protection system.
The request by the Contractor to use a particular system must be in writing accompanied by working drawings and complete information as to the materials, cons&ction and performance characteristics of the proposed system.
Pervious backfill material, if required by the plans, shall be placed as shown. %o cubic feet of pervious backfill material wrapped in filter fabric shall biplaced at each weep hole and drain hole.
At the completion of the work, footing trenches shall be filled with excavated material and compaction will not be required. 649 22.4.10.3 Precast Slope Paving Precast slabs, blocks, and shapes shall be laid on a 3inch bed of cushion sand in the pattern shown on the plans. Blocks and shapes shall be thoroughly rammed in place to provide a uniformly even surface and solid bedding under each block or shape. In the areas where grouting is called for, the blocks shall be laid in running bond with the length parallel to the slope and with %-inch joints. Following the laying of the blocks, in the area to be grouted, sufficient mortar sand shall be spread over the surface and swept into the joints to fill the latter to 4 inches from the surface.
The blocks shall be wetted to the satisfaction of the Engineer before any grout is placed. The joints shall be filled with grout flush with the top of the block. After grouting has been completed and the grout has sufficiently hardened, the blocks shall be wetted, covered and cured with curing blankets or covers for the first 7 days after grouting. Grout shall not be poured during freezing weather. 22.5 MEASUREMENT AND PAYMENT 22.5.1 Method of Measurement 22.5.1.1 Stone Riprap and Filter Blanket Hand-placed riprap, machine-placed riprap, grouted riprap, and filter blanket aggregate will be measured by the square yard, cubic yard, or ton, as listed in the schedule of bid items. The area will be that actually placed to the limiting dimensions shown on the plans, or the plan dimensions as may have been revised by the Engineer, measured along the upper surface. If measured by the cubic yard, the volume will be computed on the basis of the measured area and the thickness specified on the plans. If measured by the ton, the quantity shall be the number of tons, loose measure, incorporated into the work. 22.5.1.2 Sacked Concrete Riprap Sacked concrete riprap will be measured by the cubic yard of concrete placed. Measurement will be based on mixer volumes. 22.5.1.3 Wire-Enclosed Riprap (Gabions) Wire-enclosed riprap (gabions) will be measured as the number of square yards of surface area. 650 HIGHWAY BRIDGES 22.5.1.4 Cast-in-Place Concrete Slope Paving Cast-in-place concrete or pneumatically applied mortar slope paving will be measured on a square yard or cubic yard basis. The area will be that actually placed to the limiting dimensions shown on the plans, or the plan dimensions as may have been revised by the Engineer, n~easuredalong the upper sloped surface. If measured by the cubic yard, the volume will be computed on the basis of the measured area and the thickness shown on the plans. No additional compensation will be allowed for additional concrete or pneumatically applied mortar placed by reason of low foundation. 22.5.1.5 Precast Concrete Slope Paving Precast concrete slabs, blocks, or shapes placed as slope paving will be measured in square yards computed from the payment lines shown on the plans, or as directed by the Engineer. 22.5.1.6 Filter Fabric Filter fabric will be n ~ a ~ u rby e dthe square yard on the ground surface, excluding overlaps, complete in place. 22.5.2 Payment 22.5.2.1 General Payment for slope protection of the various classes at the unit prices bid will include full compensation for all labor, materials, equipment, or other incidentals in connection with the preparation of subgrade (except for the furnishing and placement of filter blanket material and filter fabric), excavating and backfilling toe trenches where required, furnishing and placing the stones, slabs, blocks, shapes, grout, mortar, Portland cement concrete, pneumatically applied mortar, reinforcing steel, expansion joint filler, if required, and all other work and incidental material required to complete the work in accordance with the plans and specifications. 22.5.2.2 Stone Riprap Hand-placed riprap, machine-placed riprap, and grouted riprap measured in accordance with Article 22.5.1.1 will be paid for at the price bid per square yard, per cubic yard, or per ton as set forth in the schedule of bid items. 22.5.2.3 Sacked Concrete Riprap Sacked concrete riprap measured in accordance with Article 22.5.1.2 will be paid for at the price bid per cubic yard. 22.5.1.4 22.5.2.4 Wire-Enclosed Riprap (Gabions) Wire-enclosed riprap (gabions) measured in accordance with Article 22.5.1.3 will be paid for at the price bid per square yard. Such price shall include wire baskets, connection hardware, anchors, aggregate filling, and any other materials, labor, and equipment necessary to complete the work in accordance with the plans and specifications. 22.5.2.5 Cast-in-Place Concrete Slope Paving Cast-in-place concrete or pneumatically applied mortar slope paving measured in accordance with Article 22.5.1.4 will be paid for at the price bid per square yard or per cubic yard as set forth in the schedule of bid items. 22.5.2.6 Precast Concrete Slope Paving Precast concrete slope paving measured in accordance with Article 22.5.1.5 will be paid for at the price bid per square yard. Such price shall include cushion sand and shall include Portland cement grout or mortar, if required by the plans or specifications.
22.5.2.7 Filter Blanket Filter blanket or filter gravel measured in accordance with Article 22.5.1.1 will be paid for at the price bid per yard, per cubic yard, Or per ton as set forth in the schedule of bid items. 22.5.2.8 Filter Fabric Filter fabric measured in accordance with Article 22.5.1.6 will be paid for at the price bid per square yard. 22.5.2.9 Geocomposite Drain System Geocomposite drain system will be paid for on the basis of a contract lump sum price. Such lump sum price shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in constructing geocomposite drain systems complete in place including geocomposite drain, collection and discharge systems as shown on the plans, as specified in the special provisions and as directed by the Engineer. : Section 23 MISCELLANEOUS METAL 23.1 DESCRIPTION This work shall consist of furnishing and installing metd items in structures which are not otherwise provided for. Such work includes but is not limited to the following items: (1) Expansion joint armor in bridge decks, and sliding plate and finger type expansion joints. (2) Manhole frames and covers, drainage pipes, frames and grates, ladders or ladder rungs, access opening covers, and access door assemblies. (3) Other items specifically identified as miscellaneous metal on the plans or in the specifications. practice in modem commercial shops. Burrs, rough and sharp edges, and other flaws shall be removed. Warped pieces shall be straightened after fabrication and galvanizing. 23.4 GALVANIZING Unless otherwise specified all steel items, which are not embedded at least 2 inches in concrete, and all cast iron sidewalk frames and covers shall be galvanized in accordance with Articles 11.3.2.4 and 11.3.7 of Section 11, "Steel Structures." Assemblies shall be galvanized after fabrication. 23.2 MATERIALS Miscellaneous metal items shall be constructed of materials conforming to the following AASJ3T.O (or ASTM) material specifications: Material Steel Bars, Plates and Shapes Bolts and Nuts High-Strength Bolts, Nuts and Washers Steel Castings Grey Iron Castings Sheet Metal Specification AASHTO M 183 (ASTM A 36) ASTM A 307 AASHTO M 164 (ASTM A 325) As per Article 1 1.3.5 AASHTO M 105 (ASTM A 48), class 30 Commercial quality 23.3 FABRICATION Fabrication of miscellaneous metal items shall be performed in a workmanlike manner in conformance with the 23.5 MEASUREMENT Measurement of miscellaneous metal shall be by the scale weight. When requested by the Engineer, each delivery shall be accompanied with a certified weighmaster's .weight ticket. Scale weights are not required when calculated weights are shown on the plans, in which case these weights shall be used as the basis of payment. 23.6 PAYMENT Miscellaneous metal will be paid for by the contract unit price per pound. Such payment shall include full compensation for furnishing all labor, materials, tools, equipment and incidentals, and for doing all the work involved in furnishing and installing miscellaneous metal, complete in place, as shown on the plans, and as specified in these specifications and the special provisions, and as directed by the Engineer.
Section 24 PNEUMATICALLY APPLIED MORTAR 24.1 DESCRIPTION 24.2.3 Anchor Bolts or Studs This work shall consist of the furnishing and placing of pneumatically applied mortar for the construction of portions of structures, repairing concrete structures, texturing concrete surfaces, encasement of structural steel members, lining ditches and channels, paving slopes and for other miscellaneous work, all as shown on the plans. This work also includes the preparation of surfaces to receive the mortar and the furnishing and placing of any reinforcing steel and anchors for reinforcement. Pneumatically applied mortar shall consist of either dry mixed fine aggregate and Portland cement pneumatically applied by a suitable mechanism, to which mixture the water is added immediately previous to its expulsion from the nozzle, or mortar premixed by mechanical methods and pneumatically applied through a nozzle onto the prepared surface. Anchor studs used to support reinforcing wire fabric or bars when placing mortar against existing concrete or rock shall consist of Y4-inch minimum diameter expansion hook bolts placed in drilled holes. Each bolt shall have sufficient engagement in sound masonry to resist a pullout force of 150 lbs. When permitted by the Engineer, driven steel studs of not less than %-inchdiameter and a minimum length of 2 inches may be used. The equipment used for driving such studs shall be of the type which uses an explosive for the driving force, and shall be capable of inserting the stud or pin to the required depth without damage to the surrounding concrete. 24.3 PROPORTIONING AND MIXING 24.3.1 Proportioning The Contractor shall submit the proposed mix design to the Engineer for approval prior to start of the work. Unless otherwise specified, the mix design shall provide a cement to aggregate ratio, based on dry loose volumes, of not less than 1:3.5 for the construction and repair of concrete structures and for encasing steel members, or not less than 1:5 for lining ditches and channels and for paving slopes. The water content shall be as low as practical and shall be adjusted so that the mix is sufficiently wet to a&,ere properly and sufficiently dry so that it will not sag or fall from vertical or inclined surfaces or separate in horizontal work. 24.2 MATERIALS 24.2.1 Cement, Aggregate, Water and Admixtures Cement, aggregate, water and admixtures, when used, shall conform to the requirements of Section 8, "Concrete Smctures'" Aggregate shall be fine aggregate, that 30% coarse 43 for size % inch to No. 8 or No. 16, may be substituted for fine aggregate. Recovered rebound which is clean and free of foreign material may be reused as fine aggregate in quantities not to exceed 20% of the total fine aggregate requirements. 24.3.2 w i n g Mixing shall be done either by the dry mix or wet mix process. Before being charged into the placing equipment, the materials shall be thoroughly and uniformly mixed using a mixer designed for use with pneumatic application. It may be either a paddle type or drum type mixer. Transit mix equipment and methods may be used for the wet process. 24.2.2 Reinforcing Steel Reinforcing steel shall conform to the requirements of Section 9, "Reinforcing Steel." 653 654 HIGHWAY BRIDGES 24.4 SURFACE PREPARATION 24.4.1 Earth When pneumatically applied mortar is to be placed against earth, the area shall be accurately graded to the plan dimensions and shall be thoroughly compacted, with sufficient moisture to provide a firm foundation and to prevent absorption of water from the mortar, but shall not contain free surface water. When shown on the plans, joints, side forms, headers and shooting strips shall be provided for backing or paneling.
Ground or gaging wires shall be used where necessary to establish thicknesses, surface planes, and finish lines.
24.4.2 Forms When mortar is to be placed against forms, the forms shall conform to the requirement of Section 3, "Temporary Works." 24.4.3 Concrete or Rock When mortar is to be placed against concrete or rock, all deteriorated or loose material shall be removed by chipping with pneumatic or hand tools. Square or slightly undercut shoulders shall be cut approximately 1-inch deep along the perimeter of repair areas. The surface shall be sandblasted as necessary to clean all rust from exposed steel and to produce a clean rough-textured surface on the concrete or rock.
The surface against which mortar is to be placed shall be kept wet for at least 1 hour and then allowed to dry to a surface dry condition just prior to application of the mortar. 24.5 INSTALLATION 24.5.1 Placement of Reinforcing Reinforcing steel, when required, shall be installed in conformance with the requirements of Section 9, "Reinforcing Steel." Reinforcement in new construction shall be placed as specified in the plans and secured to insure no displacement from impact of the pneumatically placed mortar during application. For repair work, the reinforcing steel shall be supported by anchor studs installed in the existing masonry except where existing reinforcing steel in the repair area is considered by the Engineer to be satisfactory for this purpose. Anchors shall be spaced no more than 12 inches, 24.4 center to center, on overhead surfaces; 18 inches, center to center, on vertical surfaces; and 36 inches, center to center, on top horizontal surfaces. At least three anchors shall be used in each individual patch area. The Engineer shall be notified in advance of the date when installation of anchor studs is to begin. The locations of the studs shall be such that damage will not occur to prestressing tendons or conduits embedded in the concrete. Unless otherwise shown or specified, for repair work, all areas where the thickness of the mortar exceeds 1% inches shall be reinforced with a single layer of either 2 X 2 - W1 X W1 or3 X 3 - W1.5 X W1.5 weldedwirefabric. For areas where the thickness of the mortar exceeds 4 inches, a single layer of wire fabric shall be used to reinforce each 4-inch thickness of patch or fractional part thereof. All fabric shall be placed parallel to the proposed finished surface. Each layer of fabric shall be completely encased in mortar which has taken its initial set, before the succeeding layer of fabric is installed. Fabric supported adjacent to the prepared masonry surface shall be no closer than % inch to said surface. Fabric shall be carefully prebent before installation to fit around comers and into re-entrant angles, and shall in no case be sprung into place.
All steel items, including anchors, reinforcing bars and wire fabric, shall be no closer than 1 inch to the finished surface of the mortar. 24.5.2 Placement of Mortar Only experienced foremen, gunmen, nozzlemen, and rodmen shall be employed, and satisfactory evidence of such experience shall be furnished when requested by the Engineer. The mortar shall be applied by pneumatic equipment that sprays the mix onto the prepared surface at a high velocity as needed to produce a compacted dense homogeneous mass.
The air compressor and delivery hose lines shall be of adequate capacity and size to provide a minimum pressure of 35 psi at the nozzle for 1-inch nozzles and proportionally greater for larger nozzles. The velocity of the material as it leaves the nozzle must be maintained uniform at a rate determined for the given job conditions to produce minimum rebound. Water which is added at the nozzle shall be supplied at a uniform pressure of not less than 15 psi greater than the air pressure at the nozzle. The mortar shall be applied as dry as practicable to prevent shrinkage cracking. Shooting strips shall be employed to insure square comers, straight lines, and a plane surface of mortar, except as otherwise permitted by the plans or approved by the Engineer. They shall be so placed 24.5.2 DMSION 11--CONSTRUCTION as to keep the trapping of rebound at a minimum. At the end of each day's work, or similar stopping periods requiring construction joints, the mortar shall be sloped off to a thin edge. Before placing an adjacent section, construction joints shall be thoroughly cleaned and wetted as required under Article 24.4. In shooting all surfaces, the stream of flowing material from the nozzle shall impinge as nearly as possible at right angles to the surface being covered, and the nozzle shall be held from 2 to 4 feet from the working surface. A sufficient number of mortar coats shall be applied to obtain the required thickness. On vertical and overhead surfaces, the thickness of each coat shall be not greater than 1 inch, except as approved by the Engineer, and shall be so placed that it will neither sag nor decrease the bond of the preceding coat. The time interval between successive layers in sloping, vertical or overhanging work shall be sufficient to allow initial but not final set to develop. At the time the initial set is developing, the surface shall be cleaned to remove the thin film of laitance in order to provide for a bond with succeeding applications. Rebound or accumulated loose sand shall be removed from the surface to be covered prior to placing of the original or succeeding layers of mortar and shall not be embedded in the work. Materials that have been mixed for more than 45 minutes and have not been incorporated in the work shall not be used, unless otherwise permitted by the Engineer. After curing and before final acceptance, all repaired areas shall be sounded. All unsound and cracked areas shall be removed and replaced. 24.5.2.1 Weather Limitations Pneumatically placed mortar shall not be placed on a frozen surface nor when the ambient temperature is less than 40°F; nor shall it be placed when it is anticipated that the temperature during the following 24 hours will drop below 32°F The application of pneumatically placed mortar shall be suspended if high winds prevent proper application, or rain occurs which would wash out the pneumatically placed mortar. 24.5.2.2 Protection of Adjacent Work During progress of the work, where appearance is important, adjacent facilities which may be permanently dis- 655 colored, stained, or otherwise damaged by overspray, dust or rebound, shall be adequately protected and, if contacted, shall be cleaned by early scraping, brushing, or washing, as the surroundings permit. 24.5.3 Finishing After mortar has been placed to desired thickness, all high spots shall be cut off with a sharp trowel, or screeded to a true plane as determined by shooting strips or by the original masonry surface, or as directed. Cutting screeds, where used, shall be lightly applied to all surfaces so as not to disturb the mortar for an appreciable depth, and they shall be worked in an upward direction when applied on vertical surfaces. Unless otherwise directed, the finished mortar surface shall be given a final flash coat of about S/8 inch of mortar. Special care shall be taken to obtain a uniform appearance on all exposed surfaces. 24.5.4 Curing and Protecting Pneumatically placed mortar shall be water cured in conformance with the requirements of Article 8.11.3.2. The minimum water curing duration shall be 96 hours. The mortar shall be protected from freezing during the curing period. 24.6 MEASUREMENT AND PAYMENT The quantity of pneumatically applied mortar will be measured either by the square foot or by the cubic foot as indicated in the schedule of bid items. Square foot measurements will be based on measurements of the surface area of acceptable mortar placed in the work made along the plane or curve of each surface. Cubic foot measurement will be based on the dimensions of such work shown in the plans or ordered by the Engineer. Pneumatically applied mortar will be paid for by the contract price per square foot or cubic yard.
Such payment shall be considered to be full compensation for the cost of furnishing all labor, materials, equipment, incidentals, and for doing all work involved in preparing the surface and installing the mortar, reinforcing steel, anchor studs, headers, joint fillers, and other items as shown on the plans or specified. Section 25 STEEL AND CONCRETE TUNNEL LINERS 25.1 SCOPE These specifications are intended to cover the installation of tunnel liner plates in tunnels constructed by conventional tunnel methods. For the purposes of these Specifications, tunnels excavated by full face, heading and bench, or multiple drift procedures are considered conventional methods. Liner plates used with any conshuction procedure utilizing a full or partial shield, a tunneling machine, or other piece of equipment which will exert a force on the liner plates for the purpose of propelling, sieering, or stabilizing the equipment are considered special cases and are not covered by these Specifications. 25.2 DESCRIPTION 25.2.1 This work shall consist of furnishing cold-formed steel tunnel liner plates or precast concrete plates conforming to these specifications and of the sizes and dimensions required on the plans, and installing such plates at the locations designated on the plans by the Engineer, and in conformity with the lines and grades established by the Engineer. The completed liner shall consist of a series of liner plates assembled with staggered longitudinal joints. Steel tunnel liner plates shall preferably be of a type which is commercially available. Precast concrete tunnel liner plates shall be such that their size and shape suits the method and equipment being used to install them. 25.3 MATERIALS AND FABRICATION Liner plates shall be fabricated to fit the cross section of the tunnel.
25.3.1 General Steel liner plates herein described must meet the Sectional Properties of thickness, area, and moment of inertia shown on the plans. If not shown on the plans, the properties shall be as listed in Division I, Article 16.3. All steel plates shall be connected by bolts on both longitudinal and circumferential seams or joints and shall be so fabricated as to permit complete erection from the inside of the tunnel. Bolt sizes and properties shall be in accordance with the manufacturer's standard but not less than those specified in Division 1, Article 16.7. Grout holes 2 inches or larger in diameter shall be provided as shown on the plans to permit grouting as the erection of tunnel liner plates progresses. Precast concrete tunnel liner plates shall conform to the details shown on the plans and the requirements of Section 8, "Concrete Structures." If such details are not provided and the plans or the specifications allow the Contractor to propose the use of concrete liner plates, the Contractor shall submit working drawings and specifications to the Engineer for approval. Such drawings and specifications shall describe materials to be used, plate dimensions, reinforcement details, connecting details, and erection procedures. The fabrication of Contractor proposed types of concrete tunnel liner plates shall not begin until the working drawings have been approved. Such approval shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work. 25.3.2 Forming and Punching of Steel Liner Plates All plates shall be formed to provide circumferential flanged joints. Longitudinal joints may be flanged or of the offset lap seam type. All plates shall be punched for bolting on both longitudinal and circumferential seams or joints.
Bolt spacing in circumferential flanges shall be in accordance with the manufacturer's standard spacing and shall be a multiple of the plate length so that plates having the same curvature shall be interchangeable and will permit staggering of the longitudinal seams. Bolt spacing at flanged longitudinal seams shall be in accordance with the manufacturer's standard spacing. For lapped longitudinal seams, bolt size and spacing shall be in accordance with the manufacturer's standard but not less than that required to meet the longitudinal seam strength requirements of Division I, Article 16.3.2. 658 HTGHWAY BRIDGES 25.4 25.4 INSTALLATION 25.4.3 Grouting 25.4.1 Steel Liner Plates When directed by the Engineer, voids occurring between the liner plate and the tunnel wall shall be forcegrouted. The grout shall be forced through the grouting holes in the plates with such pressure that all voids will be completely filled.
Full compensation for back packing or grouting shall be considered as included in the contract price paid for tunnel and no separate payment will be made therefore. All steel liner plates for the full length of a specified tunnel shall be of one type only, either the flanged or the lapped seam type of construction. Liner plates shall be assembled in accordance with the manufacturer's instruction. Coated steel plates shall be handled in such a manner as to prevent bruising, scaling, or breaking of the coating. Any plates that are damaged during handling or placing shall be replaced by the Contractor at own expense, except that small areas with minor damage may be repaired by the Contractor as directed by the Engineer. 25.5 MEASUREMENT The length of tunnel liner to be paid for will be the length measured along the tunnel liner plate invert. 25.6 PAYMENT 25.4.2 Precast Concrete Liner Plates Installation of precast concrete tunnel liner plates shall not start prior to receipt of approval of working drawings and specifications submitted as required by Article 25.3.1. Installation shall conform to the specified or approved erection procedures. Payment for the length of each size of tunnel as determined under measurement shall be at the contract unit prices per linear foot bid for the various sizes, which payment shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals to complete this item, including the force-grouting of voids. Section 26 METAL CULVERTS 26.1 GENERAL 26.3 MATERIALS 26.1.1 Description 26.3.1 Corrugated Metal Pipe This work shall consist of furnishing, fabricating, and installing metal pipe, metal structural plate pipe, arches, pipe arches, and box structures in conformance with these specifications, the special provisions, and the details shown on the plans. As used in this specification, longspan structures are metal plate horizontal ellipse, inverted pear and multiple radius arch shapes as well as special shape culverts as defined in Division I, Section 12, "SoilCorrugated Metal Structure Interaction Systems." The terms "metal pipe" and "metal structural plate pipe" shall include both circular pipe arch, underpass and elliptical shapes. "Metal structural plate arches" consist of a metal plate arch supported on reinforced concrete footings at its base (ends) with or without a paved invert slab. "Pipe arches" are constructed to form a pipe having an archshaped crown and a relatively flat invert. "Metal structural plate box structures" are conduits, rectangular in cross section, constructed of metal plates. Steel pipe shall conform to the requirements of AASHTO M 36. Aluminum pipe shall conform to the requirements of AASHTO M 196. 26.2 WORKING DRAWINGS Whenever specified or requested by the Engineer, the Contractor shall provide manufacturer's assembly instructions or working drawings with supporting data in sufficient detail to permit a structural review. Sufficient copies shall be furnished to meet the needs of the Engineer and other entities with review authority. The working drawings shall be submitted sufficiently in advance of proposed use to allow for their review, revision, if needed, and approval without delay to the work. The Contractor shall not start the construction of any metal culvert for which working drawings are required until the drawings have been approved by the Engineer.
Such approval will not relieve the Contractor of responsibility for results obtained by use of these drawings or any of his or her other responsibilities under the contract. 26.3.2 Structural Plate Steel structural plate shall conform to the requirements of AASHTO M 167.
Aluminum alloy structural plate shall conform to the requirements of AASHTO M 219. 26.3.3 Nuts and Bolts Nuts and bolts for steel structural plate pipe, arches, pipe arches, and box structures shall conform to the requirements of AASHTO M 167. Nuts and bolts for aluminum structural plate shall be aluminum conforming to ASTM F 468 or standard strength steel conforming to ASTM A 307. 26.3.4 Mixing of Materials Aluminum and steel materials shall not be mixed in any installation unless they are adequately separated or protected to avoid galvanic reactions. Hot-dip galvanizing provides such protection. Hot-dip galvanized steel and stainless steel bolts and nuts are acceptable for aluminum structural plate. 26.3.5 Fabrication Plates at longitudinal and circumferential seams shall be connected by bolts with the seams staggered so that not more than three plates come together at any one point. HIGHWAY BRIDGES 26.3.6 Welding Welding of steel, if required, shall conform to the ANSI/AASHTO/AWS Bridge Welding Code D1.5. All welding of steel plates, other than fittings, shall be performed prior to galvanizing. Welding of aluminum, if required, shall conform to the AWS D1.2, "Structural Welding Code." 26.3.7 Protective Coatings When required by the plans or the special provisions, metal pipes and structural plate shall be protected with bituminous coating or have the invert paved with bituminous material. Bituminous coatings shall be applied as provided in AASHTO M 190, Type A, unless otherwise specified. Bituminous pavings, if required, shall be applied over the bituminous coatings to the inside bottom portion of pipe as provided in AASHTO M 190, Type C, unless otherwise specified. The portion of all nuts and bolts used for assembly of coated structural plate projecting outside the pipe, shall be coated after installation.
The portions of the nuts and bolts projecting inside the pipe need not be coated. Polymeric coatings, when called for on the plans or in the special provisions, shall conform to the requirements of AASHTO M 246. The polymeric coating shall be applied to the galvanized sheet prior to corrugating and, unless otherwise specified, the thickness shall be not less than 0.010 inch. Any pinholes, blisters, cracks, or lack of bond shall be cause for rejection. Polymeric coatings will not be permitted on structural plate pipes. 26.3.8 Bedding and BackfiU Materials 26.3.8.1 General Bedding material shall be loose native or granular material with a maximum particle (or clump) size not to exceed one-half the corrugation depth. Backfill for metal culverts shall be granular material as specified in the plans and specifications and shall be free of organic material, stones larger than 3 inches in the greatest dimension, frozen lumps, or moisture in excess of that permitting thorough compaction. As a minimum, backfill materials shall meet the requirements of AASHTO M 145 for A- 1, A-2, or A-3. 26.3.8.2 Long-Span Structures Bedding and backfill materials shall meet the general requirements of Article 26.3.8.1. As a minimum backfill materials for structures with less than 12 feet of cover shall meet the requirements of AASHTO M 145 for A-1, A-2-4, 26.3.6 A-2-5, or A-3. Minimum backfill requirements for structures with 12.0 feet or more cover shall meet AASHTO M 145 requirements for A-1 or A-3. 26.3.8.3 Box Culverts Bedding and backfill materials shall meet the general requirements of Article 26.3.8.1. As a minimum, backfill shall meet the requirements of AASHTO M 145 for A- 1, A-2-4, A-2-5, or A-3. 26.4 ASSEMBLY 26.4.1 General Corrugated metal pipe and structural plate pipe shall be assembled in accordance with the manufacturer's instructions. All pipe shall be unloaded and handled with reasonable care. Pipe or plates shall not be rolled or dragged over gravel or rock and shall be prevented from striking rock or other hard objects during placement in trench or on bedding. Corrugated metal pipe shall be placed in the bed starting at the downstream end. Pipes with circumferential seams shall be installed with their inside circumferential sheet laps pointing downstream. Bituminous coated pipe, polymer coated pipe, and paved invert pipe shall be installed in a similar manner to corrugated metal pipe with special care in handling to avoid damage to coatings. Paved invert pipe shall be installed with the invert pavement placed and centered on the bottom. Structural plate shall be assembled and installed in accordance with the plans and detailed erection instructions. Copies of the manufacturer's assembly instructions shall be furnished as specified in Article 26.2. Bolted longitudinal seams shall be well fitted with the lapping plates parallel to each other. The applied bolt torque for %inch diameter high-strength steel bolts (A 449

These shapes shall be not less than)/16 inch in thickness and shall be securely anchored to the footing at a maximum spacing of 24 inches. When the metal bearing member is not completely embedded in a groove in the footing, one vertical 26.5.5 DMSION II--CONSTRUCTION leg shall be punched to allow the end of the corrugated plates to be bolted to this leg of the bearing member. Where an invert slab is provided which is not integral with the arch footing, the invert slab shall be continuously reinforced. 26.5.6 Inspection Requirements for CMP All pipe shall undergo inspection during and after installation to ensure proper performance. Inspections at the appropriate times during installation will detect and allow early correction of line and grade, jointing and shape change problems. CMP installation can be properly monitored and evaluated by visual inspection. The timing and number of inspections required will vary with the significance of the installation. Pipes shall be inspected by entering the pipe, or by inspection from both the inlet and outlet (or other access points) by visual means or through the use of video equipment. CMP shall be inspected after placement in the trench, and as required during backfilling to ensure that final installation conditions allow the pipe to perform as designed. construction inspection during early stages of the project will allow the contractor to evaluate and, if necessary, modify construction and quality control practices. This is particularly important in deep installations. The inspector will verify that bedding, backfill and compaction requirements are followed during installation.
The pipe shall be checked for alignment, joint separation, cracking at bolt holes, localized distortions, bulging, flattening,or racking. Minimum or near-minimum TABLE 26.6 Minimum Cover for Construction Loads (Round, Pipe-Arch, Ellipse and Underpass Shapes) Minimum Cover (ft) for Indicated Axle Loads (thousands of pounds)* - Pipe Span, in. 18-50 50-75 75-110 110-150 h i m u m cover may vary, depending on lo& conditions.
The contractor must provide the additional cover required to avoid damage to the pipe. hfinimum cover is measured from the top of the pipe to the top of the maintained constructionroadway surface. In unpaved situation, the surface must be maintained. 667 cover installations should be inspected prior to and immediately after vehicular load is applied. 26.6 CONSTRUCTION PRECAUTIONS These structures can carry legal highway loads once the backfill is placed and compacted to the minimum cover level over-the pipe as defined by Division I, Section 12, "Soil-Corrugated Metal Structure Interaction." For heavier construction loads, additional cover may be required. Table 26.6 provides guidance for smaller structures. Consult the Engineer or the manufacturer for guidance on structures or axle loads not listed. The structure must be protected from hydraulic forces during construction, prior to the completion of permanent erosion control and end protection.
Hydraulic forces may cause erosion, shape distortion, flotation or washout. Backfill and other earth loads must be kept balanced. (See Article 26.5.4.) 26.7 MEASUREMENT Corrugated metal and structural plate pipe, pipe arches, arches and box culverts shall be measured in lineal feet installed in place, completed and accepted. The number of lineal feet shall be the average of the top and bottom center line lengths for pipe, the bottom center line length for pipe arches and box culverts, and the average of springing line lengths for arches. 26.8 PAYMENT Separate pay items or provision for including excavation, backfill, and concrete for arches must be provided for in the contract. The lengths as measured above will be paid for at the contract prices per lineal foot bid for corrugated metal and structural plate pipe, pipe-arch, arch or box culvert of the sizes specified. Such price and payment shall constitute full compensation for furnishing, handling, erecting, and installing the pipe, pipe-arches, arches or box culverts, and for all materials, labor, equipment, tools and incidentals necessary to complete this item. Such price and payment shall also include excavation, bedding material, backfill, concrete headwalls, endwalls and foundations for pipe, pipe-arches and box culverts. Separate payment will be made for excavation, backfill, and concrete or masonry headwalls and foundations for arches. Section 27 CONCRETE CULVERTS 27.1 GENERAL Precast Reinforced Concrete This work shall consist of fabricating, furnishing, and installing buried precast concrete culverts conforming to these Specifications, the special provisions and the details shown on the plans. Precast reinforced concrete pipe shall be circular, arch or elliptical, as specified. Precast reinforced concrete box sections shall be of the dimensions specified or shown on the plans. Elliptical Pipe 27.2 WORKING DRAWINGS Box Sections When complete details are not provided in the plans, or when required or permitted by provisions in the contract, the Contractor shall prepare and submit to the Engineer working drawings of the structure or installation system proposed for use. Fabrication or installation of the structure shall not begin until the Engineer has approved the drawings. The working drawings shall show complete details and substantiating calculations of the structure, the materials, equipment and installation methods the Contractor proposes to use. Working drawings shall be submitted sufficiently in advance of the start of the affected work to allow time for review by the Engineer and correction of the submittal by the Contractor without delaying the work. Approval by the Engineer shall not relieve the Contractor of any responsibility under the contract for the successful completion of this work. 27.3 MATERIALS 27.3.1 Reinforced Concrete Culverts The materials for reinforced concrete culverts shall meet the requirements of the following specifications for the classes and sizes specified above. Circular Pipe Arch Pipe AASHTO M 170 or AASHTO M 242 (ASTM C 76 or C 655) AASHTO M 206 (ASTM C 506) AASHTO M 207 (ASTM C 507) AASHTO M 259 and AASHTO M 273 (ASTM C 789 and C 850) 27.3.2 Joint Sealants 27.3.2.1 Cement Mortar Mortar shall be composed of one part Portland cement and two parts sand by volume. Sand shall be well graded and of such size that all will pass a No. 8 sieve. The materials shall be mixed to a consistency suitable for the purpose intended and used within 30 minutes after the mixing water has been added. Admixtures, if any, shall be approved by the Engineer prior to use. 27.3.2.2 Flexible Watertight Gaskets Flexible watertight gasketed joints shall conform to the requirementsof AASHTO M 198 and shall be flexible and capable of withstanding expansion, contraction, and settlement of the pipeline. All rubber gaskets shall be stored in as cool a place as practicable, preferably at 70°For less. Rubber gaskets, of the type requiring lubrication, shall be lubricated with the lubricant recommended and supplied by the manufacturer of the pipe. 670 HIGHWAY BFUDGES 27.3.2.3 Other Joint Sealant Materials Other joint sealant materials shall be submitted for testing in advance of their use and shall not be used prior to receiving approval by the Engineer. 27.3.3 Bedding, Haunch, Lower Side and Backfill or Overfill Material 27.3.3.1 Precast Reinforced Concrete Circular, Arch, and Elliptical Pipe Bedding, haunch, lower side and overfill material shall conform to Figures 27.5A, 27.5B, 27.5C, and 27.5D which define soil areas and critical dimensions, and Tables 27.5A and 27.5B, which list generic soil types and minimum compaction requirements, and minimum bedding thicknesses for the four Standard Installation Types. The AASHTO Soil Classifications and the USCS Soil Classifications equivalent to the generic soil types in the Standard Installations are presented in Table 27.5C.
27.3.3.2 Precast ~einforcedConcrete Box Sections For precast reinforced concrete box sections, bedding and backfill material shall conform to Figure 27.5E with the following exceptions. Bedding material may be sand or select sandy soil all of which passes a U.S. Standard %-inch sieve and not more than 10% of which passes a U.S. Standard No. 200 sieve. Backfill may be select material and shall be free of organic material, stones larger than 3 inches in the greatest dimension, frozen lumps, or moisture in excess of that permitting the specified compaction. 27.4 ASSEMBLY 27.3.2.3 cracks 0.01 inch or less in width are considered acceptable without repair. Cracks determined to be detrimental shall be sealed by a method approved by the Engineer. 27.4.2 Joints Joints for reinforced concrete pipe and precast reinforced concrete box sections shall comply with the details shown on the plans, the approved working drawings, and the requirements of the special provisions. Each joint shall be sealed to prevent infiltration of soil fines or water as required by the contract documents. Joint sealant materials shall comply with the provisions of Article 27.3.2. The Contractor shall furnish to the Engineer a certificate of compliance that the material being furnished conforms to the joint property requirements. Field tests may be required by the Engineer whenever there is a question regarding compliance with contract requirements. 27.5 INSTALLATION 27.5.1 General Trenches shall be excavated to the dimensions and grade specified in the plans or ordered by the Engineer. The Contractor shall make such provisions as required to insure adequate drainage of the trench to protect the bedding during construction operations. Proper preparation of foundation, placement of foundation material where required, and placement of bedding material shall precede the installation of the culvert. This shall include necessary leveling of the native trench bottom or the top of foundation materials as well as placement and grading of required bedding material to a uniform grade so that the entire length of pipe will be supported on a uniform slightly yield bedding. The backfill material shall be placed around the culvert in a manner to meet the requirements specified. 27.4.1 General 27.5.2 Bedding Precast concrete units or elements shall be assembled in accordance with the manufacturer's instructions. All units or elements shall be handled with reasonable care and shall not be rolled or dragged over gravel or rock. Care shall be taken to prevent the units from striking rock or other hard objects during placement. Cracks in an installed precast concrete culvert that exceed 0.01-inch width will be appraised by the Engineer considering the structural integrity, environmental conditions, and the design service life of the culvert.
Generally in noncorrosive environments, cracks 0.10 inch or less in width are considered acceptable; in corrosive environments, those 27.5.2.1 General If rock strata or boulders are encountered under the culvert within the limits of the required bedding, the rock or boulders shall be removed and replaced with bedding material. Special care may be necessary with rock or other unyielding foundations to cushion pipe from shock when blasting can be anticipated in the area. Where, in the opinion of the Engineer, the natural foundation soil is such as to require stabilization, such material shall be replaced by a layer of bedding material. Where an unsuitable material Overfill - SW. ML, or CL Bc I I Bc (Min.) Haunch - See Table 27.5 Lower Side - See Toble 2 Bedding - See Table 27.5 A Outer Bedding material and compaclion eoch side. same requirements us haunch '/ " 1Bc,/3I7 7 Foundolion / Middle Bedding loosely placed u n c o m ~ a ted c beddinq excepts for Type 4 FIGURE 27.5A Standard Embankment Installations - 672 HIGHWAY BRIDGES 27.5.2.1 Overlill - SW. ML, or - Haunch See Table 27.58 Lower Side See Table 27.58 - Bedding 3, 1 lLMiddle -Outer See Toble 27.58 Bedding moterials and c0mmc1ion each side. same requirements as haunch Bedding loosely ploced uncompocted bedding Bc 3 LEGEND: BE = Outside Diameter r H -. Backfill cover above top pipe a ,'-- Foundation O f HORtZONTALELLIPTICAL PlPE - SW. Oved~ll Ioosdy ploced uncompacted bedding materials and compaqtion wch re$$!erX: as haunch / LEGEND: Bc = Outside Diameter H = Backlill cover above top of pipe VERnCAL ELLIPTICAL PIPE ML, or - Haunch See Tab* 27.58 Lower Side See Table 27.38 - Bedding d,i1 -Outer See Table 27.58 j' Bedding materials and compaction w c h side, same requirements 0s haunch Liddie *OCd hdy uncompaeted bedding / LEGEND: Bc = Outside Diameter H = Backfill cover above top of pipe Foundation ARCH PlPE FIGURE 27.5C Trench Beddings, Miscellaneous Shapes (peat, muck, etc.) is encountered at or below invert elevation during excavation, the necessary subsurface exploration and analysis shall be made and corrective treatment shall be as directed by the Engineer. 27.5.2.2 Precast Reinforced Concrete Circular Arch and Elliptical Pipe A bedding shall be provided for the type of installation specified conforming to Figures 27.5A, 27.5B, 27.5C, and 27.5D which define soil areas and critical dimensions, and Tables 27.5A and 27.5B, whiclflist generic soil types and minimum compaction requirements, and minimum bedding thicknesses for the four Standard Installation Types. 27.5.2.3 Precast Reinforced Concrete Box Sections A bedding shall be provided for the type of installation specified conforming to Figure 27.5E unless in the opinion of the Engineer, the natural soil provides a suitable bedding. 27.5.3 Placing Culvert Sections Unless otherwise authorized by the Engineer, the laying of culvert sections on the prepared foundation shall be started at the outlet and with the spigot or tongue end pointing downstream and shall proceed toward the inlet 674 HIGHWAY BRIDGES Overfill - SW. ML, or CL 27.5.3 Overlill I - SW. ML.
or CL - See Table 27.5A Haunch See Table 27.5A Lower Side See Table 27.54 - ing - See Toble 27 5A outer Bedding mate 3 11 Be13 Foundation loosely uncompacted beddifq - W D : Be = Outside Diameter ti I Backfill cover above top of pipe H Backfill cover above top of pipe VERTICAL ELLIPTICAL PIPE HORIZONTAL ELLIPTICAL PlPE Overfill - - SW. ML. or CL - Haunch See Toble 27.514 Lower Side See Table 2 7 9 - Spring Outer Bedding materials and compaction each side. same rquiremenb or haunch \ ~ i d d k Bedding laasely placed uncompacted bedding 8c/3 /- , -- LEGEND: Outside Diameter Bc H Backfill cover above - top of pipe Foundation ARCH P I E FIGURE 27.5D Embankment Beddings, Miscellaneous Shapes end with the abutting sections properly matched, true to the established lines and grades. Where pipe with bells is installed, bell holes shall be excavated in the bedding to such dimensions that the entire length of the barrel of the pipe will be supported by the bedding when properly installed. Proper facilities shall be provided for hoisting and lowering the sections of culvert into the trench without disturbing the prepared foundation and the sides of the trench. The ends of the section shall be carefully cleaned before the section is jointed. The section shall be fitted and matched so that when laid in the bed it shall form a smooth, uniform conduit. When elliptical pipe with circular reinforcing or circular pipe with elliptical reinforcing shall be laidin the trench in such position is used, the that the markings ., '"ToD" or "Bottom," shall not be more than 5" from the vertical plane through the longitudinal axis of the pipe. Multiple installations of reinforced concrete culverts shall be laid with the center lines of individual barrels parallel at the spacing shown on the plans. Pipe and box sections used in parallel installations require positive lateral bearing between the sides of adjacent pipe or box sections. Compacted earth fill, granular backfill, or grouting between the units are considered means of providing positive bearing. 27.5.4 Haunch, Lower Side and Backfill or Overfill 27.5.4.1 Precast Reinforced Concrete Circular Arch and Elliptical Pipe 27.5.4.1.1 Haunch Material Haunch material shall be installed to the limits shown on Figure 27.5A, 27.5B, 2 7 . X and 27.5D. 27.5.4.1.1 DMSION II--CONSTRUCTION 675 TABLE 27.5A Standard Embankment Installation Soils and M i u m compaction Requirements Installation Type Bedding Thickness Haunch and Outer Bedding Lower Side Bc/24" minimum, not less than 3".
If rock foundation, use Bc/12" minimum, not less than 6" I Bc/24" minimum, not less than 3". Ifrock foundation, use Bc112" minimum, not less than 6" m e3 (See Note 3.) 85% SW, 90% ML or Bc/24" minimum, not less than 3". Ifrock foundation, use Bc/12" minimum, not less than 6" 85% SW, 90% ML, or 95% CL 85% SW, 90% ML or 95% CL No bedding required, except if rock foundation, use Bc/12" minimum, not less than 6" No compaction required, except if CL, use 85% CL No compaction required, except if CL.
use 85% CL NOTES: 1. Compaction and soil symbols-i.e. ' 9 5 % SW" refers to SW soil material with a minimum standard proctor compaction of 95%.
See Table 27.5C for equivalent modified proctor values. 2. Soil in the outer bedding, haunch, and lower side zones, except within B,/3 from the pipe springline, shall be compacted to at least the same compaction as the majority of soil in the overfill zone. 3. Only Type 2 and 3 installations are available for horizontal elliptical, vertical elliptical and arch pipe. 4.1 A subtrench is defined as a trench with its top below finished grade by more than 0.1Hor, for roadways, its top is at an elevation lower than 1' below the bottom of the pavement base material. 4.2 The minimum width of a subtrench shall be 1.33 B,, or wider if required for adequate space to attain the specified compaction in the haunch and bedding zones.
4.3 For subtrenches with walls of natural soil, any portion of the lower side zone in the subtrench wall should be at least as 6rm as an equivalent soil placed to the compaction requirements specified for the lower side zone and as firm as the maioritv of soil in the overfill zone. or shall be removed and reolaced with soil compacted to the specified lev& - 676 HIGHWAY BRIDGES 27.5.4.1.1 TABLE 27.5B Standard Trench Installation Soils and Minimum Compaction Requirements Haunch and Outer Bedding Lower Side Installation Type Bedding Thickness Type 1 Bc/24" minimum, not less than 3". Ifrock foundation, use Bc/12" minimum, not less than 6" 95% SW 90% SW, 95% ML 100% CL, or natural soils of equal firmness Type 2 B,/24" minimum, not less than 3". Ifrock foundation, use Bc/12" minimum, not less than 6" 90% SW or 95% ML 85% SW, 90% ML 95% CL, or natural soils of equal firmness Bc/24" minimum, not less than 3".
Ifrock foundation, use Bc/12" minimum, not less than 6" 85%SW,90%ML,or 95% CL 85%SW,90%ML 95% CL, or natural soils of equal firmness No bedding required, except if rock foundation, use Bc/12" minimum, not less than 6" No compaction required, except if CL, use 85% CL 85% SW, 90% ML 95% CL, or natural soils of equal firmness (see Note 3) m e3 (see Note 3) Type 4 NOTES: 1. Compaction and soil symbols-i.e. "95% SW' refers to SW soil material with a minimum standard proctor compaction of 95%. See Table 27.5C for equivalent modified proctor values. 2.
The trench top elevation shall be no lower than .0.1H below finished grade or, for roadways, its top shall be no lower than an elevation of 1' below the bottom of the pavement base material. 3. Only m e 2 and 3 installations are available for horizontalelliptical, vertical elliptical and arch pipe. 4. Soil in bedding and haunch zones shall be compacted to at least the same compaction as specified for the majority of soil in the backfill zone. 5. The trench width shall be wider than shown if required for adequate space to attain the specified compaction in the haunch and bedding zones. 6. For trench walls that are within 10 degrees of vertical, the compaction or finnness of the soil in the trench walls and lower side zone need not be considered. 7.
For trench walls with greater than 10-degree slopes that consist of embankment, the lower side shall be compacted to at least the same compaction as specified for the soil in the backfill zone. 27.5.4.1.2 DWISION 11--CONSTRUCTION 677 TABLE 27.5C Equivalent USCS and AASHTO Soil Classifications for SIDD Soil Designations Representative Soil Qpes Percent Compaction SIDD Soil USCS AASHTO Gravelly Sand (SW) SW, SP GW, GP Al, A3 Sandy Silt GM, SM, ML Also GC, SC with less than 20% passing No. 200 sieve A2, A4 Silty Clay (CL) GL, MH GC,SC 27.5.4.1.2 Lower Side Material Lower side material shall be installed to the limits shown on Figures 27.5A, 27.5B, 27.5C, and 27.5D. Overfill material shall be installed to the limits shown on Figures 27.5A, 27.5B, 27.32, and 27.5D. 27.5.4.2 Precast Reinforced Concrete Box Sections Backfill material shall be installed to the limits shown on Figure 27.5E for the embankment or trench condition. Trenches shall have vertical walls and no over-excavating or sloping sidewalls shall be permitted. Standard Proctor Modified Proctor 27.5.4.3 Placing of Haunch, Lower Side and Backfill or Overfill Generally, compaction of fill material to the required density is dependent on the thickness of the layer of fill being compacted, soil type, soil moisture content, type of compaction equipment, and amount of compactive force and the length of time the force is applied. Fill material shall be placed in layers with a maximum thickness of 8 inches and compacted to obtain the required density. The fill material shall be placed and compacted with care under the haunches of the culvert and shall be brought up evenly and simultaneously on both sides of the culvert. For the lower haunch areas of Type 1,2, and 3 Standard Installations, soils requiring 90% or greater Standard Proctor densities shall be placed in layers with a maximum thickness of 4 inches and compacted to obtain the required density.
The width of trench shall be kept to the minimum required for installation of the culvert. Ponding or jetting will be only by the permission of the Engineer.
678 HIGHWAY BRIDGES 27.5.4.4 L ~ e v r ~ ~ i n g course bedding material 3" min. Existing ground or fill BOX SECTIONS EMBANKMENT BEDDING Bedding Material 2 3 min. BOX SECTIONS TRENCH BEDDING FIGURE 27.53 27.5.4.4 Cover Over Culvert During Construction Culverts shall be protected by a minimum of 3 feet of cover to prevent damage before permitting heavy construction equipment to pass over them during construction. 27.6 MEASUREMENT Culverts shall be measured in linear feet installed in place, completed, and accepted. The number of feet shall be the average of the top and bottom center line lengths for pipe and box sections. 27.7 PAYMENT The length determined as herein given shall be paid for at the contract unit prices per linear foot bid for culverts of the several sizes and shapes, as the case may be, which prices and payments shall constitute full compensationfor furnishing, handling, and installing the culvert and for all materials, labor, equipment, tools, and incidentals necessary to complete this item. Such price and payment shall also include excavation, bedding material, backfill, reinforced concrete headwalls and endwalls, and any required foundations. Section 28 WEARING SURFACES 28.1 DESCRIPTION This work shall consist of placing a wearing surface of durable and impervious material on the roadway surface of bridge decks. It also includes the preparation of the surfaces of either existing or new decks to receive such an overlay of surfacing material. The type and thickness of the wearing surface shall be as designated on the plans.
The materials and installation requirements for wearing surfaces of types other than latex modified concrete shall be as specified in the special provisions. Latex modified concrete wearing surfaces shall be furnished and installed in accordance with these Specifications. 28.2 LATEX MODIFIED CONCRETE TYPE WEARING SURFACE 28.2.1 General All equipment used to prepare the surface and to proportion, mix, place and finish the latex concrete shall be subject to approval by the Engineer prior to use. This approval will be contingent on satisfactory performance and will be rescinded in the event such performance is not being achieved. Equipment shall be on hand sufficiently ahead of the start of construction operations to be examined and approved. Any equipment leaking oil or any other containment onto the deck shall be immediately removed from the job site until repaired. A technician who is well experienced in the proportioning, mixing, placing and finishing of latex modified concrete shall be employed by the Contractor and shall be present and in technical control of the work whenever these operations are underway. The qualifications of this technician which includes a list of projects on which the technician was employed and the technician's level of responsibility on each shall be submitted to and approved by the Engineer prior to the start of these operations. Approval by the Engineer of equipment or technicians shall not relieve the Contractor of any responsibility under the contract for the successful completion of the work. If not otherwise shown on the plans, the minimum thickness of latex modified concrete wearing surfaces shall be 11/4 inches. 28.2.2 Materials 28.2.2.1 Portland Cement Portland cement shall conform to the requirement of Article 8.3.1 of Section 8, "Concrete Structures," except that only Types I or I1 shall be used. 28.2.2.2 Aggregate Aggregate shall conform to the requirements of AASHTO M 6 for fine aggregate and to AASHTO M 80 for coarse aggregate. Coarse aggregate shall be graded l/2 inch to No. 4 per AASHTO M 43. 28.2.2.3 Water Water for mixing concrete shall conform to the requirements of Article 8.3.2. 28.2.2.4 Latex Emulsion Formulated latex emulsion admixture shall be a nonhazardous, film forming, polymeric emulsion in water to which all stabilizers have been added at the point of manufacture and shall be homogeneous and uniform in composition. Physical Properties-The latex modifier shall conform to the following requirements: Polymer n p e Stabilizers Styrene Butadiene (a) Latex ..... Nonionic Surfactants (b) Portland Cement Composition ..... Polydimethyl Siloxane Percent Solids .....
46.0-49.0 Weight per Gallon (lbs at 25°C) .....8.4 Color ....White HIGHWAY BRIDGES A Certificate of Compliance signed by the manufacturer of the latex emulsion certifying that the material conforms to the above specifications shall be furnished for each shipment used in the work. Latex admixture to be stored shall be kept in suitable enclosures which will protect it from freezing and from prolonged exposure to temperatures in excess of 85°F. Containers of latex admixture may be stored at the bridge site for a period not to exceed 10 days. Such stored containers shall be covered completely with suitable insulating blanket material to avoid excessive temperatures. 28.2.2.5 Latex Modified Concrete The latex modified concrete for use on this project shall be a workable mixture and meet the following requirements. Material or Property Concrete Cement (Parts by weight) (Note 3).
. . . . . Fine Aggregate (Parts by weight) (Note 3) . Coarse Aggregate (Parts by weight) (Note 3) Latex Emulsion Admixture(Gals /Bag Cement) . . . . . Air Content of Plastic Mix, % (AASHTO T 152) . . . . . .
. . . . . Slump, Inches (Notes 1 and 2) .
. . . . .
. . . . . 1 2.5 2.0 3.5 3-6 3-6 NOTES: 1. Following sampling of the discharged, normally mixed material, the commencement of the slump test shall be delayed from 4 to 5 minutes. 2. Water may be added to obtain slump within the prescribed limits.
3. The dry weight ratios are approximate and should produce good workability, but due to gradation changes may be adjusted within limits by the Engineer. The parts by weight of sand may be increased by as much as 0.2 if the coarse aggregate is reduced by an equivalent volume. 28.2.3 Surface Preparation 28.2.3.1 New Decks The surfaces of new decks upon which a wearing surface overlay is to be placed shall be finished to a rough texture by coarse brooming or other approved methods. After curing of the deck concrete is complete and before placing the overlay, the entire area of the deck surface and the vertical faces of curbs, concrete parapets, barrier walls, etc., up to a height of 1 inch above the top elevation of the overlay shall be blast cleaned to a bright, clean appearance which is free from laitance, curing compound, dust, dirt, oil, grease, bituminous material, paint, and all foreign matter. The blast cleaning of an area of the deck shall normally be performed within the 24-hour period preceding placement of the overlay on the area. The blast cleaning may be performed by either wet sandblasting, high pressure water blasting, blasting grits, shrouded dry sandblasting with dust collectors, or other method approved by the Engineer. Water blasting equipment shall operate with a minimum pressure of 3,500 psi. The method used shall be performed so as to conform to applicable air and water pollution regulations and to applicable safety and health regulations. All debris, including dirty water, resulting from the blast cleaning operations shall be immediately and thoroughly cleaned from the blast-cleaned surfaces and from other areas where debris may have accumulated. The blast cleaned areas shall be protected, as necessary, against contamination prior to placement of the overlay.
Contaminated areas and areas exposed more than 36 hours after cleaning shall be blast cleaned again as directed by the Engineer at the Contractor's expense. Just prior to placement of the overlay, all dust and other debris shall be removed by flushing with water or blowing with compressed air. The prepared surface shall then be soaked with clean water for not less than 1 hour prior to the placement of the latex overlay. Before the overlay is applied, all free water shall be blown out and off, and this procedure shall continue until the surface appears dry or barely damp. The air supply system for blast cleaning and blowing shall be equipped with an oil trap in the air line, and provisions shall be made to prevent oil or grease contamination of the surface by any equipment prior to placement of the overlay. 28.2.3.2 Existing Decks The surface of existing decks that have become contaminated by tr c usage or by deicing salts shall be scarified to the aepth shown on the plans or specified. If no depth is shown or specified, a minimum of Y4 inch of material shall be removed by scarification. Prior to beginning scarification and until operations are completed, all deck drains, expansion joints and other openings where damage could result, as determined by the Engineer, shall be temporarily covered or plugged to prevent entry of debris. Scarifying shall be done with power-operated mechanical scarifiers, or other approved devices, capable of uniformly removing the existing surface to the depths re- 28.2.3.2 DIVISION 11-CONSTRUCTION quired without damaging the underlying concrete. Machine scarifiers shall not be operated so as to damage hardware such as drain grates and expansion joint armor. In areas where machine scarifying cannot reach and in areas of spalling and where steel reinforcement is exposed, scarifying and the removal of deteriorated or unsound concrete shall be accomplished with hand tools.
Pneumatic hammers heavier than nominal 45 pounds shall not be used. No scarrfying or chipping will be allowed within 6 feet of a new overlay until 48 hours after its placement. In areas where deteriorated or unsound concrete is encountered, as determined by the Engineer, the concrete shall be removed to a depth of 3/4-inch below the top mat of reinforcing steel. Aminimum of 3/4-inchclearance shall be required around the reinforcing steel except where lower bar mats make this impractical. Care shall be exercised to prevent damaging the exposed reinforcing steel. All reinforcing steel shall be blast-cleaned. The repair areas are to be filled during the overlay operation. After scarification and removal of unsound concrete has been completed, the deck surface shall be blast cleaned and prepared as specified for new decks. 28.2.4 Proportioning and Mixing The Contractor shall submit to the Engineer for approval, 14 calendar days prior to date of placement, the proposed mix design in writing and samples of all mix materials in sufficient quantity to produce a minimum of 3 cubic feet of concrete for laboratory mix design testing. Proportioning and mixing equipment shall be of a selfcontained, mobile, continuous-mixing, volumetric proportioning type mixer. Continuous-type mixers shall be equipped so that the proportions of the cement, natural sand, and coarse aggregate can be fixed by calibration of the mixer and cannot be changed without destroying a seal or other indicating device affiied to the mixer. In addition to being equipped with a flow meter for calibrating the water supply portion of the mixer, the mixer shall also be equipped with a cumulative-type water meter which can be read to the nearest 0.1 gallon. The water meters shall be readily accessible, accurate to within 1%, and easy to read. Both water meters shall be subject to checking by the Engineer each time the mixer is calibrated. Approved methods for adding the admixture shall be provided. The admixtures shall be added so as to be kept separated as far as is practicable. The continuous type mixer shall be calibrated to the satisfaction of the Engineer prior to starting the work. Yield checks normally will be made for each 50 cubic yards of mix. Recalibration will be necessary when indicated by the yield checks, and at any other times 681 the Engineer deems necessary to ensure proper proportioning of the ingredients. Continuous type mixers which entrap unacceptable volumes of air in the mixture shall not be used. The mixer shall.bekept clean and free of partially dried or hardened materials at all times. It shall consistently produce a uniform, thoroughly blended mixture within the specified air content and slump limits. Malfunctioning mixers shall be immediately repaired or replaced with acceptable units. Aggregate stockpiles being used should be of uniform moisture content. Mixing capability shall be such that finishing operations can proceed at a steady pace with final finishing completed before the formation of the plastic surface film. 28.2.5 Installation 28.2.5.1 Weather Restrictions The placement of latex modified concrete shall not be started when the temperature is, or is expected to fall below 45°F or rise above 80°F, or when high winds, rain or low humidity conditions are expected prior to final set of the concrete. If any of these conditions occur during placement, the placement shall be terminated and a straight construction joint formed. Placement at night may be necessary when daytime conditions are not favorable. If placement is performed at night, adequate lighting shall be provided by the Contractor. 28.2.5.2 Equipment Placing and finishing equipment shall include hand tools for placement and brushing-in freshly mixed latex modified concrete and for distributing it to approximately the correct level for striking-off with the screed. Hand-operated vibrators, screeds and floats shall be used for consolidating and finishing small areas. An approved finishing machine complying with the following requirements shall be used for finishing all large areas of work: The finishing machine shall be self-propelled and capable of forward and reverse movement under positive control. The length of the screed shall be sufficient to extend at least 6 inches beyond the edge of both ends of the section being placed. The finishing machine shall also be capable of consolidating the concrete by vibration and of raising all screeds to clear the concrete for traveling in reverse. The machine shall be either a rotating roller type or an oscillating screed type. 682 HIGHWAY BRIDGES Rotating roller-type machines shall have one or more rollers, augers, and 1,500 to 2,500 vpm vibratory pans. Oscillating screed-type machines shall have vibrators on the screeds whose frequency of vibration can be varied between 3,000 and 15,000 vpm. The bottom face of the screeds shall be not less than 4 inches wide and shall be metal. Rails will be required for the finishing machine to travel on. Rails shall be sufficiently rigid to support the weight of the machine without appreciable deflection and shall be placed outside of the overlay area. Rail anchorages shall provide horizontal and vertical stability and shall not be ballistically shot into concrete that will not be overlaid. A suitable portable lightweight or wheeled work bridge shall be furnished for use behind the finishing operation. 28.2.5.3 Placing and Finishing 28.2.5.3.1 Construction Joints Planned construction joints shall be formed by bulkheads set to grade. Before placing concrete against previously placed overlay material, the construction joint shall be sawed to a straight vertical edge. Sawing of joints may be omitted if the bulkhead produces a straight, smooth, vertical surface. The face of the joint shall be sand or water blasted to remove loose material. Longitudinal construction joints will be permitted only at the center line of roadway or at lane lines unless otherwise shown on the plans or permitted by the Engineer. In case of delay in the placement operation exceeding 1 hour in duration, an approved construction joint shall be formed by removing all material not up to finish grade and sawing the edge in a straight line. During minor delays of 1 hour or less, the end of the placement may be protected from drying with several layers of clean, wet burlap. 28.2.5.3.2 Placing The finishing machine shall be test run over the entire area to be overlayed each day before placement is started to ensure that the required overlay thickness will be achieved. Immediately ahead of placing the overlay mixture, a thin coating of the polymer modified concrete mixture to be used for the overlay shall be thoroughly brushed and scrubbed onto the surface as a grout-bond coat for the overlay. Coarser particles of the mixture which cannot be scrubbed into contact with the surface shall be removed and disposed of in a manner approved by the Engineer. Care shall be taken to insure that all vertical as well as horizontal surfaces receive a thorough, even coating and that 28.2.5.2 the rate of progress is limited so that the material brushed on does not become dry before it is covered with the full depth of latex modified concrete. The latex modified concrete shall be placed on the prepared and grout-coated surface immediately after being mixed. The mixture shall be placed and struck off approximately 1/4 inch above final grade then consolidated by vibration and finished to final grade with the approved finishing machine. Spud vibrators will be required in deep pockets, along edges, and adjacent to joint bulkheads. Supplemental vibration shall be provided along the meet lines where adjacent pours come together and along curb lines. Hand finishing with a float may be required along the edge of the pour or on small areas of repair.
Screed rails and construction bulkheads shall be separated from the newly placed material by passing a pointing trowel along their inside face. Expansion dams shall not be separated from the overlay. Care shall be exercised to ensure that this trowel cut is made for the entire depth and length of rails after the mixture has stiffened sufficiently. 28.2.5.3.3 Finishinn " The finishing equipment shall be operated so as to produce a uniform, smooth, and even-textured surface. The final surface shall not vary more than $4 inch from a 10-foot straightedge placed longitudinally thereon. Before the plastic film forms, the surface shall be textured by tining in accordance with the requirements of Article 8.10.2.3. 28.2.6 Curing The surface shall be promptly covered with a single layer of clean, wet burlap as soon as the surface will support it without deformation. Within 1 how of covering with wet burlap, the burlap shall be rewet if necessary and a layer of 4-mil polyethylene film, or wet burlap-polyethylene sheets, shall be placed on the wet burlap, and the surface cured for 24 hours. The curing material shall then be removed for an additional 72 hours of air cure. If the temperature falls below 45" during curing, the duration of the wet cure shall be extended as directed by the Engineer. The overlay shall be protected from freezing during the cure period. Tr&c will not be permitted on the overlay while it is curing.
28.2.7 Acceptance Testing After curing is completed, the overlay will be visually inspected for cracking or other damage, and inspected for delarninations and bond failures by the use of a chain drag or other suitable device. 28.2.7 DIVISION 1'-CONSTRUCTION Surface cracks not exceeding 3/s inch in depth shall be sealed with an epoxy penetrating sealer followed by an application of approved sand. Any cracks exceeding 3/a inch in depth shall be repaired by methods approved by the Engineer, or the affected portions of the wearing surface shall be removed and replaced. Any delaminated or unbonded portions of the wearing surface or portions damaged by rain or freezing shall be removed and replaced. After completion of the wet cure, the surface shall be tested for flatness and corrected, if necessary, as provided in Article 8.10.2.4. All corrective work will be at the Contractor's expense. 28.2.8 Measurement and Payment Wearing surfaces and areas requiring scarification will be measured by the square foot based on dimensions of the completed work. 683 Wearing surfaces will be paid for at the contract price per square foot. Except as otherwise provided, the payment per square foot for wearing surfaces shall be considered to be full compensation for the cost of furnishing all labor, materials, equipment, incidentals, and for doing all work involved in preparing the surface and constructing the wearing surface as shown on the plans and specified. When a separate item is included in the bid schedule for scarifying bridge decks, scarifying will be paid for by the contract price per square foot.
Such payment shall be considered to be full compensation for all costs involved with the scarifying work including removal and disposal of debris. The removal of unsound concrete which is encountered below the depth specified for scarifying will be paid for as extra work. Section 29 EMBEDMENT ANCHORS 29.1 DESCRIPTION 29.4 CONSTRUCTION METHODS This specification covers installation and field testing of cast-in-place, grouted, adhesive-bonded, expansion and undercut steel anchors. Provide adequate edge distance, embedment depth and spacing to develop the required strength of the embedment anchors. Use the correct drill hole diameter as per manufacturer's instructions. Use rotary impact drilling equipment unless diamond core drilling has been specified and tested. If reinforcing bar is encountered during the drilling operation, move to a different location, or driU through the reinforcing steel using a diamond core bit as directed by the Engineer. Patch abandoned holes with an approved bonding material. Clean holes thoroughly as recommended by the manufacturer. Remove all loose dust and concrete particles from hole. Prepare bonding material and install anchors according to instructions provided by the manufacturer or approved by the Engineer. Embedded anchors which are improperly installed or which do not have the required strength shall be removed and replaced to the satisfaction of the Engineer at the Contractor's expense. 29.2 PREQUALIFICATION Prequalify all concrete anchors, including cast-inplace, all bonded anchor systems (including grout, chemical compounds, and adhesives), and undercut by universal test standards designed to allow approved anchor systems to be employed for any construction attachment use. Conduct test for adhesive-bonded and other bonding compounds in accordance with ASTM E 1512 (Standard Test Methods for Testing Bond Performance of AdhesiveBonded Anchors). Test expansion types to ASTM E 488 (Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements). Comply with ACI 349-85 (Code Requirements for Nuclear Safety Related Concrete Structures-Appendix B, Steel Embedments).
Provide certified test reports prepared by an independent laboratory documenting that the system (except mechanical expansion anchors) is capable of achieving the minimum tensile strength of the embedment steel. 29.3 MATERIALS Provide mill test reports certifying physical properties, chemistry, and strengths. The chemical compounds acceptable for adhesive anchors may include epoxies, polyesters, or vinylesters. Adhesive compounds which are moisture-insensitive, highmodulus, high-strength, and low-shrinkage should be used.
The use of additives to grout, and bonding materials which will be corrosive to steel or zinclcadmium coatings is prohibited. 29.5 INSPECTION AND TESTING Where specified, conduct sacrificial tests of the anchor system on the job site to

304.8 mm = 0.3048 m 1 yd = 914.4 mm = 0.9144 m 1 mi = 1.609 X lo6 mm =I609 m = 1.609 km 1 m = 39.37 in = 3.281 ft = 1.094 yd 1 km = 39.37 x = 3281 ft = 0.6214 mi Area 1 in2 = 645.2 mm2 1 ft2 = 92.90 X lo3 mm2 = 92.90 X 103 d 1 yd2 = 836.1 X lo3 mm2 = 0.8361 d 1 mi2 = 2.590 x lo6 m2 = 2.590 lad 1 acre = 4047 m2 = 4.047 x 10" km2 Area - Volume 1 i n 3 = 16390mm3 1 ft 3 = 28.32 x lo6 mm3 = 28.32 x 10" m3 1 yd = 764.6 x lo6 m3= 0.7646 m3 1 acreaft = 1233 m3 Volume 1 mm3 = 61.02 X lo4 id = 35.31 X 10' ff 1 cm3 = 61.02 X lo9 id = 35.31 X 106 @ 1 m3 = 61.02 x 10j id = 35.31 ft3 = 1.308yd3 SDeed 1 d = 1.550 X 10" id 1 cm2 = 0.1550 in2 1 m2 = 1550 in2 = 10.76 ft2 = 1.196 yd2 1 km2 = 1.550 x lo9 in2 = 10.76 x 106 ft2 1 km2 = 247.1 acre = 0.3863 mi2 SEal 1 inlsec = 25.40 mmls = 0.0254 m/s 1 ftlsec = 304.8 mmls = 0.3048 m/s = 1.097 kmh 1 ftlmin = 5.080 mmls = 5.080 x 103m/s 1 mi/hr = 447.0 mmls = 0.4470 mls = 1.609 kmh 1 mmls = 39.37 x 103inlsec = 3.281 x lo3 ftlsec 1 cm/s = 393.7 x 103inlsec = 32.81 X 10" ftlsec 1 mls = 39.37 inlsec = 3.281 ftlsec = 2.237 mi1h.r 1 kmm = 0.9113 ftlsec = 0.6214 milhr Mass 1 lbm = 0.4536 kg Mass 1 slug = 14.59 kg 1 kg = 2.205 lbm 1 kg = 68.52 X lo3slug Force 1 lb = 4.448 N rn 1 N = 0.2248 lb 1 kip = 4448 N = 4.448 kN 1 ton = 8896 N = 8.896 kN 1 kN = 224.8 lb = 0.2248 kip Moment (or toraue) 1 lbin = 113.0 Nsmm = 0.1130 N-m 1 1b.ft = 1356 N-mm = 1.356 Nm 1 kip-in = 113.0 x = 113.0 N.m 1 kipft = 1.356 x lo6 Nmm = 1356 N-m Moment (or torauel 1b.h = 737.6 X 106 Ib-ft 1 Nmm = 8.851 x 1 N-m = 8.851 lbin = 0.7376 lbsft 1 kN-m = 8851 lb-in = 737.6 lbsft = 0.7376 kipft -p 1 lb/i = 0.1751 Nlmm = 175.1 Nlrn 1 lblft = 0.01459 N/mm = 14.59 Nlm 1 kiplft = 14.59 Nlmm = 14 590 Nlrn 1 in-lblft = 0.3707 N~mmlmm Distributed hadines 1 Nlmm = 5.710 lb/i = 68.52 lblft 1 Nlm = 5.710 X 103lb/m = 68.52 X 1C3 lblft 1 N-mmtmm = 0.2248 lb-idin = 2.698 in.lb/ft 702 HIGHWAY BRIDGES Stress (or uressurer 1 l b / i = 6895 Pa = 6.895 x 10 MPa 1 lblft = 47.88 Pa = 47.88 x 10 -6 MPa 1 kip/in2 = 6.895 x lo6 Pa = 6.895 MPa 1 kip/ft2 = 47.88 x lo3 Pa = 47.88 x 10" MPa 1 ton/ft2 = 95.76 x lo3 Pa = 95.76 x lo3 MPa Stress (or uressure) 1 Pa = 145.0 x lo4 l b / s = 20.89 x 10" lblff 1 MPa = 145.0 1b/in2 = 20.89 x 10' lb/ftZ (1 N/mm2 = 1 MPa) Area of reinforcing steel 1 in2/ft = 2.117 mmz/mm Area of reinforcing steel 1 d / m m = 0.4724 in2/ft APP. E A Note on Standard Units Standard units used in the Standard Specifications for Design of Highway Bridges, Appendix E: (unless otherwise stated in the Notation lists appeiuing at the beginning of each section, these units are used throughout the Appendix). Length: mm (millimeters) Force: N (newtons) Stress and pressure: MPa (megapascals), (note: 1 N/mm2 = 1 MPa) Time: s (seconds) Some noteworthy exceptions to these standards are: w, the unit weight of concrete in the empirical expression for Young's modulus for concrete is to be used in kg/m3, as this is thought to be the most appropriate way for the density of concrete to be reported, and S, the design highway speed in the calculation for centrifugal force, is to be taken in kmlhr. APP. E HIGHWAY BRIDGES 703 Section 2 GENERAL FEATURES OF DESIGN NOTATIONS Ai = area of flange, mm2 (Article 2.7.4.3) b = flange width, mm (Article 2.7.4.3) = modification factor for concentrated load, P, used in the design of rail members (Article 2.7.1.3.1) = clear unsupported distance between flange components, mm (Article 2.7.4.3) = depth of W or I section, mm (Article 2.7.4.3) = allowable axial stress, MPa (Article 2.7.4.3) = allowable bending stress, MPa (Article 2.7.4.2) = allowable shear stress, MPa (Article 2.7.4.2) = minimum yield stress, MPa (Article 2.7.4.2) = axial compression stress, MPa (Article 2.7.4.3) = height of top rail above reference surface, mm (Figure 2.7.4B) = post spacing, mm (Figure 2.7.4B) = railing design loading = 44 500 N (Article 2.7.1.3 and Figure 2.7.4B) = railing design loading equal to P, P/2 or Pl3, N (Article 2.7.1.3.5) = web thickness, mm (Article 2.7.4.3) = pedestrian or bicycle loading, Nlmm (Article 2.7.2.2 and 2.7.3.2) C D d F.

Fb Fv F~ fa h L P P' t w 704 HIGHWAY BRIDGES APP.E APP. E HIGHWAY BRIDGES 705 APP.E HIGHWAY BRIDGES Section 3 LOADS NOTATIONS A a B b b C C C CF cn CM CR D D D D.F. DL E E EQ & ES E, F F Fb F, g I I I ICE J K K K k L L L L LF MD Acceleration coefficient determined in Article 3.2, Division I-A, dimensionless length of short span of slab, mm (Article 3.24.6) buoyancy (Article 3.22) width of pier or diameter of pile, mm (Article 3.18.2.2.4) = length of long span of slab, mm (Article 3.24.6) = combined response coefficient = stiffness parameter = K(W/L), nondimensional (Article 3.23.4.3) = centrifugal force in percent of live load, percentage (Article 3.10.1) = centrifugal force (Article 3.22) = coefficient for nose inclination, nondimensional (Article 3.18.2.2.1) = steel bending stress coefficient, mm3(Article 3.25.1.5) = steel shear stress coefficient, mm-2(Article 3.25.1.5) = parameter used in determination of load fraction of wheel load, mm (Article 3.23.4.3) = degree of curve (Article 3.10.1) = dead load (Article 3.22) = fraction of wheel load applied to beam, nondimensional (Article 3.28.1) = contributing dead load = width of slab over which a wheel load is distributed, mm (Article 3.24.3) = earth pressure (~rticle3.22) = equivalent static horizontal force applied at the center of gravity of the structure = modulus of elasticity of concrete, MPa (Article 3.26.3) = modulus of elasticity of steel, MPa (Article 3.26.3) = modulus of elasticity of wood. MPa (Article 3.26.3) = horizontal ice force on pier, N (Article 3.18.2.2.1) = framing factor (Article 3.21.1.1) = allowable bending stress, MPa (Article 3.25.1.3) = allowable shear stress, MPa (Article 3.25.1.3) = gravitational acceleration = 9810 mm/s2 = impact fraction, nondimensional (Article 3.8.2) = impact load, defined as live load multiplied by impact fraction = gross flexural moment of inertia of the precast member, mm' (Article 3.23.4.3) = ice pressure (Article 3.22) = gross Saint-Venant torsional constant of the precast member, m d (Article 3.23.4.3) = stream flow force constant, N.s2/mm4(Article 3.18.1) = stiffness constant, nondimensional (Article 3.23.4) = wheel load distribution constant for timber flooring, nondimensional (Article 3.25.1.3) = live load distribution constant for spread box girders, n~n~~nensional (Article 3.28.1) = loaded length of span, mm (Article 3.8.2) = loaded length of sidewalk, mm (Article 3.14.1.1) = live load (Article 3.22) = span length, mm (Article 3.23.4) = longitudinal force from live load (Article 3.22) = moment capacity of dowel, N-mm (Article 3.25.1.4) = = = = 707 708 Mx % NB NL n P P P P P P P,,, PI, P P R R R RD R, 5 S S S S S S S S S s SF T T t t V V W W W W we W W WL w X x Z fl Y HIGHWAY BRIDGES APP. E primary bending moment, N.mmlmm (Article 3.25.1.3) total transferred secondary moment, N~mm(Article 3.25.1.4) number of beams, nondimensional &Article 3.28.1) number of traffic lanes, nondimensional (Article 3.23.4) number of dowels, nondimensional (Article 3.25.1.4) live load on sidewalk, (~ l d (Article) 3.14.1.1) stream flow pressure, MPa (Article 3.18.1) total uniform force required to cause unit horizontal deflection of whole structure load on one rear wheel of truck, N (Article 3.24.3) wheel load, N (Article 3.24.5) design wheel load, N (Article 3.25.1.3) 53 400 N (Article 3.24.3) 71 200 N (Article 3.24.3) effective ice strength, MPa (Article 3.18.2.2.1) = proportion of load carried by short span (Article 3.24.6.1) = radius of m e , mm (Article 3.10.1) = normalized rock response = rib shortening (Article 3.22) = shear capacity of dowel, N (Article 3.25.1.4) = primary shear, Nlmm (Article 3.25.1.3) = total secondary shear transferred, N (Article 3.25.1.4) = design speed, km/hr (Article 3.10.1) = soil amplification spectral ratio = shrinkage (Articlg 3.22) = average stringer spacing, mm (Article 3.23.2.3.1) = spacing of beams, mm (Article 3.23.3) = width of precast member, mm (Article 3.23.4.3) = effective span length, mm (Article 3.24.10) = span length, mm (Article 3.24.8.2) = beam spacing, mm (Article 3.28.1) = effective deck span, mm (Article 3.25.1.3) = stream flow (Article 3.22) = period of vibration, s = temperature = thickness of ice, mm (Article 3.22) = deck thickness, mm (Article 325.1.3) = variable spacing of truck axles, mm (Figure 3.7.3A) = velocity of water, mmls (Article 3.18.1) = combined weight on the first two axles of a standard HS Truck, N (Figure 3.7.7A) = width of sidewalk, mm (Article 3.14.1.1) = wind load on structure (Article 3.22) = total dead weight of the structure, N = width of exterior girder, mm (Article 3.23.2.3.2) = overall width of bridge, mm (Article 3.23.4.3) = roadway width between curbs, mm (Article 3.28.1) = wind load on live load (Article 3.22) = width of pier or diameter of circular-shaft pier at the level of ice action, mm (Article 3.18.2.2.1) = distance from load to point of support, mm (Article 3.24.5.1) = subscript denoting d i i t i o n perpendicular to longitudinal stringers (Article 3.24.1.3) = reduction for ductility and risk assessment = (with appropriate script) coefficient applied to actual loads for service load and load factor designs (Article 3.22) = load factor (Article 3.22) = = = = = = = = = = = = = = HIGHWAY BRIDGES APP-E UPL Be Bc BD BE BEa PICE BL BR r6s Bw @WL P = = = = = = = = = = = = = = proportional limit stress perpendicular to grain, MPa (Article 3.25.1.4) load combination coefficient for buoyancy (Article 3.22.1) load combination coefficient for centrifugal force (Article 3.22.1) load combination coefficient for dead load (Article 3.22.1) load combination coefficient for earth pressure (Article 3.22.1) load combination coefficient for earthquake (Article 3.22.1) load combination coefficient for ice (Article 3.22.1) load combination coefficient for live load (Article 3.22.1) load combination coefficient for rib shortening, shrinkage and temperature (Article 3.22.1) load combination coefficient for stream flow (Article 3.22.1) load combination coefficient for wind (Article 3.22.1) load combination coefficient for wind on live load (Article 3.22.1) Poisson's ratio (Article 3.23.4.3) 709 -IN raaau~!~~px zeau!~lad s u o i f i a ~OS'I iooj n a q l lad spunod 001 ?M * I'ZZ"6 a l a l w ' ol-OI x P'E El2 ol-ol x 211 9'2 ol-OI x I ' L t-1zS.N 1'8J'F EdR ; :? 91 81E I)I) (& " + W!W lwO) ' . O m) a! S! [A) [A' OE) d T'PI"€a ~ f w s aaqfi 'd $89'9 'd - 'P'U $06L 3 , , / , > jrfifqapjw ,,. . . 'P'U 000 8E + 'I 000 SI . I > I SZI + 'I 0s > > ,I7 slvn 3 u w uo!ssaidx3 3 p p ~ aiiypy iajarus.xw uo!ssaidx3 k ~ u r o p n 3sn APP.E HIGHWAY BRIDGES 711 712 HIGHWAY BRIDGES APP.E 714 HIGHWAY BRIDGES APP.E APP.E HIGHWAY BRIDGES 715 716 HIGHWAY BRIDGES APP.E APP-E HIGHWAY BRIDGES HIGHWAY BRIDGES APP.

E Section 4 FOUNDATIONS 4.4.2 NOTATIONS fi = unit ultimate compressive strength of concrete as determined by cylinder tests at age of 28 days, MPa (Article 4.5.20.2) 719 720 HIGHWAY BRIDGES APP.E HIGHWAY BRIDGES APP. E 721 Section 4 FOUNDATIONS 4.6.2 NOTATIONS ' The following notations shall apply for the design of drilled shaft foundations in soil and rock: N nondim. = Nondimensional parameter Nc = tip bearing factor to account for large diameter shaft tip, nondim. (Article 4.6.5.1.3) = area of shaft, m2 = area of shaft tip, m2 = tip bearing factor to account for large diameter shaft tip, nondim.

(Article 4.6.5.1.3) = shaft diameter, m (Article 4.6.3) = diameter of enlarged base, m (Article 4.6.3) = least width of shaft group, m (Article 4.6.5.2.4.3) = diameter of rock socket, m (Article 4.6.3) = tip diameter, m (Article 4.6.5.1.3) = uniaxial compressive strength of rock mass, MPa (Article 4.6.5.3.1) = uniaxial compressive strength of intact rock, MPa = shaft length, m (Article 4.6.3) = length of rock socket, m (Article 4.6.3) = elastic modulus of concrete shaft 05 reinforced shaft, MPa = elastic modulus of intact rock, MPa = elastic modulus of rock mass, MPa = factor of safety, nondim. = ultimate load transfer along shaft, MPa (Articles 4.6.5.1.1 and 4.6.5.1.2) = distance from shaft tip to top of weak soil layer, m (Article 4.6.5.2.4.3) = depth interval, nondim. (Articles 4.6.5.1.1 and 4.6.5.1.2) = displacement influence factor for rocksocketed shafts loaded in compression, nondim. (Article 4.6.5.5.2) = displacement influince factor for rocksocketed shafts loaded in uplift, nondim.

(Article 4.6.5.5.2) = standard penetration resistance, blowslm Ni P Q QE ep Qu q, Q (ISR Q, 9~ q, Q, Q, Q,, RQD = standard penetration test blow count corrected for effects of overburden, blowslm = bearing capacity factor, nondim.

(Article 4.6.5.1.3) = number of depth intervals into which shaft is divided for determination of side resistance, nondim. (Articles 4.6.5.1.1 and 4.6.5.1.2) = lateral load on shaft, MN = total axial compression load applied to shaft butt, MN = ultimate unit tip capacity for an equivalent shaft for a group of shafts supported in s m g layer overlying weaker layer, MPa (Article 4.6.5.2.4.3) = ultimate unit tip capacity of an equivalent shaft bearing in weaker underlying soil layer, MPa (Article 4.6.5.2.4.3) = total axial uplift load applied to shaft butt, MN = ultimate unit tip capacity of an equivalent shaft bearing in stronger upper soil layer, MPa (Article 4.6.5.2.4.3) = ultimate side resistance in soil, MN (Articles 4.6.5.1.1 and 4.6.5.1.2) = ultimate unit shear resistance along shafttrock interface, MPa (Article 4.6.5.3.1) = ultimate side resistance of rock socket, MN (Article 4.6.5.3.1) = ultimate unit tip resistance for shafts, MPa (Articles 4.6.5.1.3 and 4.6.5.1.4) = ultimate unit tip resistance for shafts reduced for size effects, MPa (Equations 4.6.5.1.3-3 and 4.6.5.1.4-2) = ultimate tip resistance in soil, MN (Articles 4.6.5.1.3 and 4.6.5.1.4) = ultimate tip reskkmce of rock socket, MN (Article 4.6.5.3.2) = ultimate axial load capacity, MN (Article 4.6.5.1) = Rock Quality Designation, nondim.

Sui = incremental undrained shear strength as a S, = W = zi = (Y = Qli = = fli I = Yi = Azi = = I APP. E HIGHWAY BIUDGES 722 function over ith depth interval, MPa (Article 4.6.5.1.1) undrained shear strength within 2B below shaft tip, MPa (Article 4.6.5.1.2) weight of shaft, MN depth to midpoint of ith interval, m (Article 4.6.5.1.2) adhesion factor, nondim. adhesion factor as a function over ith depth interval, nondim. (Article 4.6.5.1.1) reduction factor to estimate rock mass modulus and uniaxial strength from the modulus and uniaxial strength of intact rock, nondii. (Article 4.6.5.3.1) load transfer factor in the ith interval, nondii. (Article 4.6.5.1.2) effective soil unit weight in the ith interval, MN/m3 (Article 4.6.5.1.2) ith increment of shaft length, m factor to account for reduced individual capacity of closely spaced shafts in group. nondim. (Article 4.6.5.2.4.1) PC = elastic shortening of shaft, m (Articles PI = total settlement displacement at butt for 4.6.5.5.1.1 and 4.6.5.5.1.2) PU = r = u = 0, = 0; = shaft with rock socket, m (Article 4.6.5.5.2) total uplift displacement at butt for shaft with rock socket, m (Equation 4.6.5.5.2) 3.1415.. ., nondim. Poisson's ratio, nondii. unconfined compressive strength of rock mas or concrete, whichever is weaker. MPa (Article 4.6.5.3.1) effective vertical stress at midpoint of ith depth interval, MPa (Article 4.6.5.1.2) .. The notations for dimension units include the following: nondii = nondiiensional; deg = degree. The dimensional units provided with each notation are presented for illustration only to demonstrate a dimensionally correct combination of units for the shaft capacity and settlement procedures presented below. If other units are used, the dimensional correctness of the equations should be confirmed. US Customary Expression Article 4.6.5.1.2 1.5 - 0.135fi Metric Expression Parameter Metric Units :, , x, Bi 1.5 - 0.007 73h n.d. (Z in nun) 8 , 75 blowslft Table 4.6.5.1.4+ penetration resistance , , 9% w , 6 250 blowslm Q !Z 1.20N ST 0.0175N MPa(N is blowslm) 90 ST 4.3 MPa ?rBrDr(0.144%R) QSR rBPrQR MN APP.

E 725 HIGHWAY BRIDGES Section 8 REINFORCED CONCRETE* NOTATIONS = depth of equivalent rectangular stress block, mm (Article 8.16.2.7) = depth of equivalent rectaagular stress block for balanced strain conditions, mm (Article 8.16.4.2.3) = shear span, distance between concentrated load and face of support, mm (Articles 8.15.5.8 and 8.16.6.8) = effective tension area of concrete m u n d ' i g the flexural tension reinforcement and having the same calmid as that reinforcement, divided by the number of bars or wires, mm2; when the flexural reinforcement consists of several bar sizes or wires, the number of bars or wires shall be computed as the total area of reinforcement divided by the area of the largest big or wire used. For calculation purposes, the thickness of clear concrete cover used to compute A shall not be taken greater than 50 mm. (Article 8.16.8.4) = area of an individual bar, mm2 (Article 8.25.1) = area of core of spirally reinforced compression member measured to the outside diameter of the spiral, mm2 (Article 8.18.2.2.2) = area of concrete section resisting shear trausfer, d (Article 8.16.6.4.5) = area of reinforcement in bracket or corbel resisting moment, d (Articles 8.15.5.8 and 8.16.6.8) = gross area of section, mm? = area of shear reinforcement parallel to flexural tension reinforcement, mm2 (Articles 8.15.5.8 and 8.16.6.8) = area of reinforcemint in bracket or corbel resisting tensile force N, (N,), d (Articles 8.15.5.8 and 8.16.6.8) = area of tension reinforcement, d = area of compression reinforcement, mm2 = area of reinforcement to develop compressive strength of overhanging flanges of I- and T-sections, mm2 (Article 8.16.3.3.2) = area of skin reinforcement per unit height in one side face, mm21mm (Article 8.17.2.1.3) = total area of longitudinal reinforcement, mu? (Articles 8.16.4.1.2 and 8.16.4.2.1) = area of shear reinforcement within a distance s, mm2 (Article 8.19.1.2) = area of shear-friction reinforcement, mm2 (Article 8.15.5.4.3) = area of an individual wire to be developed or spliced, mm2 (Articles 8.30.1.2 and 8.30.2) = loaded area, mm2 (Articles 8.15.2.1.3 and 8.16.7.2) = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm? (Articles 8.15.2.1.3 and 8.16.7.2) = width of corhpression face of member, mm = perimeter of critical section for slabs and footings, mm (Articles 8.15.5.6.2 and 8.16.6.6.2) = width of cross section at contact surface being investigated for horizontal shear, mm (Article 8.15.5.5.3) = web width, or diameter of circular section, mm (Article 8.15.5.1.1) = distance from extreme compression fiber to neutral axis, mm (Article 8.16.2.7) = factor relating the actual moment diagram to an equivalent uniform moment diagram (Article 8.16.5.2.7) The specifications of Section 8 are patterned after and arc in general conformity with the provisions of ACI Standard 318 for reinforced concnte design pnd its commentary.

ACI 318 R, published by the American Concrete Institute. 726 d dl d" 4 a, HIGHWAY BFUDGES = distance from extreme compression fiber = = = = h = EI = & = fb = to centroid of tension reinforcement, mm. For computing shear strength of circular sections, d need not be less than the distance from extreme compression fiber to centroid of tension reinforcement in opposite half of member. For computing horizontal shear strength of composite members, d shall be the distance from extreme compression fiber to centroid of tension reinforcement for entire composite section. distance from extreme compression fiber to centroid of compression reinforcement, mm. distance from centroid of gross section, neglecting the reinforcement, to centroid of tension reinforcement, mm nominal diameter of bar or wire, mm. distance measured from extreme tension fiber to center of the closest bar or wire (mrn). For calculation purposes, the thickness of the clear concrete cover used to compute shall not be taken greater than 50 mm (Article 8.16.8.4) modulus of elasticity of concrete, MPa (Article 8.7.1) flexural stiffness of wmpression member, N.mm2 (Article 8.16.5.2.7) modulus of elasticity of reinforcement, MPa (Article 8.7.2) average bearing stress in concrete on loaded area, MPa (Articles 8.15.2.1.3 and 8.16.7.1) fc f'8 = overall thickness of member, mm = height of rolled+n transverse deformations = fmin = fr = fs = MPa on deformed reinforcing bar, mm (Article 4 I , 8.16.8.3) = compression flange thickness of I- and T- sections, mm = moment of inertia of cracked section tramformed to concrete, mm4 (Article 4 8.13.3) = effective It 8.13.3) = moment of inertia of gross concrete section I, k 0, moment of inertia for computation of deflection, mm4 (Article about centroidal axis, neglecting reinforcement, mm' = moment of inertia of reinforcement about centroidal axis of member cross section, mm' = effective length factor for compression nondimensional (Article members, 8.16.5.2.3) = additional embedment length at support or at point of inflection, mm (Article 8.24.2.3) td 01 MPa ft (Articles h h = square root of specified compressive stress = MPa f~ 8.15.2.1.1) = specified compressive strength of concrete, f, balanced conditions, = extreme fiber tensile stress in concrete at service loads, MPa (Article 8.15.2.1.1) = specified yield strength of reinforcement, ft = extreme fiber compressive stress in of concrete, where f,' is expressed in MPa. average splitting tensile strength of lightweight aggregate concrete, MPa fatigue stress range in reinforcement, MPa (Article 8.16.8.3) algebraic minimum stress level in reinforcement, MPa (Article 8.16.8.3) modulus of rupture of concrete, MPa (Article 8.15.2.1.1) tensile stress in reinforcement at service loads, MPa (Article 8.15.2.2) = stress in compression reinforcement at 8.16.3.4.3 and 8.16.4.2.3) concrete at service loads, MPa (Article fC' APP. E = development length, mm (Articles 8.24 through 8.32) = development length of standard hook in tension, measured from critical section to outside end .of hook (straight embedment length between critical section and start of hook (point of tangency plus radius of bend and one bar diameter), mm (Article 8.29) 0tb 0, M = 0 , x applicable modification factor, mm = basic development length of standard hook in tension, mm = unsupported length of compression member, mm (Article 8.16.5.2.1) = computed moment capacity, N-mm (Article 8.24.2.3) M, Mb = maximum moment in member at stage for which deflection is W i g computed, N-mm (Article 8.13.3) = nominal moment strength of a section at balanced strain conditions, Nomm (Article 8.16.4.2.3) APP. E MC Ma M, M, M n ~ Mu M, Mw M,, M, M, n N Nc HIGHWAY BRIDGES = moment to be used for design of compression member, N.mm (Article 8.16.5.2.7) = cracking moment, N-mm (Article 8.13.3) = nominal moment strength of a section, N-mm = nominal moment strength of a section in the direction of the x axis, N-mm (Article 8.16.4.3) = nominal moment strength of a section in the direction of the y axis, N-mm (Article 8.16.4.3) = factored moment at section, N-rnm = factored moment component in the d i i i o n of the x axis, N-mm (Article 8.16.4.3) = factored moment component in the direction of the y axis, Namm (Article 8.16.4.3) = value of smaller end moment on compression member due to gravity loads that result in no appreciable sidesway calculated by conventional elastic frame analysis, always positive, N-mm (Article 8.16.5.2) = value of larger end moment on compression member due to gravity loads that result in appreciable sidesway, always positive, N-mm (Article 8.16.5.2.4) = value of larger end moment on compression member due to lateral loads or gravity loads that result in appreciable sidesway, defined by a deflection A, greater than P,/1500, calculated by conventional elastic frame analysis, always positive, N-mm (Article 8.16.5.2) = modular ratio of elasticity = E,/Ec, nondimensional (Article 8.15.3.4) = design axial load normal to cross section occurring simultaneously with V to be taken as positive for compression, negative for tension and to include the effects of tension due to shrinkage and creep, N (Articles 8.15.5.2.2and 8.15.5.2.3) = design tensile force applied at top of bracket of corbel acting simultaneously with V, to be taken as positive for tension, N (Article 8.15.5.8) N" NW pb PC PO pn pm pw ,P PU r r s SW S V v VC VC Va Vh 727 = factored axial load normal to the cross section occuning simultaneously with V, to be taken as positive for compression, negative for tension, and to include the effects of tension due to shrinkage and creep, N (Article 8.16.6.2.2) = factored tensile force applied at top of bracket or corbel acting simultaneously with Vu, to be taken as positive for tension, N (Article 8.16.6.5) = nominal axial load strength of a section at balanced strain conditions, N (Article 8.16.4.2.3) = critical load, N (Article 8.16.5.2.7) = nominal axial load strength of a section at zero eccentricity, N (Article 8.16.4.2.1) = nominal axial load strength at given eccentricity, N = nominal axial load strength corresponding to M,, with bending considered in the direction of the x axis only, N (Article 8.16.4.3) = nominal axial load strength corresponding to M,, with bending considered in the direction of the y axis only, N (Article 8.16.4.3) = nominal axial load strength with biaxial loading, N (Article 8.16.4.3) = factored axial load at given eccentricity, N = radius of gyration of cross section of a compression member, mm (Article 8.16.5.2.2) = base radius of deformed reinforcing bar, mm (Article 8.16.8.3) = spacing of shear reinforcement in direction parallel to the longitudinal reinforcement, mm (Article 8.19.1.2) = spacing of wires to be developed or spliced, mm (Article 8.30.1.2) = span length, mm = design shear force at section, N (Article 8.15.5.1.1) = design shear stress at section, MPa (Article 8.15.5.1.I) = nominal shear strength provided by concrete, N (Article 8.16.6.1) = permissible shear stress canied by concrete, MPa (Article 8.15.5.2) = design horizontal shear stress at any cross section, MPa (Article 8.15.5.5.3) = permissible horizontal shear stress, MPa (Article 8.15.5.5.3) HIGHWAY BRIDGES 728 v, = nominal shear strength, vah = vs = VU = We = = YI z = a (alpha)= g = (beta)= Be Bd = = N (Article 8.16.6.1) nominal horizontal shear strength, N (Article 8.16.6.5.3) nominal shear strength provided by shear reinforcement, N (Article 8.16.6.1) factored shear force at section, N (Article 8.16.6.1) mass density of concrete, kg1m3 distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, mm (Article 8.13.3) quantity limiting distribution of flexural reinforcement, Nlmm (Article 8.16.8.4) angle between inched shear reinforcement and longitudinal axis of member angle between shear-friction reinforcement and shear plane (Articles 8.15.5.4 and 8.16.6.4) ratio of area of reinforcement cut off to total area of reinforcement at the section, nondimensional (Article 8.24.1.4.2) ratio of long side to short side of concentrated load or reaction area; for a circular cmmamed load or reaction area, 8, = 1.0, nondimensional (Articles 8.15.5.6.3 and 8.16.6.6.2) absolute value of ratio of maximum dead load moment to maximum total load moment, always positive, nondimensional APP. E = ratio of depth of equivalent compression = = = = = = = = = = zone to depth from fiber of maximum compressive strain to the neutral axis, nondimensional (Article 8.16.2.7) c o d 0 1 1factor related to unit weight for concrete (Articles 8.15.5.4 and 8.16.6.4) coefficient of friction, nondimensional (Article 8.15.5.4.3) tension reinforcement ratio = A,/b,d, A,M, nondimensional compression reinforcement ratio = ~:/bd, nondimensional reinforcement ratio producing balanced strain conditions, nondimensional (Article 8.16.3.1.1) ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member, nondimensional (Article 8.18.2.2.2) ninforcement ratio used in Equation (8-4) and Equation (8-48), nondimensional moment magnification factor for members braced against sidesway to reflect effects of member curvature between ends of compression member, nondimensional moment magnification factor for members not braced against sidesway to reflect lateral drift resulting from lateral and gravity loads, nondimensional m g t h reduction factor, nondimensional (Article 8.16.1.2) HIGHWAY BRIDGES 729 730 HIGHWAY BRIDGES APP. 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E HIGHWAY BRIDGES 737 738 HIGHWAY BRIDGES APP.E ADD. E HIGHWAY BRIDGES 739 740 HIGHWAY BRIDGES APP. E APP. E HIGHWAY BRIDGES 741 742 HIGHWAY BRIDGES APP. E HIGHWAY BRIDGES APP-E 743 Section 9 PRESTRESSED CONCRETE NOTATIONS of non-prestressed tension reinforcement, d (Articles 9.7 and 9.19) = area of compression reinforcement, d (Article 9.19) = area of prestressing steel, d (Article 9.17) = steel area required to develop the compressive strength of the overhanging portions of the flange, mm2 (Article 9.17) = steel area required to develop the compressive strength of the web of a flanged section, d (Articles 9.17-9.19) = area of web reinforcement, d (Article 9.20) = width of flange of flanged member or width of rectangular member, mm = width of cross section at the contact surface Wing investigated for horizontal shear, mm (Article 9.20) = width of a web of a flanged member, mm (Article 9.20.2.2) = loss of prestress due to creep of concrete, MPa (Article 9.16) = loss of prestress due to relaxation of prestressing steel. MPa (Article 9.16) = nominal diameter of prestressing steel, mm (Articles 9.17 and 9.27) = distance from extreme compressive fiber to centroid of the prestressing force, or to centroid of negative moment reinforcing for precast girder bridges made continuous, mm = distance from the extreme compressive fiber to the centroid of the non-prestressed tension reinforcement, mm (Articles 9.7 and 9.17-9.19) = modulus of elasticity of concrete at transfer of prestress, MPa = loss of prestress due to elastic shortening, MPa (Article 9.16) = base of Naperian logarithms (Article 9.16) = average concrete compressive stress at = area = = = = = = = = = = = the c.g. of the prestressing steel under full dead load, MPa (Article 9.16) average concrete stress at the c.g. of the presaessing steel at time of release, MPa (Article 9.16) compressive strength of concrete at 28 days, MPa (Article 9.15.2.2) compressive strength of concrete at time of initial prestress, MPa (Article 9.15.2.1) average splitting tensile strength of lightweight aggregate concrete, MPa stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads. MPa (Article 9.20) compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, MPa (In a composite member, f, is resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone) (Article 9.20) compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads, MPa (Article 9.20) guaranteed ultimate tensile strength of the prestressing steel, ~ , * f :, N. the modulus of rupture of concrete, as defined in Article 9.15.2.3, MPa (Article 9.18) total prestress loss, excluding friction, MPa (Article 9.16) effective steel prestress after losses, MPa (Article 9.17.4) average stress in prestressing steel at ultimate load, MPa (Article 9.17.4) APP.

E HIGHWAY BRIDGES = ultimate strength of prestressing steel, = = = = = MPa (Articles 9.15 and 9.17) yield strength of non-prestressed conventional reinforcement in tension, MPa (Articles 9.19 and 9.20) yield strength of non-prestressed conventional reinforcement in compression, MPa (Article 9.19) yield point stress of prestressing steel, MPa (Article 9.15) friction loss stress reduction below the level of 0.70 at the point under consideration, MPa (Article 9.16.2.1.4) overall depth of member, mm (Article 9.20) = moment of inertia about the centroid of the cross section, mm4 (Article 9.20) = friction wobble coefficient per unit length of prestressing steel, mm" (Article 9.16) = distance from end of prestressing strand to center of panel, mm (Article 9.17.4.2) = length of pmtmsing steel element from jack end to point x, mm (Article 9.16) = moment causing flexural cracking at section due to externally applied loads, N-mm (Article 9.20) = min steel cracking moment, N-mm (Article 9.18) = composite dead load moment at the section, N-mm (Commentary to Article 9.18) = noncomposite dead load moment at the section, N-mm (Article 9.18) = maximum factored moment at section = = = = = due to externally applied loads, N-mm (Article 9.20) nominal moment strength of a section, N-mm factored moment at section St$M, , N-mm (Articles 9.17 and 9.18) A,/bd, ratio of non-prestressed tension reinforcement, nondimensional (Articles 9.7 and 9.17-9.19) ratio of prestressing steel, AZM, indi dimensional (Articles 9.17 and 9.19) A~M, ratio of compression reinforcement, nondimensional (Article SH s % SC t extreme fiber of section where the tensile s a s s is caused by externally applied loads, mm3 (Article 9.18) = composite section modulus for the extreme fiber of section where the tensile stress is caused by externally applied loads, mm3 (Article 9.18) = average thickness of the flange of a flanged member, mm (Articles 9.17 and TO 9.18) = steel stress at jacking end, MPa (Article 9.16) Tx = steel stress at any point x, MPa (Article v = permissible horizontal shear stress, MPa (Article 9.20) = nominal shear strength provided by concrete, N (Article 9.20) = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, N (Article 9.20) = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web, N (Article 9.20) = shear force at section due to unfactored dead load, N (Article 9.20) = factored shear force at section due to externally applied loads occuning simultaneously with M, , N (Article 9.16) vc V* VW vd Vi vnb 9.20) = nominal horizontal shear strength, N (Article 9.20) w = vertical component of effective prestress force at section, N (Article 9.20) = nominal shear stffngth provided by shear reinforcement, N (Article 9.20) = factored shear force at section, N (Article 9.20) = mass density of concrete, kg/m3 (Article yt 9.16.2.1.2) = distance from centroidal axis of gross v~ vs vu section, neglecting reinforcement, to extreme fiber in tension, mm (Article 9.19) = factored tendon force, N = statical moment of cross sectional area, above or below the level being investigated for shear, about the centroid, mm3 (Article 9.20) = loss of prestress due to concrete shrinkage, MPa (Article 9.16) = longitudinal spacing of the web reinforcement, mm (Article 9.20) = noncomposite section modulus for the 9.20) Cc = friction curvature coefficient, nondimensional (Article 9.16) APP.

E HIGHWAY BRIDGES a = total angular change of prestresskg steelprofile from jacking end to point x, 81 = factor for concrete strength, as defined in * = factor for type of prestressing steel radians (Article 9.16) 7 Article 8.16.2.7 (Articles 9.17-9.19) (Article 9.17) = 0.28 for low-relaxation steel = 0.40 for stress-relieved steel = 0.55 for b

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biaxial loading on hollow sections in his design. Commentary to Section 10 Overview Article 10.2.3 has been revised to correspond with the rewrite of Article 14.6.1.2. Articles 10.2.6.4 and 10.32.4.2 have been deleted and replaced by the rewrite of Articles 14.6.7.1 and 14.6.1.4, respectively. The proposed changes affect the capacity of girders with longitudinally stiffened webs, and the capacity of girder webs with and without longitudinal stiffeners during construction. In addition, a separate section on constructibility has been added to the LFD portion of the specification to put all the constructibility requirements in one section. C-10 HIGHWAY BRIDGES Other than the changes to the web buckling requirements, this new section contains the same provisions as previously contained in various parts of the specification. The present specification for longitudinally stiffened webs is based upon the performance of symmetrical girders. The buckling coefficient used to set the web thickness requirements is based upon a web with the neutral axis at mid-depth and a longitudinal stiffener located 0.2 X web depth @). This stiffener location is the optimum location for a symmetrical girder. The buckling mode of the web is a buckle in panel above and below the stiffener. If the stiffener is placed in a lower position in the web, the top panel alone buckles at a lower stress due to the increase in its slenderness. If the stiffener is placed at a higher position in the web, the buckle forms in the panel below the stiffener also at a lower stress.
A similar behavior occurs in an unsymmetrical girder.
The neutral axis is not a mid-depth in an unsymmetrical girder. Consequently, a stiffener placed at 0.2D in an unsymmetrical girder is not at the optimum location and will result in a lower web buckling capacity. The proposed changes to the specification provide a method to calculate the capacity of the web as a function of the longitudinal stiffener location with respect to the web depth in compression.The equations are based upon a finite element study of webs contained in reference 1. The design of longitudinally stiffened composite plate girders is more complex.
The steel section normally used is unsymmetrical and often has various size flange plates along its length. The location of the neutral axis varies along the length and consequently the optimum location of the longitudinal stiffener varies. The neutral axis then shifts when load is applied to the composite section with a hardened composite slab. In the positive moment region, the neutral axis shifts upward reducing the web depth in compression and increasing the webs buckling capacity. In the negative moment region at a pier, the web depth in compression increases. The noncomposite construction loading normally controls the web capacity in the positive moment region and the composite section controls in negative bending. In positive-momentregions, D, of the composite section increases with increasing span length because of the increasing dead to live ratio. As a result, using D, of the short-term composite section, as has been the customary practice in the past, is unconservative.
Thus, it is stated that in positive-moment regions, the value of D, shall be calculated by summing the stresses due the appropriate loadings acting on the respective cross sections supporting the loading. In negative-bending regions of composite sections using D, of the composite section consisting of the steel section plus the longitudinal reinforcement is conservative; thus, computing D, by summing stresses from the various stages of loading is not necessarily required. Development of Proposed Changes The development of the proposed changes is presented below. The details of the development of the buckling coefficients is given in reference 1. The buckling stress of a steel plate, E = 29,000,000 psi and Poisson's ratio = 0.3, is given by the equation below: The buckling coefficient, k, relates the geometry of the plate and loading to its buckling stress, F,,. The buckling coefficient for the web subjected to inplane bending stresses is a function of the boundary conditions assumed at the boundaries of the web. The present AASHTO and AISC specification are based upon partial rotational restraint of the web by the flange and simply supported conditions at the transverse stiffeners. The buckling of a web with a longitudinal stiffener is not effected as much by the restraint offered by the flange since the flange restraint does not change the capacity of the web panel below the stiffener.
The buckling coefficients for an unstiffened and longitudinally stiffened web are given below. Unstiffened Web with Partial Rotational Restraint from Flange Longitudinally Stiffened Web Where D = web depth D, = web depth in compression d, = distance from bottom of compression flange to centerline of longitudinal stiffener.
1997 COMMENTARY Two buckling coefficients are listed for the longitudinally stiffened web. The two equations give the buckling coefficient when the stiffener is above or below the optimum location. When d, is less than 0.4 D,, the stiffener is above the optimum location and the buckling occurs in the lower panel. When d, is greater than 0.4 D,, the panel between the stiffener and the flange buckles. When d, = 0.4 D,, the stiffener is at the optimum location and buckling occurs in both panels and both equations give the same value, 129, for the buckling coefficient. The buckling coefficient for webs without a longitudinal stiffener is calculated using the buckling coefficient assuming partial restraint from the flanges. An a value of 1.3 is used with this buckling coefficient when checking a girder without a longitudinal stiffener. The a offsets the load factor of 1.3 used for dead load. The result is that webs without longitudinal stiffeners are checked against local web buckling during construction without a load factor as a serviceability condition and for their post buckling strength with a load factor in Article 10.48.4.1 using the Rbstrength reduction factor. These buckling coefficients are used to determine the capacity of the girders during construction and maximum load in ASD. The maximum load capacity of the girders in LFD is based upon their postbuckling strength.The postbuckling strength is given as before as RbM, where & is the postbuckling strength reduction due to the shedding of the stress from the buckled web to the flange. The post buckling strength of webs with and without longitudinal stiffeners is the same since the longitudinal stiffener is not adequate to resist the lateral deformation of the web after buckling. Specific Commentary of the Proposed Changes C-11 The wording is changed to clarify that the limits apply only to symmetrical girders. Section reworded to give guidance to the designer in selecting the stiffener location which will allow the thinnest web.
An equation is provided for the positive moment section stiffener location. Required thickness of stiffener written as a function of the compression flange yield stress to insure the stiffener is adequate to develop the yield strength of the section. The equation proposed gives the same value as the present equation when fb = 0.55 Fy. Reference to LFD eliminated. Section refers only to ASD provisions to eliminate confusion. See also C10.38.1 (1999). The strength of girders with or without longitudinally stiffened webs which have a web buckling stress less than the yield strength are based upon the postbuckling capacity Rbw. See also'C10.48.4.1 (1999). Footnote b of Article 10.48.1 Revised to refer to new Article 10.61.l. Equation for required web slenderness written as a function of stiffener location and depth of the web in compression. The proposed equation gives the same value as the present specification when D, = Dl2 (a symmetrical girder), and d, = 0.4 D, = 0.2 D. See also C10.34.3.2.1 (1999). Figure 10.34.1A Axis title changed to reflect that the figure is only applicable to a symmetric girder with a stiffener at the optimum location. Web limits in this section apply only to symmetrical girders.
See also C10.48.6.1(1999). Section reworded to give guidance to the designer in selecting the stiffener location which will allow the thinnest web. An equation is provided for the positive moment section stiffener location. See also C10.49.3.2 (1999). C10.50 The values for the limit are shown to three significant figures to match other similar tables in the specification. Clarified wording to indicate that the value of D, should be calculated by summing the stresses on the appropriate C-12 HIGHWAY BRIDGES sections in positive-moment regions. Constructibility requirements removed and section (d) added to direct the engineer to the new constructibility section 10.61. its, etc. However, the equations better lend themselves to direct computations. Commentary to Section 14-Bearings C14.1 General All provisions during construction are collected into this one section to eliminate confusion in the specification. The major change proposed is the limit on web stresses to the web buckling stress during construction. The present specification allowed the bending stress to exceed the buckling limit with no limitation. See also C10.61 (1999). References 1. Frank, K.H. and Helwig, T.A. "Buckling of Webs in Unsymmetric Plate Girders," Engineering Journal, AISC, Vol. 32, No. 2, 1995, pp. 43-53. Commentary to Section 12--Soil-CorrugatedMetal Structure Interaction Systems C12.4.1.4 and C12.6.1.4 The reduction of flexibility factors for some aluminum pipes effectively increases the required minimum section properties. The change has been accepted by ASTM and is needed because the current flexibility factors allow too light a gauge to be used for some pipe corrugations. This draft specification is an allowablestress design conversion of the draft LRFD Specification developed as part of the NCHRP 10-20A research program. The reader is referred to the Final Report of this research project for details and rationale behind the provisions in this specification. Bridge bearings must allow movements due to temperature changes, creep and shrinkage, elastic shortening due to prestressing, traffic loading, construction tolerances or other effects. If these movements are restrained, large horizontal forces may result. If the bridge deck is cast in place concrete, the bearings in a single support should permit transverse expansion and contraction. In order to permit such movements to take place freely, externally applied transverse loads such as wind, earthquake, or traffic braking forces should be carried either on a small number of bearings near the centerline of the bridge or by an independent guide system. The latter is likely to be needed if the horizontal forces are large.
Distribution of vertical load among bearings may adversely affect individual bearings. This is particularly critical when the girders are stiff in bending and torsion and bearings are stiff in compression and the construction method does not allow minor misalignments to be corrected. C14.2 Definitions Several changes are made to the current specification in order to provide a more complete specification in the areas of design of foundation, design of the backfill envelope, and end treatment design. Similar changes are made in the corresponding sections of the LRFD specification which are completely rewritten to improve its organization. For more background on changes made, see the LRFD commentary. Changes here are to clarify where dimensions should be measured to and remove ambiguity when a box culvert is installed in a keyway on a concrete footing. These changes replace the current tables of factored dead and live load moments with the original, unfactored live and dead load moment equations used to generate them. The change does not alter final designs, design lim- C14.3 Notation C14.4 Movements and Loads Bridge movements arise from a number of different causes. The direction and magnitude of these movements must be accurately estimated to determine the design requirements for the joint or bearing. Simplified estimates of bridge movements, particularly on bridges with complex geometry, may sometimes lead to improper estimation of the direction of motion and, as a result, an improper selection of the bearing or joint system. Curved bridges and skewed bridges may have transverse as well as longitudinal movement due to temperature effects and creep or shrinkage. Rotations caused by permissible levels of misalignment during installation must also be allowed for, and in many cases these will be larger than the live load rotations. Transverse movement of the superstructure may become significant for very wide bridges. 1997 COMMENTARY A significant portion of the design rotation may be caused by construction loads, camber of beams, out of level supports, and temporary load conditions.
The bridge girders are often more flexible during construction since the stiffness of the deck slab is not available. Further, there may be severe and unusual concentrations of loading due to the sequence of deck slab placement and construction equipment. The rotation will sometimes be maximum due to the construction effects, and these rotations must be considered in the design process. The neutral axis of a girder which acts compositely with its bridge deck is typically close to the underside of the deck. As a result, the neutral axis of the beam and the center of rotation of the bearing seldom coincide. Under these conditions, end rotation of the girder induces either horizontal movements or forces at the bottom flange or bearing level. The failure of bridge bearings or joint seals is frequently a serviceability failure which may ultimately lead to deterioration or damage to the bridge. This damage may be very expensive to repair. As a result, some of the design limitations are based on serviceability limits rather than strength or resistance. Each bearing should be clearly identified in design documents and the requirements should be identified in Figure 14.4. Bearings must accommodatemovements in addition to supporting loads, so factored displacements, and in particular factored rotations, are needed for design. Live load rotations are typically less than 0.005 RAD, but the total rotation due to fabrication and setting tolerances for seats, bearings, and girders may be significantly larger than this. Therefore, the rotation to be used in the design is defined by adding to the dead and live load rotations allowances for such tolerances. An owner may reduce the fabrication and setting tolerance allowances if justified by a suitable quality control plan, and therefore these tolerance limits are stated as recommendations rather than absolute limits. Rotations are considered at the service and factored load states. Metal or concrete components are susceptible to damage under a single rotation that causes metal to metal contact, and so they must be designed using the maximum rotation with a very low risk of over revolution. As a result, Om is (OL + OD + 0.02) RAD for these bearings. Failure of deformable components such as elastomeric bearings is generally governed by a gradual deterioration under many cycles of load rather than sudden failure under a single load application. As a result, Om for elastomeric pads and steel reinforced elastomeric bearings is then (OL + OD 0.005) RAD. The difference in + C-13 procedure is not intended to encourage excess rotation of these bearings, instead it avoids practical problems, since temporary local uplift caused by light load and a large rotation and less serious limit states might unreasonably govern the design of these more flexible bearings. C14.5 General Requirements for Bearings Bearings support relatively large loads while accommodating large movements or rotations. The resistances of the bearings as described in this specification are often based on judgement and experience, but they are generally thought to be conservative. C14.5.1 Load and Movement Capabilities It is important that the loads and movements be properly defined in both magnitude and direction, and that the proper bearing system be selected to accommodate them. C14.5.2 Characteristics Table 14.5.2-1 indicates the suitability of individual bearing components for different functions. Practical bearings will often consist of several components in order to fulfill multiple functions. For example, a pot bearing may be combined with a PTFE sliding surface to permit translation and rotation. ' Bearing systems differ in their ability to support these loads and movements. Table 14.5.2-1 is a guide for establishing these capabilities. The ratings are based on general judgement and observation, and there will obviously be a few exceptions to the information listed, for which the engineer must use his judgement. Bearings which are listed as suitable for a specific application are likely to be suitable with little or no effort on the part of the Engineer other than good design and detailing practice. Bearings which are listed as unsuitable are likely to be marginal even if the Engineer makes extraordinary efforts to make the bearing work properly. Bearings which are listed as suitable for limited application may work if the load and rotation requirements are not excessive. C14.5.3 Forces in the Structure Caused by Restraint of Movement at the Bearing Restraint of movement results in a corresponding force or moment in the structure.
The forces should be calculated taking into account the flexibilities of the bridge and the bearing. The latter should be estimated by the methods outlined in Article 14.6. In some cases, the bearing resistance depends on time and temperature, as well as on the movement. C-14 HIGHWAY BRIDGES C14.5.3.2 Bending Moment C14.6.1.3 Geometric Requirements The moment for curved sliding bearings is caused by friction resistance at the curved surface, and it acts about the center of the curved surface. The moment imposed on individual components of the bridge structure may be different, depending on the location of the axis of rotation, and must be calculated by a rational method.
The addition of a flat sliding surface in addition to the curved surface increases the rotational resistance as noted in NCHRP 10-20A. The load-deflection curve of an elastomeric bearing is nonlinear, so Ec is load-dependent. However, an acceptable constant approximation is A cylindrical roller is in neutral equilibrium. The provisions for bearings with two curved surfaces achieve at least neutral, if not stable, equilibrium. The choice of radius for a curved surface is a compromise: a large radius results in low contact stresses, but large rotations of the point of contact, and vice versa. The latter could be important if, for example, a rotational bearing is surmounted by a PTFE slider, since the PTFE is sensitive to eccentric loading. If pintles or gear mechanisms are used to guide the bearing, their geometry shall be such as to permit free movement of the bearing. C14.6.1.4 Contact Stresses where S is the shape factor and G is the shear modulus of the elastomer. C14.6 Special Design Provisions for Bearings C14.6.1 Metal Rocker and Roller Bearings C14.6.1.1 General Design Considerations Cylindrical bearings contain no deformable parts and so are susceptible to damage if the superstructure rotates about an axis perpendicular to the axis of the bearing. Thus they are unsuitable for bridges in which the axis of rotation may vary significantly under different loadings, such as bridges with a large skew. They are also unsuitable for use in seismic regions because the transverse shear caused by earthquake loading can cause substantial overturning moment. Good maintenance is essential if mechanical bearings are to perform properly. Dirt attracts and holds moisture, which, combined with locally high contact stresses, can promote stress corrosion.
Metal bearings, in particular, must be designed for easy maintenance. C14.6.1.2 Materials Carbon steel has been the traditional steel used in mechanical bearings because of its good mechanical properties. Surface hardening may be considered. Corrosion resistance is also important. The use of stainless steel for the contact surfaces may prove economical when life-cycle costs are co-nsidered.Weathering steels should be used with caution as their resistance to corrosion is often significantly reduced by mechanical wear at the surface. The compressive loads are limited so that the maximum shear stress is maintained below the shear yield stress and maximum compressive stress is below compressive yield with appropriatefactors of safety. The maximum compressive stress is at the surface, and the maximum shear stress occurs just below it. The two diameters have the same sign if the centers of the two curved surfaces in contact are on the same side of the contact plane, such as is the case when a circular shaft fits in a circular hole. The formulas are derived from the theoretical value for contact stress between elastic bodies, Roark and Young (1976). They are based on the assumption that the width of the contact area is much less than the diameter of the curved surface. If two surfaces have curves of the opposite sign, the value of D2 is negative. This would be an unusual situation in bridge bearings. C14.6.2 PTFE Sliding Surfaces PTFE is also known as TFEand is commonly used in bridge bearings in the United States. This article does not cover guides. The friction requirements for guides are less stringent and a wider variety of materials and fabrication methods can be used for them. C14.6.2.1 PTFE Surface PTFE may be provided in sheets or in mats woven from fibers. The sheets may be filled with reinforcing fibers to reduce creep, i.e., cold flow, and wear, or they may be made from pure resin. The friction coefficient depends on many factors, such as sliding speed, contact pressure, lubrication, temperature, and properties such as ' 1997 COMMENTARY the finish of the mating surface, Campbell and Kong (1987). The material properties which influence the friction are less well understood, but the crystalline structure of the PTFE is known to be important, and it is strongly affected by the quality control exercised during the sintering process. C14.6.2.2 Mating Surface Stainless steel is the most commonly used mating surface for PTFE sliding surfaces. Anodized aluminum is sometimes used in spherical and cylindrical bearings. The finish of this mating surface is also extremely important, since it affects the coefficient of friction. ASTM A 240, Type 304, stainless, with a surface finish of 16 yIN RMS or better, is appropriate, but the surface measurements are inherently inexact. The NCHRP 10-20A research showed that woven PTFE may achieve lower coefficients of friction with a slightly rougher surface, and as a result different surfaces are permitted if substantiated by test results. Friction testing is required for all types of PTFE and its mating surface because of the many variables involved.
C14.6.2.3 Minimum Thickness Requirements C14.6.2.3.1 PTFE A minimum thickness is specified to ensure uniform bearing and to allow for wear. During the first few cycles of movement, small amounts of PTFE transfer to the mating surface and contribute to the very low friction achieved subsequently. This wear is acceptable and desirable. Wear of PTFE continues with time. Campbell and Kong (1987), and movement and is exacerbated by deteriorated or rough surfaces. Wear is undesirablebecause it usually causes higher friction and it reduces the thickness of the remaining PTFE. Unlubricated, flat PTFE wears more severely than the lubricated material. The evidence on the rate of wear is tentative. High travel speeds, such as those associated with traffic movements, appear to be more damaging than the slow ones due to thermal movements. However they may be avoided by placing the sliding surface on an elastomeric bearing which will absorb small longitudinal movements. No further allowance for wear is made in this specification due to the limited research available to quantify or estimate the wear as a function of time and travel. However, wear may ultimately cause the need for replacement of the PTFE, so it is wise to allow for future replacement in the original design. C-15 C14.6.2.3.2 Stainless Steel Mating Surface The minimum thickness requirements for the mating surface are included primarily to prevent its wrinkling or buckling. This surface is usually quite thin to minimize the cost of the highly finished mating surface. Some mating surfaces, particularly with curved surfaces, are made of carbon steel on which a stainless steel weld is deposited. This welded surface is then machined and polished to achieve the desired finish. C14.6.2.4 Contact Pressure The contact pressure must be limited to prevent excessive creep or plastic flow of the PTFE, which causes the PTFE disc to expand laterally under compressive stress and may contribute to separation or bond failure. The lateral expansion is controlled by recessing the PTFE into a steel plate or by reinforcing the PTFE, but there are adverse consequences with both methods. Edge loading may be particularly detrimental because it causes large stress and potential flow in a local area near the edge of the material on hard contact between steel surfaces. The actual values of the contact pressure are in appropriate proportions to one another relative to the best available research knowledge at this time, but the actual numbers are subject to'adjustment as better data become available. These numbers are in the lower range of those used in Europe. C14.6.2.5 Coefficient of Friction The friction factor decreases with lubrication and increasing contact stress, but increases with sliding velocity, Campbell and Kong (1987). The coefficient of friction also tends to increase at low temperatures. Static friction is larger than dynamic friction, and the dynamic coefficient of friction is larger for the &st cycle of movement than it is for later cycles. Friction increases with increasing roughness of the mating surface and decreasing temperature.The friction factors used in the earlier AASHTO Specification are suitable for use with dimpled, lubricated PTFE.They are much too small for the flat, dry PTFE commonly used in the US. This specification is changed to recognize this fact. The coefficients provided in Table 14.6.2.5-1 are based upon the results of experimental research performed as part of the NCHRP10-20Aresearch program. The specified friction values are intended to be smaller than the friction that may be expected for static breakaway in the initial cycle of slip and larger than the maximum friction achieved in later cycles. The coefficients of friction given in Table 14.6.2.5-1 are not applicable to high velocity movements such as those occurring in seismic events. High velocity C-16 HIGHWAY BRIDGES coefficients are included in the AASHTO "Guide Specifications for Seismic Isolation Design." Coefficients are provided for lower temperatures based on the experimental results, but these data are less accurate than room temperature data since they are extrapolated from limited experimental results. Coefficients of friction, somewhat smaller than those given in Table 14.6.2.5-1, are possible with care and quality control.
Certification testing from the production lot is essential for PTFE sliding surfaces primarily to insure that the friction actually achieved in the bearing is appropriate for the bearing design. Testing is the only reliable method for certifying the coefficient of friction and bearing behavior. Contaminationof the sliding surface with dirt and dust increases the coefficient of friction and increases the wear of the PTFE. To prevent contamination, the bearing should be sealed by the manufacturer and not separated at the construction site. To prevent contamination and gouging of the PTFE, the stainless steel should normally be on top and should be larger than the PTFE, plus its maximum travel. Woven PTFE is sometimes formed by weaving pure PTFE strands with a reinforcing material. These reinforcing strands may increase the resistance to creep and cold flow, and they can be woven so that reinforcing strands do not appear on the sliding surface. This separation is necessary if the coefficients of friction provided in Table 14.6.2.5-1 are to be used. C14.6.2.6.2 Mating Surface The restrictions on the attachment of the mating surface are primarily intended to assure that the surface is flat and retains uniform contact with the PTFE at all times, without adversely affecting the friction of the surface or gouging or cutting the PTFE. The mating surface of curved sliding surfaces must be machined to the required surface finish from a single piece. C14.6.3 Bearings with Curved Sliding Surfaces These provisions are directed primarily toward spherical or cylindrical bearings with bronze or PTFE sliding surfaces. C14.6.3.1 Geometric Requirements C14.6.2.6 Attachment The geometry of a spherical bearing controls its ability to resist lateral loads, its moment-rotation behavior, and its frictional characteristics.The geometry is relatively easy to define, but it has some consequences which are not widely appreciated.
The stress may vary over the contact surface of spherical or cylindrical bearings. Cylindrical and spherical surfaces cannot be machined as accurately as a flat smooth surface. It is important that the radius of the convex and concave surfaces be within appropriate limits. If these limits are exceeded the bronze may crack due to hard bearing contact or there may be excessive wear and damage due to creep or cold flow of the PTFE. The stress limits used in this article are based on average contact stress levels. C14.6.2.6.1 PTFE C14.6.3.2 Lateral Load Capacity Recessing is the most effective way of preventing creep in unfilled PTFE and it is required here. The PTFE discs may also be bonded into the recess, but this is optional and the benefits are debatable. Bonding helps to retain the PTFE in the recess during the service life of the bridge, but it makes replacement of the disc more difficult. If the adhesive is not applied uniformly it can cause an uneven PTFE sliding surface which could lead to premature wear. Some manufacturers cut the PTFE slightly oversize and pre-cool it before installation, since this results in a tighter fit at room temperature. Sometimes, PTFE is bonded to the top cover layer of an elastomeric bearing.
This layer should be relatively thick and hard to avoid rippling of the PTFE; Roeder, Stanton, and Taylor (1987). PTFE must be etched prior to epoxy bonding in order to obtain good adhesion. However, ultra-violet light attacks the etching and can lead to delamination, so PTFEexposed to ultra-violet light should not be attached by bonding alone. The geometry of a curved bearing combined with gravity loads can provide considerable resistance to lateral load. An external restraint is often a more reliable method of resisting large lateral loads. C14.6.4 Pot Bearings C14.6.4.2 Materials Softer elastomers pennit rotation more readily and so are preferred. Corrosion resistant steels, such as AASHTO M 270 grade 50W are not recommended for applications where they may come into contact with salt water or be permanently damp, unless their whole surface is completely corrosion-protected, Most pot bearings are machined from a solid plate, so use of high strength steel to decrease the wall thickness results in only a very small reduction in volume of material used.
1997 COMMENTARY Other properties, such as corrosion resistance, ease of machining, electrochemicalcompatibility with steel girders, availability, and price should also be considered. The choice of specifications for brass simply reflects present practice. C-17 The vertical clearance between top of piston and top of pot wall, h,, may be determined from h,, r 0.5Dp0, + 2.08, + l/s (IN) (C14.6.4.3-2) where C14.6.4.3 Geometric Requirements 8, = maximum vertical load deflection (IN) D, = pot internal diameter (IN) The requirements of this article are intended to prevent the seal from escaping and the bearing from locking up even under the most adverse conditions. Use of the design rotation, Om, means that the designer must take into account both the anticipated movements due to loads and those due to iabrication and installation tolerances, including the rotation imposed on the bearing due to out-of-level of other bridge components, such as undersidesof prefabricated girders, and permissible misalignments during construction. Vertical deflection caused by compressive load should also be taken into account, because it will reduce the available clearance. Anchor bolts projecting above the base plate should be taken into consideration when clearance is checked. Rotation capacity can be increased by using a deeper pot, a thicker elastomeric pad, and a larger vertical clearance between the pot wall and the piston or slider. The minimum thickness of the pad specified herein results in edge deflections due to rotation no greater than 15% of the nominal pad thickness. Figure C14.6.4.3-1 and Equations (C14.6.4.3-1) and ((214.6.4.3-2) may be used to verify that the major components will have adequate clearance. The pot cavity depth hplmay be determined from hpl a (0'5Dp0,83 hr + + Note that the Equation (C14.6.4.3-1) does not contain any allowance for vertical deflection, 6,. This omission is conservative because Equation (C14.6.4.3-1) addresses the possibility of the piston lifting clear of the pot, and compressive deflection would inhibit this behavior. Research performed as part of the NCHRP 10-20A Research Program has shown that thicker elastomeric pads are preferable to thinner pads. Thicker pads with deeper pots cause smaller strains in the elastomer, and they appear to experience less wear and abrasion. Recessing of the rings into the pad is necessary for satisfactory pad performance, but it also decreases the effective thickness of the pad at that location. Further, the recess has sometimes been cut into the pad, and this cut appears to make the pad susceptible to additional damage. Therefore it is generally better to use a deeper pot and thicker pad even though this leads to greater material and machining cost. C14.6.4.4 The average stress on the elastomeric disc is largely limited by the seal's ability to prevent escape of the elastomer. The 3.5 ksi level has been used as a practical upper limit for some years and most bearings have performed satisfactorily, but a few seal failures have occurred.
The experimental research of NCHRP 10-20A showed that greater wear and abrasion due to cyclic rotation occurred hw 'C14'6'4'3-1) where h, = thickness of elastomeric pad (IN) hw = height from underside of piston to top of rim which contacts pot wall (IN) Flat sealing rings Elastomeric Disc DD Circular sealing ring FIGURE C14.6.4.3-1 Pot Bearing-Critical Dimensions for Clearances HIGHWAY BRIDGES when higher stress levels are employed, but this correlation is not strong.
As a result, the 3.5 ksi stress limit is retained as a practical design limit. Lubrication helps prevent abrasion of the elastomer during cyclic rotation, however research has shown that the beneficial effect of the lubrication tends to be lost with time. Silicon grease has been used with success. It performed well in experiments and is recommended. Thin sheets of PTFE have also been used.
These sheets performed quite well in experimental studies, but they are less highly recommended since there is a concern that they may wrinkle and become ineffective. Powdered graphite has been used but did not perform well in rotation experiments.As a result, silicon grease is the preferred lubricant and powdered graphite is not recommended. PTFE disks are permitted as a method of lubrication, but the user should be aware that some problems have been reported. C14.6.4.5 Sealing Rings Failure of seals has been one of the most common problems in pot bearings. Multiple flat brass rings, circular brass rod formed and brazed into a ring, and proprietary plastic rings have been found to be successful. Experimental research suggests that solid circular brass rings provide a tight fit and prevent leakage of the elastomer, but they experience severe wear during cyclic rotation. Experiments suggest that flat brass rings are somewhat more susceptible to elastomer leakage and fracture, but they are less prone toward wear.
PTFE rings should not be used. The rings should preferably be recessed into the elastomer or vulcanized to it in order to minimize distortion of the elastomer. Cyclic rotation of the bearing due to temperature variations or traffic loading can cause chafing of the elastomer against the pot wall, which can give rise to some loss of elastomer past the seal. The detailed design of the sealing system is important in preventing this. The details of the tests for alternate sealing systems are left to the discretion of the Engineer. However, tests should include cyclic rotation. ing must be assured by appropriate inspection. The finished inside profile of the pot must satisfy the required shape and tolerances. Straightening and machining may be needed to rectify welding distortions. The lower bounds on the thickness of the baseplate are intended to provide some rigidity to counteract the effects of uneven bearing. If the base plate were to deform significantly, the volume of the elastomer would be inadequate to fill the space in the pot and hard contact could occur between some components. The minimum wall thickness criterion for unguided pots is based on hoop strength. If the pot is guided or fixed, horizontal forces will occur, and the wall must be thicker than the minimum given here, as required by Article 14.6.4.8. The surface finish on the inside of the pot may have considerable impact on the bearing performance.
A smooth finish reduces rotational resistance and wear and abrasion of the elastomer. It probably also improves the performance of the sealing rings, but at present there are no definitive limits as to what the surface finish should ideally be for good bearing performance. Metallization on the inside of the pot tends to cause a rougher surface finish, which leads to significant increases in damage under cyclic rotation, and as a result metallization may not be a good method of protection. C14.6.4.7 Piston The required piston thickness is partly controlled by rigidity and strength. A central internal guide bar fitted in a slot in the piston causes bending moments which are largest where the piston is weakest. In this case the piston must also be thick enough to supply an adequate grip length for any bolts used to secure the guidebar. The clearance between piston and pot is critical to the proper functioning of the bearing. In most bearings the finished value, after anti-corrosioncoatings have been applied, should be about 0.02-0.04-IN, arange which is easily achievable. The equation for minimum clearance is based on geometry. C14.6.4.8 Lateral Loads C14.6.4.6 Pot Pots are constructed most reliably by machining from a single piece of plate. For very large bearings, this may become prohibitively expensive, so fabrication by welding a ring to a base plate is implicitly accepted. However the ring must be welded to the plate by a full penetration weld, because the wall is subject to significant bending moments where it joins the base plate.
The quality of weld- If the piston rotates while a horizontal load is acting, the piston rim will be subject to bearing stresses due to horizontal load and to shear forces. If the rim surface is cylindrical, contact between it and the pot wall will theoretically be along a line when the piston rotates. In practice, some localized yielding is then inevitable. Experiments have shown that the increased rotational resistance caused by this contact causes considerable wear and abra- sion of the piston and the pot wall. Equation (14.6.4.8-2) is an approximateequation which limits this bearing stress. If the pot is guided or fixed, horizontal forces will occur and the walls must be thicker than required by Equation (14.6.4.6-1). Equation (14.6.4.8-1) is a simple limit for the thickness of both the pot base and pot wall which is intended to limit the deformation of the pot bearing under horizontal load, since excess deformation may lead to elastomer leakage and other potential problems. This limit is likely to control the minimum thickness when the lateral load is large, but compressive load limits such as Equation 14.6.4.6-1 will control if the horizontal load is relatively small. where d is the diameter of the hole or holes in the bearing. C14.6.5 C14.6.5.2 C14.6.5.1 Steel Reinforced Elastomeric BearingsMethod B General The stress limits associated with Method A, specified in Article 14.6.6, usually result in a bearing with a lower capacity than a bearing designed using Method B. This increased capacity resulting from the use of Method B requires additional testing and quality control. Steel reinforced elastomeric bearings are treated separately from other elastomeric bearings because of their greater strength and superior performance in practice, Roeder, Stanton, and Taylor (1987), Roeder and Stanton (1991). The design method described in this article allows higher compressive stresses and more slender bearings than are permitted for other types of bearings, both of which can lead to smaller horizontal forces on the substructure. To qualify for the more liberal design, the bearing must be subjected to more rigorous testing. These test requirements are detailed in Article 18.7 of Division IZ of this specification. Tapered layers are expressly prohibited because they cause larger shear strains and bearings made with them fail prematurely due to delamination or rupture of the reinforcement. All internal layers should be the same thickness because the strength and stiffness of the bearing in resisting compressive load are controlled by the thickest layer. The shape factor, Si, is defined in terms of the gross plan dimensions of layer i. Refinements to account for the difference between gross dimensions and the dimensions of the reinforcement are not warranted because quality control on elastomer thickness has a more dominant influence on bearing behavior. Holes are strongly discouraged in steel-reinforced bearings. However, if holes are used, their effect should be accounted for when calculating the shape factor because they reduce the loaded area and increase the area free to bulge.
Suitable formulas are: for rectangular bearings for circular bearings Material Properties Shear modulus, G, is the single most important material property for design, and it is therefore the preferred means of specifying the elastomer.
Hardness has been widely used in the past because the test for it is quick and simple. However, the results obtained from it are variable and correlate only loosely with shear modulus, and the ranges given in Table 1 represent the variations to be found in practice. If the material is specified by hardness, a safe and presumably different estimate of G must be taken for each of the design calculations. Specifying the material by hardness thus imposes a slight penalty in design. The zones are defined by their extreme low temperature or the largest number of consecutive days when the temperature does not rise above 32' F, whichever gives the more severe condition. Materials with a nominal hardness greater than 60 are prohibited because they generally have a smaller elongation at break, greater stiffness and greater creep than their softer counterparts. This inferior performance is generally attributed to the larger amounts of filler present. Their fatigue behavior does not differ in a clearly discernible way from that of softer materials. Shear modulus increases as the elastomer cools, but the extent of stiffening depends on the elastomer compound, time, and temperature. It is therefore important to specify a material with low temperature properties which are appropriate for the bridge site. The bridge site should be classified as being in one of the five zones A-E, according to the definitions in Table 14.6.5.2-2. In order of preference, the low temperature classification should be based on: the 50-year temperature history at the site, or a statistical analysis of a shorter temperature history, or Figure 14.6.5.2-1 HIGHWAY BRIDGES Table 14.6.5.2-2 gives the minimum elastomer grade to be used in each zone. A grade suitable for a lower temperature may be specified by the Engineer, if desired, but improvements in low temperature performance can often be obtained only at the cost of reductions in other properties.
The definitions and tests for the elastomer grades, which are based on ASTM D 4014, are given in Section 18 of Division 11 of this Specification. This low temperature classification is intended to limit the force on the bridge under extreme environmental conditions to 1.5 times the maximum design force. Creep varies from one compound to another and is generally more prevalent in harder elastomers, but is seldom a problem if high quality materials are used.
This is particularly true because the deflection limits are based on serviceability and are likely to be controlled by live load, rather than total load. The creep values given in Table 14.6.5.2-1 are representative of neoprene, and are conservative for natural rubber.
C14.6.5.3 Design Requirements C14.6.5.3.1 Scope Steel reinforced bearings are designed to resist relatively high stresses. Their integrity depends on good quality control during manufacture, which can only be assured by rigorous testing. C14.6.5.3.2 Compressive Stress These provisions limit the shear stress and strain in the elastomer. The relationship between the shear stress and the applied compressive load depends directly on shape factor, with higher shape factors leading to higher capacities. If movements are accommodated by shear deformations of the elastomer, they cause shear stresses in the elastomer. These add to the shear stresses caused by compressive load, so a lower load limit is needed. The compressive limits were derived from static and fatigue tests correlated with theory, Roeder and Stanton (1986), Roeder, Stanton, and Taylor (1991). There was tremendous scatter in the stress at which delamination started in different tests, both fatigue and static. The absolute limits of 1.6 and 1.75 ksi came from the static tests. The static load limits were based on the fact that, of all the bearings which had not been previously tested, none showed any delamination at a compressivestress less than 2.5 ksi. The limits of 1.6 and 1.75 ksi thus provide a safety factor of approximately 1.5 against debonding in a wellmade bearing with a shape factor comparable to those used in the test program. However, long term loading was not investigated in the test program although it is known to be more detrimental to the bond, so the real safety factor against initiation of debonding may be somewhat less than 1.5. The compressive stress limits, in terms of GS, were derived from fatigue tests and are based on the observation that fatigue cracking in the experiments remained acceptably low, if the maximum shear strain due to total dead and live load was kept below 1.5. The level of damage considered acceptable had to be selected arbitrarily, therefore, the limits are not clear-cut. Two limits are given, one for total load and one for live load, and the more restrictive one will control. C14.6.5.3.3 Compressive Deflection Limiting instantaneous deflections is important to ensure that deck joints and seals are not damaged. Furthermore, bearings which are too flexible in compression could cause a small step in the road surface at a deck joint when traffic passes from one girder to the other, giving rise to impact loading. A maximum relative deflection across a joint of %-IN is suggested. Joints and seals that are sensitive to relative deflections may require limits that are tighter than this. Long-term deflections should be accounted for when joints and seals between sections of the bridge rest on bearings of different design, and when estimating redistribution of forces in continuous bridges caused by support settlement. Provided high quality materials are used, the effects of creep are unlikely to cause problems. Laminated elastomeric bearings have a nonlinear loaddeflection curve in compression. In the absence of information specific to the particular bearing to be used, Figure C14.6.5.3.3-1 may be used as a guide. Reliable test data on total deflections are rare because of the difficulties in defining the true zero for deflection. However, the change in deflection due to live load can be reliably predicted either by design aids based on test results or by using theoretically based equations, Stanton and Roeder (1982). In the latter case, it is important to include the effects of bulk compressibility of the elastomer, especially for high shape factor bearings.
C14.6.5.3.4 Shear The shear strain should be limited to +I- 0.5 h, in order ro avoid rollover at the edges and delamination due to fatigue problems. The fatigue tests were conducted to 20,000 cycles, which represents one expansion/contraction cycle per day for approximately 55 years, Roeder, Stanton, and Taylor (1990). The provisions will, therefore, be unconservative if the shear deformation is caused by high cycle loading 1997 COMMENTARY C-21 1600 Shppr 60 d u r o r n c l e r I2O0 - r e i n f o r c e d IOCIOI 12 1400 ' 0 1 2 3 Cornpresslve 4 5 scrr in 6 7 (Ye) Compressive strain (-A) FIGURE C14.6.5.3.3-1 Load Deflection Behavior of Elastomeric Bearings due to braking forces or vibration. The maximum shear deformation due to these high cycle loadings should be restricted to no more than +I- 0.10 h, unless better information is available. At this strain amplit

restriction device, as described in Article 14.6.8.5, or by some other means such as recessing the disc into the steel plates. Rotational experiments have shown that uplift occurs at relatively smdl moments and rotations in disc bearings. This leads to edge loading on PTFE sliding surfaces and increases the potential for dmage to the PTFE.
As a result, the allowable contact stress on PTHE is reduced to 75% of the value specified in Article 14.6.2.4 when the PTFE is used with a disc bearing. C14.6.8.5 Shear Resisting Mechanism The shear resisting device may be placed either inside or outside the urethane disc. If shear is carried by a separate transfer device external to the bearing, such as opposing concrete blocks, the bearing itself may be unguided. In unguided bearings, the shear force which must be transmitted through the body of the bearing is yP,where C14.6.9.1 General Guides are frequently required to control the direction of movement of a bearing. If the horizontal force becomes too large to be carried reliably and economically on a guided bearing, a separate guide system may be used. C14.6.9.2 Design Loads The minimum horizontal design load, equal to 10% of the maximum vertical load, is intended to account for responses which cannot be calculated reliably, such as horizontal bending or twisting of a bridge deck caused by nonuniform or time-dependent thermal effects. Large ratios of horizontal to vertical load can lead to bearing instability, in which case a separate guide system should be considered. C14.6.9.3 Materials Many different low-friction materids have been used in the past. Because the total transverse force at a bent is usually smaller than the total vertical force, the guides may contribute less than the primary sliding surfaces to the total longitudinal friction force. Thus material may be used which is more robust but causes higher friction than the primary material. Filled PTFE is common, and other proprietary materials, such as PTFE impregnated metals, have proved effective. C14.6.9.4 Geometric Requirements Guides must be parallel to avoid binding and inducing longitudinal resistance. The clearances in the transverse direction are fairly tight and are intended to ensure that C-26 HIGHWAN BRIDGES excessive slack does not exist in the system. Free transverse slip has the advantage that transverse restraint forces are not induced, but if this is the objective a nonguided bearing is preferable. On the other hand, if applied transverse loads are intended to be shared among several bearings, free slip causes the load to be distributed unevenly, possibly leading to overloading of one guide. C14.6.9.5 Design Basis C14.6.9.5.1 Load Location Guides are often bolted to the slider plate to avoid welding distortions. Horizontal forces applied to the guide cause shear and moment, both of which must be resisted by the bolts. The tension in the bolt can be reduced by using a wider guide bar. If high-strength bolts are used, the threaded hole in the plate must develop the full tension strength of the bolt. C14.6.9.5.2 Contact Stress Appropriate compressive stresses for proprietary materials must be developed by the Manufacturer and approved by the Engineer on the basis of test evidence. Strength, cold flow, wear and friction coefficient should be taken into consideration.
On conventional materials, higher stresses are allowed for short-term loading because the limitations in Table 14.6.2.4.1 are based partly on creep considerations. Shortterm loading includes wind, earthquake etc., but not thermal or gravity effects.
C14.6.9.6 Attachment of Low-Friction Material Some Miculties have been experienced when PTFE is attached to the metal backing plates by'bonding alone.
Ultra-violet light attacks the PTFE surface which is etched prior to bonding and this has caused bond failures. Thus, at least two separate methods of attachment are required. Mechanical fasteners should be countersunk to avoid gouging the mating surface. C14.6.10 Other Bearing Systems In appraising an alternative bearing system, the Engineer should plan the test program carefully because the tests constitute a larger part of the assurance program than is the case with more widely used bearings. In bearings which rely on elastomeric components, aspects of behavior such as time dependent effects, response to cyclic loading, temperature sensitivity, etc., should be checked.
C14.7 Load Plates and Anchorage for Bearings C14.7.1 Plates for Load Distribution Large forces may be concentrated in a bearing and they have to be spread out so as not to damage the supporting structure.
In general, metal rocker and roller bearings cause the most concentrated loads, followed by pots, discs and sphericals, while elastomeric bearings cause the least concentrated loads. Masonry plates may be required to prevent such damage to concrete or grout surfaces. Many simplified methods have been used to design masonry plates, some based on strength and some on stiffness. Several studies have indicated that masonry plates are less effective in distributing the load than these simplified methods would suggest (McEwen and Spencer, 1981 and Saxena and McEwen, 1986). NCHRP 10-20A has shown that there is substantial room for improvement of the design criteria for load plates, but there is not enough information to complete these changes in this specification. The present design rules represent an attempt to provide a uniform basis for design which lies within the range of traditional methods. Design based on more precise information, such as Finite Element Analysis, is preferable but may not be practical in all cases. Some types of bearings were only developed in the last 20 or 30 years, so their longevity has yet to be proven in the field. Hence the requirement for bearing replaceability. One common way to provide for replacement is to use a masonry plate, attached to the concrete pier head by embedded anchors or anchor bolts. The bearing can then be attached to the masonry plate by seating it in a machined recess and bolting it down. The bridge needs then to be lifted only through a height equal to the depth of the recess in order to replace the bearing. The deformation tolerance of joints and seals, as well as the stresses in the structure, should be considered in determining the allowable jacking height. C14.7.2 Tapered Plates Tapered plates may be used to counteract the effects of end slope in a girder. In all but short-span bridges, the dead load will dominate the forces on the bearing, so the tapered plate should be designed to provide zero rotation of the girder under full dead load. The limit of 0.01 RAD out of level corresponds to the 0.01 RAD component which is required in the design rotation in Article 14.4. C14.7.3 Anchorage Bearings should be anchored securely to the support to prevent their moving out of place during construction or 1997 COMMENTARY the service life of the bridge. Elastomeric bearings may be left without anchorage if adequate friction is available. A design coefficient of friction of 0.2 may be assumed between elastomer and clean concrete or steel. Girders may be located on bearings by bolts or pintles. The latter provide no uplift capacity. Welding may be used provided it does not cause damage to the bearing or dS1culties with replacement. Uplift must be prevented both between the major elements, such as the girder, bearing and support, and between the individual components of a bearing. If it were allowed to occur, some parts of the structurecould be misaligned when contact was regained, causing damage. C14.8 Corrosion Protection The use of stainless steel is the most reliable protection against corrosion, since coatings of any sort are subject to damage by wear or mechanical impact. This is particularly important in bearings where metal-to-metal contact is inevitable, such as rocker and roller bearings. Weathering steel is excluded because it forms an oxide coating which may inhibit the proper functioning of the bearing. When using hot-dip galvanizing for corrosion protection, several factors must be considered. Embrittlement of very high strength fasteners such as M 253 (ASTM A 490) bolts may occur due to acid cleaning (pickling) before galvanizing, and quenched and tempered material, such as Grade 70W and 100W, may undergo changes in mechanical properties, so galvanizing these should be avoided (see ASTM A 143 on avoiding embrittlement). With good practice, commonly used steels such as Grades 36,50, and SOW should not be adversely affected if their chemistry and the assembly's details are compatible (see ASTM A 385 on ensuring high quality coating). Certain types of bearings such as intricate pot or spherical bearings are not suitable for hot-dip galvanizing. REFERENCES 1. Campbell, T.I. and Kong, W.L., "TFE Sliding Surfaces in Bridge Bearings," Report ME 87-06, Ministry of Transportation, Downsview, Ontario, July 1987,57 pp. 2. 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The proposed direct design procedure is only a slight modification to the current direct design method used in Section 17. This procedure was accepted in 1993 by ASCE in the publication ASCE 93-15, "Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)." The design method was developed along with the research performed on the Standard Installations. However, the design equations are applied after the required bending moments, thrusts, and shear forces at all critical sections of the pipe have been determined using any one of the pressure distribution1 structural analysis methods allowed in Section 17; and are not intended for use only with the Standard Installations and Heger Pressure Distribution. Direct design for pipe, using the design equations that have been part of Section 17 since 1983, is facilitated using the Federal Highway Administration Computer Program PIPECAR. This program has recently been updated to include the design method proposed for Section 17. See also C Section 17 (1996). C17.1.2 Notations Several new notations are defined. These notations are used within the revisions proposed to Section 17. C17.4.6 Direct Design Method For Precast Reinforced Concrete Circular Pipe All of the subsections in 17.4.6 have been renumbered to allow for more information to be incorporated concerning the Direct Design Method and the pressure 'ptions available. C17.4.6.1 Application and C17.4.6.2 General These two sections replace the existing- Article 17.4.6.1. They include references to the new Heger Pressure Distribution and the possible pressure distributions that may be used for design. C17.4.6.3 Strength Reduction Factors and C17.4.6.4 Process and Material Factors These two sections have been renumbered. C17.4.6.6.1 Reinforcement for Flexural Strength fyhas been moved to the right side of Equation (17-9), and b has been defined as 12 inches. C17.4.6.6.2 M i m u m Reinforcement In Equations (17-lo), (17-ll), and (17-12), bl12 has been added to correct the equation units, and 65,000 has been replaced with fyto indicate other reinforcement yield strengths may be used. In Equations (17-10) and (17-ll), A, has been changed to ASiand A,, respectively to correctly correlate with the use of the equations. b has been defined as equal to 12 inches. Minimum outside steel (A,,) is reduced from 65% to the commonly used 60% of the inside steel area. This complies with the ratio of inside to outside steel used in ASTM C 76. C17.4.6.6.3 Maximum Flexural Reinforcement f, has been moved to the right side of Equations (17-13) and (17-14). The expression bl12 has been added to correct the equation units. The notation has been corrected to +,. A factor f, has been added to account for the increasing radial tension strength, shown in industry tests, as pipe diameter decreases below 72 inches. Conversely, F, gives a decrease in radial tension strength for pipe diameters above 72 inches. Three formulas defini@ F, have been included. +, C17.4.6.6.4 Crack Width Control (Service Load Design) Definition of the crack control factor has been added. I,Equation (17-15), 4, has been correctly placed in the denominator, rather than in the numerator where it has been incorrectly shown since publication. The formula defining eld has been modified to indicate that for values below 1.15, crack control will not govern. The table of B1and C1coefficient values for type of reinforcement has been modified to show only C1values. The formula defining Bl has been modified to reflect a constant control of the limiting crack width at 1 inch from the tension reinforcement, instead of at the surface. This information is necessary in designing pipe having cover thickness greater than 1 inch. B1 is also applied to all types of reinforcement, which was the intent of the design procedure as originally developed. Since cover and reinforcing spacing effect crack width for all types of reinforcing. C17.4.6.5 Orientation Angle C17.4.6.6.5 Shear Strength Possible misorientation of the pipe invert during installation is accounted for in the design process when designing quadrant mats, stinups, andlor elliptical cages. The location of the critical shear, Mn,N,d has been modified to use the effective moment, M, which has been 1997 COMMENTARY defined by a formula. This formula takes into account the effect of thrust. The thrust factor FN,has been modified in accordance with the thrust factors for shear strength in Article 8.16.6.2, to produce an increase in shear strength with compressive thrust and a reduction for tensile thrust. The resulting modified shear strength is close to the shear strength determined using the previous empirically determined thrust modification factors for the common case of compressive thrust. The shear resistance factor, is maintained for overall shear strength in Equation (17-16), but has been removed from various subsidiary terms and formulas, so as to be consistent with proper design practice. C-29 for typical reinforced concrete beams. Thus, P is included as a variable in Equation (17-19). The maximum d, is limited to a maximum clear cover of 2 in, consistent with the provisions of Article 8.16.8.4. See also C17.6.4.7 (2000). Commentary to Section IWoil-Thermoplastic Pipe Interaction Systems +, C17.4.6.6.6.1 Radial Tension Stirrups In Equation (17-17), +,has been corrected to +,in both the numerator and the denominator. The designation of the spacing of circumferential stirrups has been changed from S, to s, to be consistent with the nomenclature of other sections and articles. In accordance with the final recommendations made in the NCHRP Project 20-7, Task 68, Polyethylene Pipe Specijications Report, the material specification for Corrugated PE pipe is revised from cell class 315412C to 335420C.
A corresponding change in the Subcommittee on Materials will be made to revise the specified cell classification from 324420C to 325420C.
This change resolves a long-standing discrepancy between the Bridge Design and the Materials Specification. See also C18.4.3.1.2(2000). C17.4.6.6.6.2 Shear Stirrups +, In Equation (17-18), the which is applied to V, has been removed since is already applied in the terms outside the brackets. The term S, has been corrected to s,. In the formula for V,, the resistance factor has been removed since it already is applied in Equation (17- 18), and Muhas been corrected to M,. +, Commentary to Section 3--Temporary Works +, C17.4.7 Development of Quadrant Mat Reinforcement Current standards do not provide guidance for the detailing of quadrant mats. Article 17.4.7 is added to provide the necessary details for minimum main cage steel area and quadrant mat development length. Recommends default specifications for design. Clarifies erection trusses-access scaffolding is covered under OSHA, but stability trusses used for erection of structural steel are designed as falsework. Calls out Registered Professional Engineer. Commentary to Section l&Bearings C18.4 Materials C18.4.1 General The same crack control criteria that is the basis of Equation (8-61) is also the basis for Equation (17-19) for castin-place box sections. The z value of 130 specified in Article 8.16.8.4 for severe exposure is also used for castin-place box sections.
The basic derivation of this z value includes an assumption that a typical ratio of the distance from the neutral axis to the location of crack width divided by the distance from the neutral axis to the centroid of tensile reinforcing P, is 1.2, a typical value for reinforced concrete beams.
However, because cast-in-place box sections may have a range of p ratios from about 1.1 for thick slabs to about 1.6 for thin slabs, the variation in the P ratio for typical box sections is greater than the range of P values C18.4.1.1 Steel C18.4.1.1.1 The steel plate chosen should be compatible with that in the bridge, and the same steel is often chosen for both. Availability often influences the choices too. Sometimes it is difficult to obtain small quantities of specialized steel in relatively large thicknesses.
Many bridges are now made from A 588 (weathering) steel. However, this is not necessarily a good choice for the bearing unless it is completely protected against corrosion. C18.4.1.1.2 Steel laminates for steel reinforced elastomeric bearings are frequently less than '/a" thick, thus they cannot conform to A 36 or A 709 steels for which the HIGHWA.YBRIDGES specification does not extend below '/a" thickness. For thin plate, AISI C1020 is frequently used. This is a relatively ductile steel for cold formed metal structures which has no specified yield strength but usually has a yield strength of approximately 33 ksi.
Since the minimum thickness of the steel is often governed by fabrication criteria, little is to be gained by using a higher strength steel than necessary. Holes are not allowed in the steel plate unless they have been accounted for in design because the steel is in tension and would be weakened by holes. C18.4.1.1.6 Stainless steel welding is sometimes used to create a continuous overlay over carbon steel plate, for example in spherical sliding bearings. The stainless steel layer so created is then machined to give a smooth surface. C18.4.3 Special Material Requirements for PTFE Sliding Surfaces The lubricant most frequently used in Europe with sheet PTFE is based on lithium grease. It has proved to be effective and stable over long periods. C18.4.3.1 PTFE The tests described in this section are intended to determine the purity and crystallinity of the PTFE. The purity influences the friction factor which can be obtained, and the crystallinity influences both the resistance to wear and the friction. The specific gravity test is indirectly a test of the crystallinity. The strength tests are necessary to make sure that the PTFE does not creep excessively and does not fail in direct tension.
The test strength required of woven fabric material is extremely high because it is at present conducted on a single strand of PTFE fiber. This is a carry-over from the existing practice, but a strength test which measures the strength of the finished fabric in pounds per inch of fabric would probably be better. C18.4.4 Special Material Requirements for Pot Bearings C18.4.4.1 The rotational element should be made from a flexible elastomer. The elastomer is fully confined in the pot and therefore cannot undergo large deflections, so no advantage is gained by using a stiffer elastomer as might be the case in a laminated elastomeric bearing. C18.4.4.3 Sealing Rings Sealing rings are presently made from brass in the USA. Attempts were made to use PTFE in the past, but these were unsuccessful because the PTFE ring squeezed out through the gap between the pot and the piston and thereafter was ineffective as a seal. However, certain proprietary materials have also been used in Europe with success. They would require verification testing before being accepted in the USA. C18.4.5 Special Material Requirements for Steel Reinforced Elastomeric Bearings and Elastomeric Pads C18.4.5.1 Properties of the Elastomer At present only natural Rubber (polyisoprene) and neoprene (polychloroprene) are permitted. This is because both have an extensive history of satisfactory use. In addition, much more field experience exists with these two materials than with any other, and almost all of it is satisfactory.
The low-temperature grading system addresses the problem of stiffening of the elastomer at low temperatures. Special compounding and curing are needed to avoid the problem but they increase cost and in extreme cases may adversely affect some other properties. These adverse effects can be minimized by choosing a grade of elastomer appropriate for the conditions prevailing at the site. The grades follow the approach of ASTM D 2000 and D 4014, with more stringent low temperature test criteria for higher grades. Tables 25.3.1A and B outline the required properties of the elastomer.
The standards are sometimes different for neoprene and natural rubber, which appears irrational because in other ways the requirements resemble a performance specification. However, the present state of knowledge is inadequate to pin down precisely those material properties needed to assure good bearing behavior, so the tests are intended to achieve a generally good quality material rather than specific properties. Natural rubber and neoprene have different strengths and weaknesses, so different tests are indeed appropriate. (Generally natural rubber creeps less, suffers less low-temperature stiffening, and has a better elongation at break-but neoprene has better chemical, ozone, and aging resistance.) The previous low temperature brittleness test has been augmented by two others: the Clash-Berg test for low temperature stiffness (ASTM D 1043) and a test for low temperature crystallization stiffening (the ASTM D 4014 quad shear test conducted at low temperature). All three tests are required for elastomers of grade 3 and above. Previously, the brittleness test at -40' F was required for all elastomers, including those to be used in the southern tier states, yet no test was required for thermal or crystallization stiffening, even in the northern tier states or Alaska.
The brittleness test essentially detects glass transition, but gives no indication of stiffening. The Clash-Berg test is introduced to detect instantaneous low temperature 1997 COMMENTARY C-31 The tests required here are intended to insure a good quality material. stiffening. It is quick to perform and requires only a modest investment in special equipment. Crystallization stiffening is both time- and temperature-dependent, but constitutes a significant portion of the total low temperature stiffening of many elastomers. Detecting it is therefore important and is done by the long-duration shear stiffness test. In addition to the ASTM D 4014 quad-shear rig, this test requires a freezer which surrounds the rig. Because of the nature of the crystallization, the test may take up to 28 days, so it is not required for every lot of bearings. Harder elastomers have a greater shear stiffness and thus exert larger pier forces due to thermal expansion than materials of low hardness, unless the plan area of the bearing is reduced proportionately. This could cause the bearing to be rather slender, possibly leading to instability problems. Further, 70 durometer material generally creeps more than its softer counterparts. Thus, when largekcompressive stiffness is required, it is recommended that reinforced bearings of softer elastomer with thinner layers and higher shape factor be used. Hardness is maintained as a material property because it is widely used in rubber technology and is easy to measure. However, measurements are sensitive to who takes them, and hardness generally gives only rough indication of mechanical properties particularly at low temperatures. The shear modulus is a much more useful property, but is more time consuming to measure. C18.5.1.4 Designing bearings for replacement is important because even high quality bearings have in some cases been known to fail because of unanticipated forces or other conditions. Setting the bearing in a shallow recess in the masonry plate is a simple way of making replacement easy. C18.4.5.2 C18.5.1.5 Fabric Reinforcement Fiberglass is the only fabric proven to perform adequately as reinforcement, and only one grade is currently permitted. Polyester has proved too flexible, and both it and cotton are not strong enough. The strength of the reinforcement governs the compressive strength of the bearing when minimum amounts are used, so if stronger fabric with acceptable bond properties is developed, the stress limits of Article 14.6.6.3.2of Division I may be reconsidered. However, thorough testing over a wide range of loading conditions, including fatigue, will be needed prior to acceptance. C18.4.5.3 Bond Adequate bond is essential if the reinforcement is to be effective. It is particularly important at the edges of the bearing. C18.4.7 Special Material Requirements for Disc Bearings Polyether urethane is a hard tough plastic material. However, its tensile strength varies significantly depending on the quality control exercised during processing. C18.4.8 Special Material Requirements for Guides Very low friction coefficients are less necessary for guides than for the PTFE slider which supports the gravity load. This is because friction on the guides contributes only a small percentage of the longitudinal resisting force of the bearing. Thus filled PTFE, which has a better resistance to creep than pure PTFE, is often used for guides. The use of a filler means that it is not necessary to recess the PTFE in a metal backing plate, and this therefore saves some machining. PTFE filled with fiberglass or carbon fibers, and a PTFE and sintered metal mixture have been used with success. C18.5 Fabrication C18.5.1 . General Tolerances Each bearing type has one or more tolerances which are particularly important.
In bearings which depend on rocking or rolling surfaces, it is most important to ensure that the curvature of the curved surface is constant to within a fine tolerance.
This is more important than the actual value of the radius of curvature. In nested roller bearings it is also important to ensure that all the rollers have exactly the same radius of curvature, because if they do not, the load will not be equally shared between them. In flat PTFE sliding surfaces, the surface finish of the mating surface, usually stainless steel, is particularly important. A number 8 mirror finish or better is recommended in all cases. In bearings which depend on the sliding of one curved surface over another, such as curved PTFE sliding bearings, curved bronze sliding bearings, or pins and bushings which allow rotation, the difference in diameter of the two curved surfaces is the most important tolerance. The out of round or the variation in curvature of the curved surface is also important, and again the actual value of the radius of curvature is less important. If two parts of the bearing are made by different fabricators, machining by fitting the two parts is not possible and it is necessary to machine HIGHWAY BRIDGES C-32 each part to a specific radius within a very high accuracy. In the past, bearings made of components which are fabricated by different manufacturers have given problems because of lack of a good fit.
In pot bearings, the most important tolerances are those on the clearance between the pot and the piston and on the vertical clearance between the upper and lower parts of the bearing. C18.5.3 Special Fabrication Requirements for PTFE Sliding Bearings C18.5.3.1 PTFE sheets should be both recessed and bonded to obtain the best performance. The recessing inhibits creep or cold flow and the bonding keeps the PTFE in the recess when the mating surface of stainless steel slides over it. Without the bond there is a risk that conditions such as eccentric loading would cause the PTFE to come out of the recess. The minimum bonding pressure of 100 psi is intended to ensure that the adhesive under the PTFE is well distributed and that the final PTFE surface will be flat. C18.5.5 Special Fabrication Requirements for Pot Bearings C18.5.5.1 Pots The most common way of fabricating a pot is to machine it from a single piece of steel plate. However, for very large pots this may be uneconomical because it means a large amount of machining. In such cases, casting, forging or fabrication by welding are possible but they introduce extra difficulties beyond those found in pots machined from plate. If the pot is made by welding a ring to a base plate, the weld is critically important. The weld must be made on both the inside and the outside of the ring and then the weld on the inside must be machined if necessary to give the correct final profile. The welds must be verified by suitable ultrasonic or radiographic examination methods and the flatness of the plates after welding must be ensured. C18.5.6 Special Fabrication Requirements for Steel Reinforced Elastomeric Bearings and Elastomeric Pads - . C18.5.3.2.3 Woven PTFE Woven PTFE cannot be kept in place by a recess in the same way that sheet PTFE can, so some other means is necessary. It can be attached to its backing substrate either by bonding or by forming in the metal substrate mechanical indentations into which the PTFE weave is pressed.
The effectiveness of such a mechanical connection can be judged by a test in which one piece of woven FTFE is compressed between two indented metal substrates and the PTPE is pulled out from between them. C18.5.3.3 Stainless Sted Stainless steel should be attached by welding all around. This not only ensures a uniform transfer of stress from the PTFE to the backing plate when the shinless steel is subjected to shear from sliding forces, but it dso minimizes the corrosion which can occur behind the PTFE. C18.5.4 Special Fabrication Requirements for 'sfor all Elastomeric Bearings which are designed as a single unit must be built as a single unit, because the shape factor, bearing stiffness and strength, and general behavior under load will be different if built in sections. C18.5.6.2 Steel Laminated Elastomeric Bearings In order to achieve good bond, the steel laminates must first be thoroughly sand blasted and cleaned and then protected against contaminationuntil fabrication is complete. Edge cover is primarily needed to prevent corrosion of the reinforcement and ozone attack of the bond. However, it also decreases the probability of delamination by reducing the stress concentrations at the exposed outer surface. In the past, bonding during vulcanization has been the most successful method of attaching the laminates, and is required for bonding of internal laminates. However, practical difficulties may arise in hot bonding of external plates, thus hot bonding is strongly recommended for them, but not required. Curved Sliding Bearings C18.5.9 Special Fabrication Requirements for The mating parts of each bearing, especially machined metal bearings such as self-lubricating bronze bearings, shpuld be furnished by a single manufacturer in order to ensure proper fitting of the mating surfaces. Guides Guide bars are usually attached by bolting because welding tends to introduce distortions. If the bolts are held 1997 COMMENTARY C18.5.9.3 The low friction material used for guides must be attached by two of the three given methods to avoid the debonding problems which have occurred in the past. the complete bearing. They represent a check that the materials being used will have suitable resistance to wear and deterioration over the long term.
Since the quality of materials both in the short and the long term is essentially being assured by material sample tests rather than tests on the complete bearing, it is also necessary to make sure that the material will behave in the same way in the complete as it will in the material sample.
This is done by complete bearing tests, which again need be done at the least frequent rate. C18.7 Testing and Acceptance C18.7.2.2 Material Friction Test C18.7.1 General It is important that the material tested here is identical to that used in the finished bearing. In particular, no lubricant whatsoever should be applied during the test unless it is also required in the finished bearing, and the (stainless steel) mating surface should be new for every piece of material tested. Thus the same piece of stainless steel should not be used for more than one PTFE specimen in the PTFE material tests. The friction coefficients which constitute the performance criteria for the tests are directly related to the values used in design. This is considered more realistic than the arrangements in the 15th edition of the AASHTO Specification. in threaded holes in the plate rather than by nuts, the threaded holes should have adequate length to develop the full tension strength of the bolt. The guide bars must be attached with a relatively fine tolerance in order to prevent locking up when the bearing displaces longitudinally. The purpose of testing is to ensure a good quality finished bearing. The obvious way to achieve this is to conduct rigorous tests on every bearing. However, this is economically infeasible and resorting to other methods is necessary. The testing outlined here is based on four different levels. The first is to ensure that the proper materials are supplied and this is achieved by material tests. The second and third are concerned with immediate testing for short term behavior. In the second, the dimensions of the bearing, and in particular some critical dimensions which are different for each bearing type, are checked to ensure that they are within the proper tolerances. The third type of test is then short term proof loading tests. This is required only in cases where it will produce useful information.
For example, applying a vertical load to a spherical sliding bearing for a short period of time is unlikely to be informative. The fourth and last category of tests consists of long term proof loading tests to demonstrate the adequacy of the bearing under long term conditions which could cause wear or damage. Because the long term tests require more complicated test machinery and a longer test time, they are inevitably more expensive than short term ones. Therefore the frequency with which each of these tests must be done is determined separately. The material quality control tests and the dimensional check must essentially be made for every bearing.
These are routine tests and are simple and cheap to perform. Where short term compressive proof load tests are simple to do and produce useful information, they are required. For example, in steel laminated elastomeric bearings such a proof load test gives an indication of the quality of the bond and the placement of the steel and rubber layen within the bearing. This test can be done by the manufacturer in the same press in which the bearing was fabricated. It therefore requires very little extra time or effort and no additional equipment. The long term tests need be done the least frequently, and they may be conducted on samples of material rather than on C18.7.2.9 Bearing Horizontal Force Capacity This test is only for bearings which must resist prescribed horizontal forces. It is important to select only realistic combinations of vertical and horizontal load.
Selection of an impossible load combination may result in unwarranted rejection of the bearing. Bearings which must carry a large ratio of horizontal to vertical force are frequently an indicator of a poorly thought out bearing system. (218.7.4 Special Testing Requirements The minimum testing frequencies laid out in this section represent absolute minimums. In cases where the bearing performance is particularly critical, the engineer may demand more stringent testing. An example might be a bridge in which the bearings are relatively inaccessible and would be difficult and expensive to replace. C18.8 Packing, Shipping and Storing Small amounts of grit, dirt, or other contamination can seriously detract from the good performance which could otherwise be obtained from a bearing.
It is therefore very important that the bearing should not be opened up on site except under the supervision of the manufacturer or his agent. C-34 HIGHWAY BRIDGES C18.9 Installation Bridge bearings are precisely engineered products and must be treated as such when they are installed. Frequently the bearing has a relatively low profile which is gained by providing only the minimum required rotation capacity. Thus it is crucial that the bearing be installed level and that the girder which will be seated on it also has a level underside. Furthermore, guided bearings must be oriented correctly, or else large horizontal forces may be introduced. Bearings should be marked in such a way that the identification number can be seen after erection.
This will assist in settling any disputes that may arise, because the test results for that bearing should be traceable. It also provides a means of monitoring and investigating the long term field behavior of bearings. Commentary to Section 19-Pot Bearings Section replaced by new Section 14 in 1997. Commentary to Section 20-Disc Bearings Section replaced by new Section 14 in 1997. 1998 Commentary to Standard Specifications for Highway Bridges DMSION I COMMENTARY TO "SECTION 5-RETAINING WALLS" Figure C5.2A For consistency in presentation. Concerns have been raised by some state DOT's and wall suppliers that the new Simplified Method does not fully account for the effect of large or steep sloping surcharges.
(See commentary for Article 5.8.4.1.) The Simplified Method as well as the Coherent Gravity Method do not fully accountfor large, steep continuous sloping surcharges, as such surcharges can effect a combination of internal and external stability, resulting in failure surfaces which are partially internal and external (compound failure surfaces). Therefore, it is prudent to perform additional analyses to check reinforcement requirements and overall stability. See also C5.8.2 (1996). The 1997 Interims eliminated the use of the Coherent Gravity and Structure Stiffness Methods for determining internal stability of ME wall systems, and required all ME wall systems to be designed using the new Simplified Method. This action caused significant concern among some of the state DOT's and some of the wall suppliers. Changing to a new design method results in a need to redesign all existing wall standards and computer software for those systems. They did not consider the extra effort for these changes to be warranted, especially considering that many walls have been designed using these methods over the last 20 years and the walls have not exhibited poor performance. In addition, there was a perception that the Simplified Method was unconservativefor some walls and overly conservativefor others. Additional studies have been performed comparing the Simplified Method and the Coherent Gravity Method.
Re- sults of these studies indicate that both methods provide reasonably adequate results for a variety of loading conditions and wall heights relative to the actual reinforcement stresses measured, and that there is very little difference in the amount of reinforcement needed in a given wall designed by either method. However, for welded wire wall and bar mat walls, the Simplified Method shifts the density of reinforcement from the bottom to the top of the wall. The Simplified Method is the preferred design method for a national specification because it provides the most uniform results for all wall types and it can easily be adapted to any future ME wall type. However, other design methods have been used successfully for many years with no apparent performance problems, and some state DOT's have co dence in those methods. The proposed revision retains the Simplified Method as the preferred method, but allows other proven design methods to be used. Measured loads in the reinforcement at the wall face tend to be less than T, near the upper portion of the wall. However, construction practices can introduce uncertainties in the loads in the reinforcement at the wall face. Therefore, it is prudent to design for 100% of T, at the wall face. Figure C5.8.5.2A Source document (FHWA Publication No. FHWA-SA97-071) indicates maximum value of 2.0 should be used. Furthermore, the minimum value of C, of 4 will result in a maximum F* value greater than 1.5. Table C5.8.6.1.2.A The test methods quoted in the currentAASHT0 specification do not fully address the polymers used in geosynthetics nor do they reflect industry practice. These test procedures will yield,incorrect results which could affect C-38 HIGHWALYBRIDGES In the New Zealand Code, which accepts the philosophy that it is uneconomical to design a bridge to resist a large earthquake elastically, bridges are designed to resist small-to-moderate earthquakes in the elastic range. For large earthquakes the design philosophy is that bridges be ductile where possible. Flexural plastic hinging in the columns is acceptable but significant damage to the foundations and other joints is not. Consequently, as a second step in the design process, forces resulting from plastic hinging in all columns are determined and the capacities of connections to columns are checked to determine if they are able to resist these forces. Hence, critical elements in the bridge are designed to resist the maximum forces to which they will be subjected in a large earthquake. In the Caltrans approach the member forces are determined from an elastic design response spectrum for a maximum credible earthquake. The design forces for each component of the bridge are then obtained by dividing the elastic forces by a reduction factor (Z). The Z-Factor is 1.0 and 0.8, respectively, for hinge restrainers and shear keys. These components are therefore designed for expected and greater-than-expected (in the case of shear keys) elastic forces resulting from a maximum credible earthquake. Well-confined ductile columns are designed for lowerthan-expected forces from an elastic analysis as Z varies from 4 to 8. This assumes that the columns can deform plastically when the seismic forces exceed these lower design forces. The end result is similar to the New Zealand approach although the procedures are quite different. In assessing bridge failures of past earthquakes in Alaska, California and Japan, many loss-of-span type failures are attributed in part to relative displacement effects. Relative displacements arise from out-of-phase motion of different pars of bridge, from lateral displacement andlor rotation of the foundations and differential displacements of abutments. Therefore in developing these Specifications the design displacements and forces were considered equally important.
Thus minimum support lengths at abutments, columns and hinge seats are specified, and for bridges in areas of high seismic risk ties between noncontinuous segments of a bridge are specified. Special attention to the problem of relative displacements is required for bridges with high columns or piers. The methodology used in these Specifications is, in part, a combination of the New Zealand2 and Caltrans3 "force design" approaches but also addresses the relative displacement problem. The complexity of the methodology increases as the seismic intensity of an area increases. Four additional concepts are included in these Specifications that are not included in either the Caltrans or New Zealand approach. First, minimum requirements are specified for support lengths of girders at abutments, columns and hinge seats to account for some of the important rel- ative displacement effects that cannot be calculated by current state-of-the-art methods. A somewhat similar requirement is included in the Japanese bridge design riter ria.^ Second, member design forces are calculated to accountfor the directional uncertainty of earthquake motions and the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions. Third, design requirements and forces for foundations are intended to minimize foundation damage which is not readily detectable. Fourth, a basic premise in developing the Specifications was that they be applicable to all parts of the United States. In order to provide flexibility in specifying design provisions associated with areas of different seismic risk, four Seismic Performance Categories (SPC) were defined. The four categories permit variation in the design requirements and analysis methods in accordance with the seismic risk associated with a pmicular bridge location. Bridges classified as SPC D are designed for the highest level of seismic performance and bridges classified as SPC A for the lowest level of seismic performance. For bridges classified as SPC A, prevention of superstructure collapse is all that was deemed necessary for their level of seismic exposure. The requirements for these bridges are minimal and specify the support lengths for girders at abutments, columns and expansion joints, and that the design of the connections of the superstructure to the substructure be for 0.20 times the dead load reaction forces. For bridges classified as SPC B the approach used is similar to that of Caltrans where elastic member forces are determined from a single-mode spectral method of analysis. Design forces for each component are obtained by dividing the elastic forces by a response modification factor (R). For connections at abutments, columns and expansion joints, the R-Factor is either 1.0 or 0.8; therefore these components are designed for expected or greaterthan-expected elastic forces. For colum

a 50-year period is estimated to be about 90%. At present, this probability cannot be estimated precisely. Moreover, since the maps were adjusted and smoothed by the committee after consultation with seismologists, the risk may not be the same at all locations. It is believed that this probability of the design ground motion not being exceeded is in the range of 80% or 90%. The use of a 50-year interval to characterize the probability is a rather arbitrary convenience, and does not imply that all structures are thought to have a useful life of 50 years. The probability that an ordinate of the design elastic response spectrum will not be exceeded, at any period, is approximately the same as the probability that the EPA will not be exceeded. The veracity of this statement lies in the fact that the uncertainty in the EPA that will occur in a future earthquake is much greater than the uncertainty in spectral ordinates, given the EPA. Thus the probability that the ordinates of the design elastic response spectrum will not be exceeded during a 50-year interval is also roughly 90%, or in the general range of 80 to 95%. F. Site Effects and Elastic Seismic Response Coefficient and Spectrum (Articles 3.5 and 3.6) It is known that the characteristics of ground shaking and the corresponding spectra are influenced by: 1. The characteristics of the soil deposits underlying the proposed area. 2. The magnitude of the earthquake producing the ground motions. 3. The source mechanism of the e m q u a k e producing the ground motions. 4. The distance of the earthquake from the proposed site and the geology of the travel path. While it is conceptually desirable to include specific consideration of all four of the factors listed above it is not possible to do so at the present time because of lack C-43 of adequate data. Sufficient information is available to characterize in a general way the effects of specific soil conditions on effective peak acceleration and spectral shapes. The effects of the other factors are so little understood at this time that they are often not considered in spectral studies. The present recommendations therefore only consider effects of site conditions and distance from the seismic source zone.

At such times that potential effects of other significant parameters can be delineated and quantified, the current recommendations can be modified to reflect these effects. Thus, the starting points in the development of the ground motion spectra are the seismic design regionalization maps that express, by contours, the EPA that would be developed on firm ground. Site Effects -- The fact that the effects of local soil conditions on ground motion characteristics should be considered in structural design has long been recognized in many countries of the world. Most countries considering these effects have developed different design criteria for several different soil conditions. Qpically these criteria use up to four different soil conditions. In the early part of the ATC3-06 study consideration was given to four different conditions of local site geology.

On the basis of available data, the following four conditions were selected: 1. Rock--of any characteristic, whether it be shalelike or crystalline in nature. In general, such material is characterized by a shear wave velocity greater than about 2,500 ft/sec (750 mlsec). 2. Stiff soil conditions or firm ground-including any site where soil depth is less than 200 ft (60 m) and the soil types overlying rock are stable deposits of sands, gravels, or still clays.

3. Deep cohesionless or stiff clay soil conditionsincluding sites where the soil depth exceeds about 200 ft (60 m) and the soil types overlying rock are stable deposits of sands, gravels, or still clays.

4. Soft to medium-stiff clays and sands-characterized by several tens of feet of soft to medium-stiff clay with or without intervening layers of sand or other cohesionless soils. Effective Peak Accelerations for DifSerent Site Conditions The values of EPA for rock conditions were first modified to determine corresponding values of EPA for the three other site conditions outlined above. This modification was C-44 HIGHWAY BRIDGES based on a statistical study of peak accelerationsdeveloped at locations with different site conditions and the exercise of judgment to extrapolate beyond the data base. After evaluating these effects and rounding out the results obtained, the values of EPA were modified as follows. For the first three soil types (rock, shallow stiff soils and deep cohesionlessor stiff clay soils) there is no reduction. For the fourth soil type (soft to medium-stiff clays) a reduction factor of 0.8 is used for all seismicity index areas. It should be pointed out that statistical data show that the reduction effect is not constant for all ground motion levels and that the value of the reduction factor is generally smaller than is recommended here. number of soil types to three by combining the spectra for rock and stiff soil conditions. But when preparing the present specification a fourth soil type was added to represent very soft sites where shear wave velocities may be as low as 500 ftlsec (150 dsec). Damage sustained by bridges on such sites during recent earthquakes in California (1989 and 1994), the Philippines (1990), Costa Rica (1991), and Japan (1995) have highlighted the hazardous nature of these particular sites and the need to address them with a separate site coefficient. Accordingly four normalized spectra are shown in Figure C3.5B. These curves apply to the following four soil conditions. Soil Profle Type I: Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 Wsec (750 mlsec); or stiff soil conditions where the soil depth is less than 200 ft (60 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays. Soil Profle Type IZ: Deep cohesionless or stiff clay soil conditions, including sites where the soil depth exceeds 200 ft (60 m) and the soil types of overlying rock are stable deposits of sands, gravels, or stiff clays. Soil Profle Type 111: Soft to medium-stiff clays and sands, characterized by 30 ft (9 m) or more of soft to Spectral Shapes Spectral shapes representative of the different soil conditions discussed above were selected on the basis of a statistical study of spectral shapes developed on such soils close to the seismic source zone in past earthquakes. The mean spectral shapes determined directly from the study by Seed et aL7based on 104 records, primarily from earthquakes in the Western United States, are shown in Figure C3.5A. These spectral shapes were also compared with spectral shapes from studies conducted by Blumes, Newmark: and Mohraz.l0 During the development of ATC-3-06, it was considered appropriate to reduce the SPECTRA FQR 5% DAMPING T TO MEDIUM CLAY 8 SAND EEP COHESIONLESS SOIL TIFF SITE CONDITIONS 0, 0 ! I 03 1.0 I L5 I 20 PER100- S f CON05 I 25 30 FIGURE C3.5A Average Acceleration Spectra for Different Site Conditions (after Seed, et al., 1976) 1998 COMMENTARY 4 I I 1 /SOIL C-45 I I PROF ILE TYPE S4 J 3' SOIL PROF lLE TYPE S1 . FIGURE C3.5B Normalized Response Spectra medium-stiff clay with or without intervening layers of sand or other cohesionless soils. greater than w:Soft 'lays Or 40 ft (12 m) in depth, characterized by a shear wave less than 500 n/sec mlsec) and P ~ including loose natural deposits and man-made, nonengineered fill. velOciv Ground motion spectra for 5% damping for the different map areas are thus obtained by multiplying the normalized spectral values shown in Figure C3.5B by the appropriate EPA and by the correction factor of 0.8 if Soil Profile Type III or IV exists. The resulting ground motion spectra for an EPA of 0.4 are shown in Figure C3.5C. The spectra from Figure C3.5C are shown in Figure C3.5D plotted in tripartite form. It can be readily seen in Figure 12 that for aU soil conditions the response spectra for periods near l second are horizontal or equivalent to a constant spectral velocity.

It should also be noted that these spectra are modified as discussed in the following section before they are used in the design provisions. On the basis of studies of spectral shapes conducted by Blume8 and Newmark? spectra for 2% damping may be obtained by multiplying the ordinates of Figure C3.5B by a factor of 1.25. Spectra for vertical motions may be determined with sufficient accuracy by multiplying the ordinates of the spectra for horizontal motions by a factor of 0.67. Elastic Seismic Response Coeficient and Spectra The equivalent lateral force method of design requires hata horizontal forcebe a c c o m o ~ a t ein ~ the structuraldesign. The magnitudeof this force is a function of several pa~ te ~ including ~ theAcceleration ~ ~ Y Caeficient, the Qe of rame rs soil at the site, and the fundamental period of the structure. For use in a design - -provision or code it is distinctly advantageous to express the lateral design force coefficient in as simple a manner as possible.

The recommended procedure for determining the lateral design force coefficient C, is given by Equation (3-1) in Article 3.6 as: The value of C, need not exceed 2.5A for all soil types. The site coefficient, S, is given in Table 3.5.1. The use of a simple soil factor in Equation (3-1) approximates the effect of local site conditions on the design requirements. This direct method eliminates the need for the estimation of a predominant site period and the computation of a soil factor based on the site period and the fundamental period of the bridge. It is apparent from the discussion on spectral shapes in the foregoing paragraphs and from Figures C3.5B and C3.5C that the elastic acceleration response spectrum decreases approximately as 1/T for longer periods. How- HIGHWAY BRIDGES C-46 U' I nrn:oirrm,unr&~~~d~h~~ll saLlw€ss~lusLL#tWa3 SOIL PROF lLt TYPE $4 SOIL PROFILE TYPE 5 SOIL PROFILE TYPE $2 I FIGURE C3.SC Ground Motion Spectra for A = 0.4 UNOAMED NATURAL PER100 (SECONDS) FIGURE C3.5D Ground Motion Spectra for A = 0.4 1998 COMMENTARY ever, because of the concerns associated with inelastic response of longer period bridges it was decided that the ordinates of the design coefficients and spectra should not decrease as rapidly as 1/T but should be proportional to 1 P as shown in Equations (3-1) and (3-2). A comparison of the spectra resulting from Equations (3- l) and (3-2) and those of the ATC-3-06 elastic acceleration response spectra is given in Figure C3.5E. It will be seen that the elastic seismic response coefficient is approximately 50% greater at a period of 2 seconds for the stiff soil condition than would be obtained by direct use of the elastic acceleration response spectra. This increase gradually decreases as the period of the bridge shortens. The two major reasons for introducing this conservatism in the design of long period bridges are: 1. The fundamental period of a bridge increases as the column height increases, the span length increases and the number of columns per bent decreases. Hence the longer the period the more likely that high ductility requirements will be concentrated in a few columns. 2. Instability of a bridge is more of a problem as the period increases.

The relationship between the response coefficient and bridge period is discussed further below. Elastic Seismic Response Coeficient for Multimodal Analysis Equation (3-4) is to be used if a modal period exceeds 4 seconds. It can be seen that Equations (3-4) and (3-2) coincide at Tm= 4 sec, so that the effect of using Equation (3-4) is to provide a more rapid decreasein C, as a function of Tmthan implied by Equation (3-2). This modification is C-47 introduced in consideration of the known characteristics of earthquake response spectra at intermediate and long periods. At intermediate periods the average velocity spectrum of strong earthquake motions from large earthquakes (magnitude 6.5 and larger) is approximately horizontal, this irnplies that C, should decrease as IDm.In Equation (3-2), C, for reasons discussed earlier in this Artidecreases as cle and this slower rate of decrease, if extended to very long periods, would result in an unbalanced degree of conservatism in the modal forces for very flexible bridges. In addition, for very long periods, the average displacementspectrum of strong earthquake motions becomes horizontal; this implies that C, which has the form of an accelerationspectrum, should decay as I n m .The period at which the displacement response spectrum becomes horizontal depends on the size of the earthquake, being longer for large earthquakes, and arepresentative period of 4 seconds was chosen to make the transition. A central feature of modal analysis is that the earthquake response is considered as a combination of the independent responses of the bridge vibrating in each of its important modes. As the bridge vibrates back and forth in a particular mode at the associated period, it experiences maximum values of member forces and displacements. The coefficient Csmis determined for each mode from Equation (3-2) using the associated period of the mode, T,, in addition to the factors A and S, which are discussed elsewhere in this Article. An exception to this procedure occurs for higher modes of those bridges which have periods shorter than 0.3 seconds and which are founded on Type IU and IV soils. For such modes, Equation (3-3) is used. Equation (3-3) gives values ranging from 0.8 A for very short periods to 2.0 A for T, = 0.3. Comparing these values with the limiting value of C, of 2.0 A for Type IU and IV soils as specified following Equation (3-2), it is lm FIGURE C3.5E Comparison of Free Field Ground Motion Spectra and Lateral Design Force Coefficients C-48 HIGHWA,YBRIDGES seen that the use of Equation (3-3), when applicable, reduces the modal base shear. This is an approximation introduced in consideration of the conservatism embodied in using the spectral shape specified by Equation (3-2) and its limiting values. This shape is a conservative approximation to that of average spectra which are known to first ascend, then level off, and then decay as period increasessee Figures C3.5A and C3.5B. Equation (3-2) and its limiting values conservatively replace the ascending portion for small periods by a level portion.

For Type I or I1 soils, the ascending portion of the spectrum is completed by the time the period reaches a value near 0.1 or 0.2 seconds.

On the other hand, for soft soils the ascent may not be completed until a larger period is reached. Equation (3-3) is then a replacement for Type IIIand IV soils at short periods, which is more consistent with spectra for measured accelerations. It was introduced because it was judged unnecessarily conservative to use Equation (3-2) for modal analysis in the case of Type III and IV soils. SEISMIC MOMENT Seismic moment is another measure of the strength or size of an earthquake caused by a fault slip. It is denoted by M, and is equal to the product of (1) average fault slip, (2) slip area, and, (3) shear modulus of undisturbed rock of the fault slip region. It is also proportional to the fourier spectral amplitude of displacement record for very low, approaching zero, frequency waves. That spectral amplitude corresponds to the permanent (average) slip on the fault during the earthquake. Thus, seismic moment can be estimated from (a) analysis of instrumental records and (b)also from geologic/geodeticobservations.

As such, it is a fundamental quantity, which is useful for quantifying and comparing earthquakes. Instrumental earthquake magnitude scale originated by Richter and the subsequent modificationsare used by seismologists to quantify and compare earthquake sizes. All such magnitude scales have limitations imposed by the recording instruments and particular seismic waves used for magnitudes calculations. They all have saturation points beyond which magnitudes are fixed.

For example, the limit for Richter magnitude scale appears to be around 6 and surface wave around magnitude 8.

This problem is very critical especiallyfor large earthquakes because such magnitude scales will not recognize them. Again, such large earthquakes are damaging and most important for engineering and other applications. Significantly, none of the above magnitudes provides information about the physical nature of an earthquake source such as the length, width and slip of fault rupture. In order to solve the above limitations, seismologists came up with a new magnitude scale called "moment magnitude," denoted by M. Moment magnitude is based on seismic moment. It does not saturate because it depends on the physical parameters of fault. It is a very robust estimate of earthquake size, from very small to largest sizes, in a uniform manner. The overall character of fault rupture process is considered here. M,, can be correlated to other magnitudes using derived relationships. Moment magnitudes are used for estimating potential earthquake sizes on young faults for earthquake hazard estimates. Based on the evaluation of the lengths and widths of faults, it is a straightforwardmanner to estimate moment magnitudes of maximum credible earthquakes (MCEs) for particular faults. These magnitudes are used for scaling ground motion parameters such as peak accelerations for given project site locations. Peak accelerations are obtained from attenuation relationships, which also used moment magnitude for their development. Most modem seismic hazard estimates use moment magnitude for estimating potential earthquake sizes and the associated strong ground motions.

C3.3 IMPORTANCE CLASSIFICATION The Importance Classification (IC) is used in conjunction with the Acceleration Coefficient (A) to determine the Seismic Performance Category (SPC) for bridges with an Acceleration Coefficient greater than 0.29. The SPC controls the degree of complexity and sophistication of the analysis and design requirements. Two Importance Classifications are specified. An IC of I is assigned for essential bridges and 11 for all others. Essential bridges are those that must continue to function after an earthquake.

The determination of the Importance Classification of a bridge is necessarily subjective. Consideration should be given to the following SociaVSurvival and Security Defense requirements. An additional consideration would be average annual dzily tr c.

The Social/Survival evaluation is largely concerned with the need for roadways during the period immediately following an earthquake. In order for civil defense, police, fire department or public health agencies to respond to a disaster situation a continuous route must be provided. Bridges on such routes should be classified as essential. Survival and mitigation of the effects of the earthquake are of primary concern following a seismic event.

Transportation routes to critical facilities such as hospitals, police and fire stations and communication centers must continue to function and bridges required for this purpose should,beclassified as essential. In addition, a bridge that has the potential to impede traffic if it collapses onto an essential route should also be classified as essential. The health and well-being of the community is another major concern. Victims with critical injuries or illnesses 1998 COMMENTARY must be treated; food, water and shelter provided and utilities restored. Routes to such facilities as schools and arenas, which could provide shelter or be converted to aid stations must suffer little or no damage and bridges on such routes should be classified as essential. Access must be available to power installations and water treatment plants, and bridges required for these purposes should also be classified as essential. The importance evaluation of a bridge of SocialISurviva1 significance in a disaster situation depends on the range of options available and the possibility of a bridge being in parallel or series with other bridges in a roadway network. Discussion may be required with highway, civil defense and police officials. A basis for the Security Defense evaluation is the 1973 Federal-Aid Highway Act which required that a plan for defense highways be developed by each state. This plan had to include, as a minimum, the Interstate and FederalAid Primary routes; however, some of these routes can be deleted when such action is considered appropriate by a state. The defense highway network provides connecting routes to active military installations, industries and resources not covered by the Federal-Air Primary routes and includes: 1. Military bases and supply depots and National Guard installations. 2. Hospitals, medical supply centers and emergency depots. 3. Major airports. 4. Defense industries and those industries that could easily or logically be converted to such. 5. Refineries, fuel storage, and distribution centers. 6. Major railroad terminals, railheads, docks, and truck terminals. 7. Major power plants including nuclear power facilities and hydroelectric centers at major dams. 8. Major communication centers. 9. Other facilities that the state considers important from a national defense viewpoint or during emergencies resulting from natural disasters or other unforeseen circumstances. Bridges serve as important links in the Security1 Defense roadway network and such bridges should be classified as essential. C3.4 SEISMIC PERFORMANCE CATEGORIES The basic premise in developing these Specifications was that they be applicable to all parts of the United States. The seismic risk varies from very small to high C-49 across the country and design requirements applicable to the higher risk areas are not always appropriate for the lower risk areas. In order to provide flexibility in specifying design provisions associated with areas of different seismic risk, four Seismic Performance Categories (SPC) were defined.

The four categories permit variation in the requirements for methods of analysis, minimum support lengths, column design details, foundation and abutment design requirements in accordance with the seismic risk associated with a particular bridge location.

The Seismic Performance Category is determined from the Importance Classification of Article 3.3 and the Acceleration Coefficient of Article 3.2. Thus the importance of a bridge in a road network and the level of seismic exposure at a bridge site are used to determine the SPC. Different degrees of complexity in analysis and design requirements are specified for each SPC. Bridges classified as SPC D are those designed for the highest level of seismic performance and bridges classified as SPC A are those designed for the lowest level of seismic performance. C3.5 SITE EFFECTS See Article C3.2(F). C3.6 ELASTIC SEISMIC RESPONSE COEFFICIENT See Article C3.2(F). C3.7 RESPONSE MODIFICATION FACTORS Response modification factors (R) shown in Table 3.7 are used to modify the component forces obtained from the elastic analysis. Inherent in the R values is the assumption that columns will yield when subjected to forces induced by the design ground motions and that connections and foundations are designed to accommodate the design ground motion forces with little, if any, damage. The rationale used in the development of the R-Factors for columns, piers and pile bents was based on considerations of redundancy and ductility provided by the various supports. The wall type pier was judged to have minimal ductility capacity and redundancy in its strong direction and was assigned an R-Factor of 2. Amultiple column bent with well-detailed columns, as specified in Sections 6 and 7, was judged to have good ductility capacity and redundancy and was assigned the highest value of 5. The ductility capacity of single columns is similar to that of columns in a multiple column bent; however, there is no redundancy C-50 HIGHWAY BRIDGES and therefore a lower R-Factor of 3 was assigned to single columns to provide a level of performance similar to that of multiple column bents. Unfortunately little information was available on the performance of pile bent substructures in actual earthquakes and the R-Factors were based on an assessment of potential pile bent performance in comparison to that of the other three types of substructure. It was believed that there would be a reduction in the ductility capacity of pile bents with batter piles and therefore lower R-Factors were assigned to these systems. The R-Factors of 1.0 and 0.8 assigned to connections mean that the connections are designed for the design elastic forces and for greater-than-the-designelastic forces in the case of abutments. This approach was adopted in part to accommodate the redistribution of forces that occurs when a bridge responds inelastically.llThe other reason for adopting these values was to maintain the overall integrity of the bridge structure at these important joints. Increased protection can be obtained for a minimum increase in construction cost by designing connections for these large force levels. However, it should be noted that for bridges classified as SPC C and D the recommended design forces for column connections are the forces that can be developed by plastic hinging of the columns. Since these are the maximum forces that can be developed and are generally smaller than the elastic values, the desired integrity will be obtained at lower cost. The connection design forces associated with plastic hinging are not calculated for bridges classified as SPC B because plastic hinging requires a more detailed analysis. See Article C7.2.2 for additional commentary on this topic. would be subjected if it responded elastically and the actual ground motion had similar characteristics to the design ground motion. Thus, the displacements resulting from this analysis are used as a lower bound for the design displacements. C3.9 COMBINATION OF ORTHOGONAL SEISMIC FORCES The method of combining forces for each of the load cases is given by means of an example. The two principal transverse axes of a column, abutment or pier, may be designated as the z and y axes.

The shear (V), moment (M), and axial (P) forces resulting from an analysis of the bridge subjected to loads in the transverse direction are designated as: VT,V,T,Mr,M;, and PT, respectively.

The corresponding forces resulting from an analysis of loads in the longitudinal direction are designated VE,Vk,Mk,M,L, and PL respectively. The design shear (V,D,V,D),moment (M:,M,D) and axial (PD) forces for the z and y axes of the member for the two load cases are as follows: LOAD CASE 1 M?= I.oIMEI + 0.31~T1 + MF= 1.01~)I 0.31~TI PD = l.OIPLI C3.8 DETERMLNATION OF ELASTIC FORCES AND DISPLACEMENTS Current knowledge of earthquake ground motions indicates that structures will be subjected to simultaneous ground motion in three orthogonal directions.12For many bridges the effect of the vertical component of motion may not be important and a detailed analysis in the vertical direction is not required. However, for bridges classified as SPC C and D, the effect is accounted for by the design requirements of Article 7.2.5(B). To account for the two horizontal components of motion, an analysis is required in two orthogonal directions, generally the longitudinal and transverse directions of the bridge. Forces and moments resulting from these analyses are then combined as specified in Article 3.9 to account for the simultaneous occurrence of forces in two horizontal directions. The forces and displacements obtained from an elastic analysis should be similar to those to which the bridge + 0.31~~1 LOAD CASE 2 vp = 0.31v;I + 1.01vrI + 1.0lv;l M?= 0.31MkI + 1.OIMFI MF= 0.31~)I+ 1.01M;SI PD = 0.31PLI + 1.01~~1 VF = 0.3lV)l where the symbol 11 denotes the absolute value or the magnitude of the force or moment without regard to its sign, since a seismic force can act in either direction. It should be noted that, for a straight bridge with no skewed piers, columns or abutments, the above combinations simplify significantly because a transverse load will primarily produce moments and shear forces in the "z" direction of the structural member and the longitudinal load will primarily produce moments and shear forces in the "y" direction. 1998 COMMENTARY The exception to these load combinations indicated at the end of this section, should also apply to SPC-B bridges when foundation forces are determined from plastic hinging of the columns. (See C6.2.) C3.10 MINIMUM SEAT WIDTH REQUIREMENTS In developing these Specifications, design displacements were considered to be as important as design forces because many of the loss-of-span failures in past earthquakes have been attributed in part to relative displacement effects. The length of support provided at abutments, columns and hinge seats must accommodate displacements resulting from the overall inelastic response of the bridge structure, possible independent movement of different parts of the substructure, and out-of-phase rotation of abutments and columns resulting from traveling surface wave motions. The minimum support length also provides for possible translation and rotation of the footings due to ground failure andlor deformations due to liquefaction. A reasonable estimate of the displacements resulting from the overall elastic dynamic response of the bridge structure can be obtained from the multimode spectral method of analysis if the flexibility of the foundations is included. Better estimates can be obtained if an inelastic time history analysis is performed; however, this is not recommended in these Specifications because of the complexities involved in performing this method of analysis. Either the elastic or inelastic time history analysis will give reasonable estimates of the out-of-phase movements of different parts of the substructure whereas the multimode method of spectral analysis will not.

The recent work of Elms et al13J4 can be used to give the order of magnitude of abutment movement and the recent work of Werner et al15J6 gives some indication of the effects of traveling waves on the responses of a limited number of bridges. However, much research remains to be done in both these areasi2. In summary, the current state of the art precludes a good estimate of the differential column and abutment displacements to be expected when a bridge is subjected to an earthquake. It is therefore prudent to specify minimum support lengths at abutments, piers and hinge seats to provide for the effects discussed above. If the displacements resulting from the elastic analysis of Article 3.8 exceed the minimum specified values, the values resulting from the elastic analysis must be used in the design. The minimum support lengths specified are dependent on the deck length between expansion joints and the column height since both dimensions influence one or more of the C-51 factors that cause the differentialdisplacements.Although a considerable amount of judgment was exercised when developing these minimum requirements using current knowledge, these criteria should be refined as the state of the art develops. C3.11 DESIGN REQUIREMENTS FOR SINGLE SPAN BRIDGES Requirements for single span bridges are not as rigorous as for multi-span bridges because of their favorable response to seismic loads in past earthquakes. As a result, single span bridges need not be analyzed for seismic loads regardless of the SPC and design requirements are limited to minimum seat widths and connection forces. Adequate seat widths must be provided in both the transverse and longitudinal directions. Connection forces based on the premise that the bridge is very stiff and that the fundamental period of response will be short. This assumption acknowledges the fact that the period of vibration is cult to calculate because of significant interaction with the abutments. These reduced requirements are also based on the assumption that there are no vulnerable substructures (i.e., no columns) and that a rigid (or near rigid) superstructure is in place to distribute the in-plane loads to the abutments. If, however, the superstructure is not able to act as a stiff diaphragm and sustains significant in-plane deformation during horizontal loading, it should be analyzed for these loads and designed accordingly. Single span trusses may be sensitive to in-plane loads and the designer may need to take additional precautions to ensure the safety of truss superstructures. C3.12 REQUIREMENTS FOR TEMPORARY BRIDGES Temporary bridges are not exempt from the seismic design requirements of this specification.

However, in view of their short life, they have less exposure to the seismic hazard, and the Acceleration Coefficient, A, may be reduced by a factor of 2 when calculating the lateral forces that such a bridge can be reasonably expected to sustain. Exceptions exist for bridges in high seismic zones and close to active faults which require site specific studies. Furthermore, the performance criteria assumed for conventional bridges may be relaxed for these temporary structures and certain Response Modification Factors may therefore be increased by up to 50% (i.e., by a factor of 1.5). It is noted that in low-to-moderate seismic zones, wind loads may well exceed these reduced seismic loads and thus govern design for lateral loads.

In this event, care C-52 HIGHWAY BRIDGES should be.taken with the design of the foundations as noted in Articles 6.2 and C6.2. 12. REFERENCES 1. Applied Technology Council, "Tentative Provisions for the Development of Seismic Regulations for Buildings," ATC Report No. ATC-3-06, Berkeley, CA, June 1978. 2. American Association of State Highway and Transportation Officials, "Guide Specifications for Seismic Design of Highway Bridges," AASHTO, Washington, DC, 1983.

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5. Federal Emergency Management Agency, "NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings,"Part 1: Provisions, FEMA Report 95; Part 2: Commentary, FEMA Report 96; Washington, DC 1988. 6. Newmark, N.M. and Hall, W.J., "Seismic Design Criteria for Nuclear Reactor Facilities," Proceedings of 4th World Conference on Earthquake Engineering, Santiago, Chile, 1969. 7. Seed, H.B., Ugas, C., and Lysmer, J., "Site Dependent Spectra for Earthquake Resistant Design," Bulletin of the Seismological Society of America, Vol. 66, No. 1, February 1976. 8. Blume, J.A., Sharpe, R.L., and Dalal, J., "Recommendations for the Shape of Earthquake Response Spectra," report prepared for the Directorate of Licensing, United States Atomic Energy Commission, February 1973. 9. Newmark, N.M., Hall, W.J., and Morhaz, B., "A Study of Vertical and Horizontal Spectra," report prepared for the Directorate of Licensing, United States Atomic Energy Commission, Report No. WASH1255, April 1973. 10. Mohraz, B., "A Study of Earthquake Response Spectra for Different Geological Conditions," Bulletin of Seismological Society of America, Vol. 66, No. 3, June 1973. 11. Imbsen, R.A., Nutt, R.V., and Penzien, J., "Evaluation of Analytical Procedures Used in Bridge Seismic Design Practice," Proceedings of a Workshop on Earth- 13. 14. 15. 16. quake Resistance of Highway Bridges, Applied Technology Council, Berkeley, CA, January 1979.

Hall, W.J. and Newmark, N.M., "Seismic Design of Bridges-An Overview of Research Needs," Proceedings of a Workshop on Earthquake Resistance of Highway Bridges, Applied Technology Council, Berkeley, CA, January 1979. Richards, R. and Elms, D.G., "Seismic Behavior of Retaining Walls and Bridge Abutments," Report No. 77-10, University of Canterbury, Christchurch, New Zealand, June 1977. Elms, D.G. and Martin, G.R., "Factors Involved in the Seismic Design of Bridge Abutments," Proceedings of a Workshop on Earthquake Resistance of Highway Bridges, Applied Technology Council, Berkeley, CA, January 1979. Werner, S.D., Lee, L.C., Wong, L.H., and Trifunac, M.D., "An Evaluation of the Effects of Travelling Seismic Waves on the Three Dimensional Response of Structures,"Agbabian and Associates, El Segundo, CA, October 1977. Werner, S.D., Lee, L.C., Wong, L.H., and Trifunac, M.D., "Effects of Traveling Waves on the Response of Bridges," Proceedings of a Workshop on Earthquake Resistance of Highway Bridges,Applied Technology Council, Berkeley, CA, January 1979. Commentary SECTION 4--ANALYSIS REQUIREMENTS C4.1 GENERAL This section of the Specifications presents four analytical procedures to determine the distribution of forces for the prescribed seismic loadings.

All are based on linear elastic analysis techniques. C4.2 SELECTION OF ANALYSIS METHOD An elastic analysis procedure is used for the seismic design of bridges to give the designer an indication of the force distribution to the structural members and to give him or her some indication of the relative deformations. It also provides the basis for the design of the components. The actual forces and displacements in bridges subjected to the design ground motions may be quite different from those obtained from the elastic analysis because at these high levels of excitation the bridge may respond inelastically. Procedures 1 and 2 both assume that the seismic response of a bridge can be represented by a single mode of vibration when in actual fact bridges have many possible 1998 COR vibration modes and most will participate to some degree when responding to an earthquake. However, for "regular" bridges one mode usually dominates (in the direction of the earthquake) and this mode is sometimes called the fundamental mode. Both Procedures require the calculation of the period (T) of this mode from which a reasonable estimate of the elastic forces and displacements can then be made using standard analytical methods. The principal difference between the two methods is the way in which the fundamental period is calculated; Procedure 1 is not as rigorous as Procedure 2 but it is more intuitive in its approach and easier to apply. However, both methods are approximate and the limits on their application must be clearly understood and observed. Table 4.2B defines a "regular" bridge for which both methods are applicable. The limits prescribed in Table 4.2B were determined after reviewing the results of a limited parameter study on 27 bridges which included continuous 2, 3, and 6 span structures. Eleven of these bridges were straight and 16 were curved (subtending angles of arc of either 40° or 80° at the center of curvature). Ratios between adjacent pier stiffnesses ranged from unity up to 8.0. Force and displacement results were obtained from Procedures 1 and 2 for each bridge and compared against corresponding results from the multi-mode method (Procedure 3). Reference 1 describes this parameter study in more detail. It is noted that in developing the provisions in Table 4.2B, the results of this parameter study were modified to permit up to 6-span bridges to be considered "regular" provided that tighter restrictions on span-length ratio and pier-stiffness ratio were imposed on these longer bridges. Furthermore, Reference 1 also describes a similar parameter study on simplysupported 2,3, and 6 span bridges. The results of this study show that the requirements of Table 4.2B are unconservative for curved multiple simple span bridges with a subtended angle in plan greater than 20°, and the use of simplified methods of analysis for such bridges is not allowed. The results of this second parameter study also show that particular care should be taken when applying simplified methods of analysis to straight simple span bridges when calculating response to longitudinal earthquake loads. In this situation, the bridge should be analyzed in segments, where a segment is defined as one span of a multiple simple span bridge.

Similar care should also be taken with straight bridges comprised of 2 or more sections of continuous girders. In this case, a segment is defined as that section of superstructure that is continuous from one expansion joint to the next or from one abutment to the closest expansion joint. Whereas further studies are required, the limits in Table 4.2B are believed to be adequate in most situations and should not underestimate the governing design forces and displacements by more than 10%. These errors are of little consequence once flexural yielding commences in the columns. Higher modes of vibration may be important in bridges that are "not regular" and a multimode method of analysis is then required to adequately describe the response. Procedure 3 is one such method which is based on combining the individual responses from each of the participating modes subject to the same response spectrum. This "multimode spectral analysis" method does not directly account for the phase relationships between the modes of vibration but instead combines the modes by a statistical approach known as the complete quadratic combination method (CQC method). The Procedure is very efficient and can be used with confidenceto analyze almost any elastic structure for which the dynamic loads are specified by response spectra. Appropriate computer software is readily available. Procedure 4 also addresses higher mode effects but by performing a step-by-step time history analysis of response. Phasing between modes is directly included in the solution and no modal combination rules are required. The solution is the most rigorous of the four Procedures specified but it is also the most computationally intensive. Again computer software is available and application to nonlinear, inelastic analysis is feasible. A major limitation of the method is the need to know the time history of the design ground motion to be used for the analysis. This is not specified in the current Specification and must be determined by a site-specific study by a qualified professional. Furthermore, it is generally considered that one time history is insufficient to represent all possible responses and the current specification therefore calls for at least five analyses using five different input time histories. Each of the above Procedures has strengths and limitations. Increased rigor from Procedure 1 to Procedure 4 is accompanied by increased effort and less intuitive feel for the analytical process or the results. Also greater care in modelling is required. Nevertheless, important andlor irregular bridges deserve careful analysis and the effort spent on more rigorous analysis may prevent a failure due to an unanticipated response or an unexpected load path. It may also save construction costs and particularly so in the foundations where careful modelling has been shown to lead to substantial savings. For bridges classified as SPC C and D with three or more spans, the designer should consider including the flexibility of the foundations and abutments in the analysis. C4.3 UNIFORM LOAD METHODPROCEDURE 1 The uniform load method is an equivalentstatic method of analysis that may be used to calculate seismic forces C-54 HIGHWAY BRIDGES and displacements in bridges provided the structure is "regular" and responds predominately in its first transverse mode of vibration during an earthquake. It is further assumed that the shape of this mode is given by the deflected shape of the bridge when subject to a uniform horizontal load acting at the level of the superstructure. It is also assumed that the period of this mode can be calculated using the total weight of the bridge and the maximum deflection obtained under the uniform load. In fact, seismic loads are not uniformly distributed along the length of a bridge even if it has uniform weight, because inertia loads are also proportional to acceleration which vary with the deflected shape. However, reasonably good results can be obtained by this method for regular bridges due to some compensatory effects in the modelling. But it is known to overestimate the transverse shear forces at the abutments. This is because Equation (4-4) overestimates the total base shear (i.e., the total earthquake load acting on the bridge) by about 20 to 25% when compared to say the total shear calculated from Equation (4-9) (the single mode method for regular structures). The situation is further aggravated if the lateral stiffnesses of the intermediate piers are high compared to the in-plane flexural stiffness of the superstructurein which case the abutment shears, by the uniform load method, are even higher than suggested above. In extreme cases this overestimation can reach 100%. Compensatory effects in the uniform load method mean that this conservatism at the abutments is not seen elsewhere in the structure nor is there a corresponding under-prediction elsewhere in the structure. It occurs because the uniform load method has more load on the structure than the more "exact" methods and this extra load is mainly in the end spans close to the abutments. If this effect is undesirable, then the single mode method of analysis (Article 4.4) is recommended. C4.4 SINGLE MODE SPECTRAL ANALYSIS METHOD-PROCEDURE 2 The single mode spectral analysis method is used to calculate the seismic design forces for bridges that respond predominately in the fist mode of vibration.

The method, although completely rigorous from a structural dynamics point of view, reduces to a problem in statics after the introduction of inertia forces. The method, as formulated, can be applied to many types of bridges which have both continuous and noncontinuous superstructures. Boundary conditions at the abutments and piers can also be modeled to include the effects of foundation flexibility. Bridges are generally continuous systems consisting of many components which contribute to the overall resistance capacity of the system. Consider a bridge subjected FIGURE C4.4A Plan View of a Bridge Subjected to a Transverse Earthquake Motion to a transverse earthquake ground motion. The bridge is composed of several spans restrained transversely at the end abutments and intermediate piers, as shown in Figure C4.4A. Typically the bridge deck may have expansion joints at the piers or within the spans. These expansion joints do not have the capability to transmit transverse deck moments between adjacent deck sections. The equation of motion for a continuous system representing this system is conveniently formulated using energy principles. The principle of virtual displacements may be used to formulate a generalized parameter model of a continuous system in a manner which approximates the overall behavior of the system. Assuming transverse motion in a single mode shape, a single degree-of-freedom "generalized parameter" model may be formulated.

To obtain an approximation to this mode shape, a uniform static loading, p,, is applied to the superstructure and the resulting deflection, vs(x), is obtained. The dynamic deflection, v(x,t), of the structure under seismic excitation as shown in Figure C4.4B is then approximated by the shape function multiplied by a generalized amplitude function, v(t), as shown by Equation (C4-1). This function will describe the deformed bridge structure in a manner which is consistent with the support conditions and intermediate expansion joint hinges in the deck.

Note that it is an admissible function which satisfies the geometric boundary conditions of the system. Initially, to establish the deflected shape for the generalized parameter model, apply a uniform loading p, to the structure as shown in Figure C4.4C.

Assume that the loading is applied gradually so that the kinetic energy of the FIGURE C4.4B Displacement Function Describing the Transverse Position of the Bridge Deck 1998 COMMENTARY where: FIGURE C4.4C Deflected Shape Due to Uniform Static Loading mass of the structure is zero. The external work, WE,done by the uniformly applied loading in deforming the structure is given by: and o is the frequency of the vibrating system. The factor y defined in

and able to transfer lateral loads to the substructure. It is important that not only the bearing but also the details that fasten the bearing to the sole and masonry plates (including the anchor bolts which engage the supporting members), have sufficient capacity to resist the above forces. At a fixed bearing, it is necessary to consider the simultaneous application of the longitudinal and transverse connection forces when checking these capacities.
Note that the primary purpose of this requirement is to ensure that the connections between the superstructure and its supporting substructures remain intact during the design earthquake and thus protect the girders from being unseated. The failure of these connections has been observed in many earthquakes and imposing minimum strength requirements is considered to be a simple but effective strategy to minimize the risk of collapse. However, in low-seismic zones such as SPC A it is not necessary to design the substructures or their foundations for these forces since it is expected that if a column does yield it will have sufficient inherent ductility to survive without collapse. Even though bridge columns in SPC A are not required to be designed for seismic loads, default reinforcement requirements will provide a minimum level of capacity for ductile deformations which is considered to be adequate for the magnitude and duration of the ground motions expected in SPC A. 1998 COMMENTARY This has been recently confirmed by the dynamic testing of a full-scale bridge column taken from a bridge in the Eastern United States which had not been designed for seismic loads and to which demonstrated surprisingly ductile performance. C53 DESIGN DISPLACEMENTS FOR SEISMIC PERFORMANCE CATEGORY A The rationale for these minimum seat width requirements for bridges in all seismic performance categories has been given in Article C3.10. Since an elastic analysis is not required for bridges classified as SPC A the minimum support lengths specified in Article 5.3 are the only design displacement requirements for these bridges. Skewed bridges are known to develop large displacements at their supports due to the superstructure rotating about a vertical axis through the center of stiffness of the substructure. This has been observed in many earthquakes and the relatively frequent occurrence of damaged and unseated skewed spans is attributed to this phenomenon. For this reason, the seat width for skewed spans is increased in proportion to the square of the angle of skew. This allowance for skew increases the minimum width (N in Equation (5-1)) by approximately 25% for bridges with skew angles of 45". Note that N is measured normal to the leading edge of the seat and thus the minimum width parallel to the bridge centerlineis larger than N and given by N times the secant of the angle of skew. C5.4, C5.5, AND C5.6 FOUNDATION AND ABUTMENT DESIGN, STEEL AND CONCRETE DESIGN REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY A Consistent with the overafl philosophy for bridges classified as SPC A, special seismic design requirements are not specified because of the low level of seismic risk and the low probability that a foundation or a column will be subjected to seismic forces that will cause yielding. REFERENCE 1. Mander, J., Mahmoodzadegan, B., Bhadra, S., and Chen, S.S., "Seismic Evaluation of a 30-Year Old Non-Ductile Highway Bridge Pier and Its Retrofit," Technical Report NCEER-96-0008, National Center for Earthquake Engineering Research, Buffalo, NY, 1996. C-61 Commentary SECTION 6DESIGN REQUIREMENTS FOR BRIDGES IN SEISMIC PERFORMANCE CATEGORY B C6.2 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORY B The seismic design forces specified for bridges classified as SPC B areTntended to be relatively simple but consistent with the overall design concepts and methodology. Inherent in any simplification of a design procedure, however, is a degree of conservatism and for SPC B this occurs in the determination of the design forces for the foundations and connections to columns. If these forces appear to be excessive, then the method specified for bridges classified as SPC C and D in Articles 7.2.5 and 7.2.6 should be used. The major difference is that, for SPC C and D, foundations and connections to columns are designed for the maximum forces that a column can transmit to these components. In some cases, these may be considerably less than the design forces specified in Article 6.2. Article 6.2.1 specifies the design forces for the structural components of the bridge. In the first step, the elastic forces of Load Cases 1 and 2 of Article 3.9 are divided by the appropriate R-Factors of Article 3.7. These forces are combined with those from other loads and the group loading combination is the same as that used in the current AASHTO Specifications with all y and P factors equal to 1.0. Furthermore, each component shall be designed to resist the two seismic group load combinations of Article 3.9, one including Load Case 1 and the other including Load Case 2. Each load case incorporates different proportions of bi-directional seismic loading. This may be important for some components (e.g., biaxial design of columns) and unimportantfor others. In the design loads for each component-the sign of the seismic forces and moments obtained from Article 3.9 can be taken as either positive or negative. The sign of the seismic force or moment that gives the maximum magnitude for the design force (either positive or negative) shall be used.
Either the load factor or service load method of design as specified in Division I can be used with the specified forces. For essential bridges in SPC B, a designer may wish to consider the column design requirements for SPC C and D in Section 7 to enhance the column ductility capacity. However, for most bridges, the Division I requirements and the additional requirements of Article 6.6 were deemed reasonable in view of the seismic risk level associated with SPC B. The ductility capacity of a column designed to Division I is difficult to estimate because the C-62 HIGHWAY BRIDGES potential mode of failure could be shear, flexure, compression, or loss of anchorage, or a combination of any two or more. The design requirements of Section 7 for bridges classified as SPC C and D are specified such that the potential for a shear, compression or loss of anchorage mode of failure is minimized and the column is forced to yield in flexure with reasonable ductility capacity when subjected to significant seismic force levels. Article 6.2.2 specifies the design forces for foundations which include the footings, pile caps and piles.
The design forces are essentially twice the seismic design forces of the columns. This will generally be conservative and was adopted to simplify the design procedure for bridges classified as SPC B. However, if seismic forces do not govern the design of columns and piers there is a possibility that during an earthquake the foundations will be subjected to forces larger than the design forces. For example this may occur due to unintended column overstrengths which may exceed the capacity of the foundations.
An estimate of this effect may be found by using overstrength factors of 1.3 for reinforced concrete columns and 1.25 for steel columns. It is also possible that even in cases when seismic loads govern the column design, the columns may have insufficient shear strength to enable a ductile flexural mechanism to develop but instead allow a brittle shear failure to occur. Again this situation is due to potential overstrengthin the flexural capacity of columns and could possibly be prevented by arbitrarily increasing the column design shear by the overstrength factor (1.3 for concrete columns and 1.25 for steel columns). Conservatism in the design, and in some cases underdesign, of foundations and columns in SPC B based on the simplified procedure of this Article has been widely debated. (See for example Gajer and Wagh5.) In light of the above discussion it is recommended that for important bridges classified as SPC B consideration should be given to the use of the forces specified in Article 7.2.6 for foundations in SPC C and D.
It should be noted that ultimate soil and pile strengths are to be used with the specified foundation seismic design forces. C6.3 DESIGN DISPLACEMENTS FOR SEISMIC PERFORMANCE CATEGORY B For bridges classified as SPC B, the design displacements are specified as either the maximum of those calculated by the elastic analysis of Article 3.8 or the minimum specified support lengths given by Equation (6-3). This "either/or" specification was introduced to account for larger displacements that may occur from the analysis of more flexible bridges. Displacements obtained from the elastic analysis of bridges should provide a reasonable es- timate of the displacements resulting from the inelastic response of the bridge. However, it must be recognized that displacements are very sensitive to the flexibility of the foundation and if the foundation is not included in the elastic analysis of Article 3.8 consideration should be given to increasing the specified displacements for bridges founded on very soft soils. This increase may be of the order of 50% or more but as with any generalization, considerable judgment is required. A better method is to determine upper and lower bounds from an elastic analysis which incorporates foundation flexibility. Special care in regard to foundation flexibility is required for bridges with high piers. The rationale for these minimum seat width requirements is given in Article C3.10. Skewed bridges are known to develop large displacements at their supports due to the superstructure rotating about a vertical axis through the center of stiffness of the substructure.
This has been observed in many earthquakes and the relatively frequent occurrence of damaged and unseated skewed spans is attributed to this phenomenon. For this reason, the seat width for skewed spans is increased in proportion to the square of the angle of skew. This allowance for skew increases the minimum width (N in Equation (6-3)) by approximately 25% for bridges with skew angles of 45". Note that N is measured normal to the leading edge of the seat and thus the minimum width parallel to the bridge centerline is larger than N and given by N times the secant of the angle of skew. C6.4 FOUNDATIONS AND ABUTMENT DESIGN REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY B Since the design of bridge foundations in SPC B is essentially the same as for bridges in SPC C and D, a joint commentary is provided in Appendix A.
C6.5 STRUCTURAL STEEL DESIGN REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY B C6.5.1 General The 50% increase in allowable stresses for service load design is based on the following: 1.
The margin of safety between the yield strength and allowable stress of short columns. 2.
The margin of safety between the yield strength and allowable tensile stress. 3. The margin of safety of compression members, which varies between 1.7 and 1 .9.1.2 1998 COMMENTARY It is noted that seismic design requirements for steel bridges are currently under review and it is likely that the provisions given in these articles will be modified in the near future.
Some indication of the likely trends in seismic design of steel bridges may be found in Report ATC-32 prepared by the Applied Technology Council for Caltrans6 and in a report prepared for the American Iron and Steel Institute Task Force on Seismic Design by A. Astaneh7. C6.5.2 P-delta Effects This Article provides modifications to the interaction equations when the P-delta effects are explicitly determined. In columns, the reductions to the allowable stresses are in part a result of the consideration of member P-delta effects. These P-delta reductions are modified in AASHTO by a K-factor which is arecognition of the effect of end restraint in the member P-delta relationship. The bases for the values of this ratio where joint translation is prevented are well documented. The selection of the value of C, where joint translation is permitted was an approximation applicable primarily to designs for which significant applied horizontal forces are not present. Since the advent of computer analysis, the solution of the interaction equations when secondary effects resulting from deflection are taken into account, has become much easier. In most cases, with significant horizontal displacements,the first iteration of deflection is sufficient. It is possible that for some members, such as weak axis columns depending on end-support conditions, critical stress may occur at the midheight rather than the column ends. Thus the stress limits specified when joint translation is prevented should not be exceeded. C6.6 REINFORCED CONCRETE REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY B Bridges classified as SPC B have a reasonable probability of being subjected to lower level seismic forces that will cause localized yielding of the columns. Thus, it was deemed necessary that columns have some ductility capacity although it was recognized that the ductility demand will not be as great as for columns of bridges classified as SPC C and D. The most important requirement to ensure some level of ductility is the transverse reinforcement requirement specified3s4.This will prevent buckling of the longitudinal steel and provide confinement for the core of the column. The maximum spacing for the transverse reinforcement was increased to 6 in. (150 mm) because of the anticipated lower ductility demand. C-63 REFERENCES 1. "Manual of Steel Construction," American Institute of Steel Construction, Inc., 1979. 2. Johnson, B.G., Structural Stability Research Council's "Guide to Stability Design Criteria for Metal Structures," Third Edition, John Wiley and Sons, New York, 1976. 3. Priestley, M.J.N. and Park, R., "Seismic Resistance of Reinforced Concrete Bridge Columns," Proceedings of a Workshop on the Earthquake Resistance of Highway Bridges, Applied Technology Council, Berkeley, CA, January 1979. 4. Jirsa, J.O., "Applicability to Bridges of Experimental Seismic Test Results Performed on Subassemblages of Buildings," Proceedings of a Workshop on the Earthquake Resistance of Highway Bridges, Applied Technology Council, Berkeley, CAYJanuary 1979. 5. Gajer, R.B., and Wagh, V.P., "Bridge Design for Seismic Perfoniance Category B: The Problem With Foundation Design," Proc 1 lth Ann1 Intl Bridge Conf., Paper IBC-94-62, Pittsburgh, PA, 1994.
6. Applied Technology Council, "Recommended Revisions of Caltrans Seismic Design Procedures for Bridges," Report ATC-32, 1996 (in press). 7. A. Astaneh, "Seismic Behavior and Design of Steel Bridges-Response Modification Factor Based Design," Report to American Iron and Steel Institute Task Force on Seismic Design, 1995. Commentary SECTION 7-DESIGN REQUIREMENTS FOR BRIDGES IN SEISMIC PERFORMANCE CATEGORIES C AND D C7.2 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORIES C AND D For bridges classified as SPC C and D two sets of design forces are defined and either one, or both sets, is specified as the design force for different components. If two sets are specified, the designer has an eitherlor option with one set being more conservative than the other. The major difference between these design forces and those specified for bridges classified as SPC B is that one set of these defined forces corresponds to forces resulting from plastic hinging in the columns. The design forces for the various components are specified in Articles 7.2.3 to 7.2.7. C-64 HIGHWAY BRIDGES p -- - -~~ ~ C7.2.1 Modified Design Forces Article 7.2.1 defines the modified design forces which are used for the design of some components of the bridge. In the first step the elastic forces of Load Cases 1 and 2 of Article 3.9 are divided by the appropriate R-Factor of Article 3.7. In combining these forces with those from other load types, the group loading combination is the same as that used in Group VII of the current Division I with all y and p factors equal to 1.0. Two seismic group load combinations are defined; one for Load Case 1 of Article 3.9 and the other for Load Case 2.
This may be important for some components (e.g., biaxial design of columns) and unimportant for others.
In the design loads for each componeit for the group load combination, the sign of the seismic forces and moments obtained from Article 3.9 can be either positive or negative. The sign of the seismic force or moment that gives the maximum magnitude for the design force (either positive or negative) shall be used. The exception to this is for column axial loads in which the seismic axial load is considered alternately as a positive and negative load so that a minimum and maximum axial force is calculated for each load case. Column Size and Reinforcement Configuration The design engineer should select the minimum column section size and steel reinforcement ratio when satisfying structural design requirements.
As these parameters increase, the overstrength capacity increases. This may lead to an increase in the foundation size and cost. Column size will also influence whether a column is functioning above or below Pb(Articles 8.1.2 and 8.16.4.2.3, Division I). A size and reinforcement ratio which forces the design below Pbis preferable, especially in high seismic areas. However, the selection of size and reinforcement must also satisfy architectural, and perhaps other requirements, which may govern the design. Increase in Reinforcement Strength Almost all reinforcing bars will have a yield strength larger than the minimum specified value (up to 30% higher, with an average increase of 12%). Combining this increase with the effect of strain hardening, it is realistic to assume an increased yield strength of 1.25fy, when computing the column overstrengthl.
Increase in Concrete Strength C7.2.2 Forces Resulting from Plastic Hinging in Columns, Piers, or Bents Article 7.2.2 defines the forces resulting from plastic hinging (a column reaching its ultimate moment capacity) in the columns and presents two procedures. One is for a single-column hinging about its two principal axes; this is also applicable for piers and bents acting as single columns. The other procedure is for a multiple column bent in the plane of the bent. The forces are based on the potential overstrength capacity of the materials and to be valid the design detail requirements of this section must be used so that plastic hinging of the columns can occur. The overstrength capacity results from actual properties being greater than the minimum specified values. This fact must be accounted for when forces generated by yielding of the column are used as design forces. Generally, overstrength capacity depends on the following factors: 1.
The actual size of the column and the actual amount of reinforcing steel. 2. The effect of an increased steel strength over the specified fyand for strain hardening effects. 3. The effect of an increased concrete strength over the specified f: and confinement provided by the transverse steel. Also, with time, concrete will gradually increase in strength. 4. The effect of an actual concrete ultimate compressive strain above 0.003. Concrete strength is defined as the specified 28-day compression strength; this is a low estimate of the strength expected in the field. Typically, conservative concrete batch designs result in actual 28-day strengths of about 20 to 25% higher than specified. Concrete will also continue to gain strength with age. Tests on cores taken from older California bridges (built in the 1950's and 1960's) have consistently yielded compression strengths in excess of 1.5fL. Concrete compression strength is further enhanced by the possible confinement provided by the transverse reinforcement. Rapid loading due to seismic forces could also result in significant increase in strength (strain-rate effect). In view of all the above, the actual concrete strength when a seismic event occurs is likely to significantly exceed the specified 28-day strength. Therefore, an increased concrete strength of 1.5fL could be assumed in the calculation of the column overstrength capa~ity.l-~~~~~
Ultimate Compressive Strain (E,) Although tests on unconfined concrete show 0.003 to be a reasonable strain at first crushing, tests on confined column sections show a marked increase in this value. The use of such a low extreme fiber strain, is a very conservative estimate of strains at which crushing and spalling first develop in most columns, and considerably less than the expected strain at maximum response to the design seismic event. Research has supported strains of the order of 1998 COMMENTARY C-65 0.01 and higher as the likely magnitude of ultimate comproperties than specified, is illustrated by comparing the pressive strain. Therefore, designers could assume a value actual overstrength curve (computed with realistic fk, f, of ultimate strain equal to 0.01 as a realistic value.1~2~3~4J0and E, values) to the nominal strength interaction curve (P,, M,). It is generally satisfactory to approximate the For calculation purposes, the thickness of clear conoverstrength capacity curve by multiplying the nominal crete cover used to compute the section overstrength,shall moment strength by the 1.3 factor for axial loads below Pb not be taken to be greater than 2 inches (50 mm). This re(P,, 1.3Mncurve). However, as shown in the plot, this duced section shall be adequate for all applied loads assocurve may be in considerable error for axial loads above ciated with the plastic hinge. Pb. Therefore, it is recommended that the approximate overstrength curve be obtained by multiplying both P, and Overstrength Capacity M, by c) = 1.3, (1.3Pn, 1.3Mn)'. This curve follows the The derivation of the column overstrength capacity is general shape of the actual curve very closely at all levels depicted in Figure C7.2.2A. The effect of higher material of axial loads. Column proportier: 6 1 W 4 " x 5' 10-U4" d o n A, = 32 Y 1 1 bm (1.05%) c, r0.003 r, = 4000 pi 5 = eo,000 pri 1 Approdmata owntrongth cum using nnngth reduction *cbr or. = 1.3 on both nominal moment urd nominal udal load (lap, la M.) '0000 Actual oventmngrh interaction curve .a. .... * .. Nominal rtmngth inbraction -.... curve (Pw K) 4000 p i , 5 rn 60,000 pri, r, I r **.
**. . wing nalirtic mrrrrirl (over-) strenqtha: f,' r 6000 pri, 5 = 76,000, g, 0.01 I%, 0.008 (K I P-FT) FIGURE C7.2.2A Development of Approximate Overstrength Interaction Curves from Nominal Strength C u ~ e s (after Gaer and Wagh5) C-66 HIGHWAY BRIDGES In the light of the above discussion, it is recommended that: 1. For all bridges with axial loads below Pb,the overstrength moment capacity be assumed to be 1.3 times the nominal moment capacity. 2. For SPC C and D bridges with ImportanceClassification (IC) = I1 and for all SPC B bridges, the overstrength curve for axial loads greater than Pb be approximated by multiplying both P, and M, by = 1.3. 3. For SPC C and D bridges with Importance Classification (IC) = I, the overstrengthcurve for axial loads greater than Pb, be computed using realistic values for f',, fyand E, as recommended in Table C7.2.2A or from values based on actual test results. The overstrength thus calculated should not be less than the value estimated by the approximate curve based on (1.3Pn, 1.3Mn). + TABLE C7.2.2A Recommended Increased Values of Materials Properties Increased fy(minimum) Increased fb Increased E, 1.25 f,. 1.5 f: 0.01 umn is expected to yield when subjected to the forces of the design earthquake. The design axial forces which control both the flexural design of the column and the shear design requirements are either the maximum or minimum of the unreduced design forces or the values corresponding to plastic hinging of the columns. In most cases, the values corresponding to plastic hinging of the columns will be lower than the unreduced design forces. The design shear forces are specified so that the possibility of a shear failure in the column is minimized. C7.2.4 Pier Design Forces The design forces for piers specified in Article 7.2.4 are based on the assumption that a pier has low ductility capacity and no redundancy.
As a result, a low R-Factor of 2 is used in determining the reduced design forces and it is expected that only a small amount of inelastic deformation will occur in the response of a pier when subjected to the forces of the design earthquake. If a pier is designed as a column in its weak direction then both the design forces and, more important, the design requirements of Articles 7.2.3 and 7.6.2 are applicable. C7.2.5 Connection ~ e s i g Forces n Shear Failure The shear mode of failure in a column or pile bent will probably result in a partial or total collapse of the bridge; therefore, the design shear force must be calculated conservatively. In calculating the column or pile bent shear force, consideration must be given to the potential locations of plastic hinges. For flared columns, these may occur at the top or bottom of the flare. For multiple column bents with a partial-height wall, the plastic hinges will probably occur at the top of the wall unless the wall is structurally separated from the column. For columns with deeply embedded foundations, the plastic hinge may occur above the foundation mat or pile cap.
For pile bents the plastic hinge may occur above the calculated point of fixity.
Because of the consequences of a shear failure, it is recommended that conservatism be used in locating possible plastic hinges such that the smallest potential column length be used with the plastic moments to calculate the largest potential shear force for design. C7.2.3 Column and Pile Bent Design Forces The design forces for columns specified in Article 7.2.3 are based on the design philosophy of the Specifications discussed in the Introduction to the Commentary. The design moments are specified on the assumption that the col- Connections are important elements in maintaining the overall integrity of a bridge structure. Therefore, specific attention was given to the displacements that occur at movable supports (Article 7.3) and, for fixed connections, reasonably conservative design forces are specified to provide increased protection for a minimum increase in consmction cost. The recommended design forces specified in Article 7.2.5 are such that column connections are designed for the maximum forces that a column can transmit to the connection (Article 7.2.5(C)). The design forces for other connections and the alternate forces for column connections are the elastic seismic forces specified in Article 7.2.1. Forces greater than the elastic seismic forces are specified in the case of abutment connections. An additional requirement to prevent significant relative displacements at connections is given in Article 7.2.5(A). Positive horizontal linkage shall be provided by cables or an equivalent mechanism. Friction shall not be considered as positive linkage. As a further safety measure, minimum bearing support lengths are required. The problem of relative displacement is discussed in more detail in Article C3.10. Section 7.2.5(B) represents the only provision included in the Specifications to minimize the potential adverse effects of vertical seismic excitation as discussed in Article C3.8. This is a reasonably straightforwardrequirement and 1998 COMMENTARY will be subject to refinement as the state-of-the-art in the effects of vertical seismic excitation develops. C7.2.6 Foundation Design Forces The foundation design forces specified in Article 7.2.6 are consistent with the design philosophy of minimizing damage that would not be readily detectable. The recommended design forces are the maximum forces that can be transmitted to the footing by plastic hinging of the column. The alternate design forces are the elastic design forces.
It should be noted that these may be considerably greater than the recommended design forces although where architectural considerations govern the design of a column the alternate elastic design forces may be less than the forces resulting from column plastic hinging.
C7.3 DESIGN DISPLACEMENTS FOR SEISMIC PERFORMANCE CATEGORIES C AND D For bridges classified as SPC C and D, the design displacements are specified as either the maximum of those calculated by the elastic analysis of Article 3.8 or the minimum specified support lengths given by Equation 7-3. This eitherlor specification was introduced to account for larger displacements that may occur from the analysis of more flexible bridges. It was the opinion of the PEP that displacements obtained from the elastic analysis of bridges should provide a reasonable estimate of the displacements resulting from the inelastic response of the bridge. However, it must be recognized that displacements are very sensitive to the flexibility of the foundation and if the foundation is not included in the elastic analysis of Article 3.8 consideration should be given to increasing the specified displacements for bridges founded on very soft soils. This increase may be of the order of 50% or more but as with any generalization, considerable judgment is required.
A better method is to determine upper and lower bounds from an elastic analysis which incorporates foundation flexibility. Special care in regard to foundation flexibility is required for bridges with high piers. The rationale for these minimum seat width requirements is given in Article C3.10.
Skewed bridges are known to develop large displacements at their supports due to the superstructure rotating about a vertical axis through the center of stiffness of the substructure. This has been observed in many earthquakes and the relatively frequent occurrence of damaged and unseated skewed spans is attributed to this phenomenon. For this reason, the seat width for skewed spans is increased in proportion to the square of the angle of skew. This allowance for skew increases the minimum width (N in Equation 7-3) by approximately 25% for bridges with skew angles of 45".
Note that N is measured normal to the leading edge of the seat and thus the minimum width parallel to the bridge centerline is larger than N and given by N times the secant of the angle of skew. C7.4 FOUNDATIONS AND ABUTMENT DESIGN REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORIES C AND D Since the design of bridge foundations in SPC C and D is essentially the same as for bridges in SPC B, a joint commentary is provided in Appendix A. C7.5 STRUCTURAL STEEL DESIGN REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORIES C AND D C7.5.1 General The 50% increase in allowable stresses for service load design is based on the following: 1. The margin of safety between the yield strength and allowable stress of short columns. 2. The margin of safety between the yield strength and allowable tensile stress. 3.
The margin of safety of compression members, which varies between 1.7 and 1.9.697 It is noted that seismic design requirements for steel bridges are currently under review and it is likely that the provisions given in these articles will be modified in the near future. Some indication of the likely trends in seismic design of steel bridges may be found in Report ATC32 prepared by the Applied Technology Council for Caltranss and in a report prepared for the American Iron and Steel Institute Task Force on Seismic Design by A. Astaneh? C7.5.2 P-delta Effects This Article provides modifications to the interaction equations when the P-delta effects are explicitly determined.
In columns, the reductions to the allowable stresses are in part a result of the consideration of member P-delta effects. These P-delta reductions are modified in AASHTO by a K-factor which is a recognition of the effect of end restraint in the member P-delta relationship.
The bases for the values of this ratio where joint translation is prevented are well documented. The selection of the value of C, where joint translation is permitted was an approximation applicable primarily to designs for which C-68 HIGHWAY BRIDGES significant applied horizontal forces are not present. Since the advent of computer analysis, the solution of the interaction equations when secondary effects resulting from deflection are taken into account, has become much easier.
In most cases, with significant horizontal displacements, the first iteration of deflection is sufficient. It is possible that for some members, such as weak axis columns depending on end support conditions, critical stress may occur at the midheight rather than the column ends. Thus the stress limits specified when joint translation is prevented should not be exceeded. C7.6 REINFORCED CONCRETE DESIGN REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORIES C AND D C7.6.1 General The purpose of the additional design requirements of this Article is to ensure that, for bridges classified as SPC C and D, the design of the components of these bridges is consistent with the overall design philosophy and that the potential for failures observed in past earthquakes is minimized. The additional column design requirements of this Article for bridges classified as SPC C and D are such that a column is forced to yield in flexure with a reasonable ductility capacity and that the potential for a shear, compression or loss of anchorage mode of failure is minimized. The additional design requirements for piers provide for some inelastic capacity; however, the R-factor specified for piers is such that the anticipated inelastic capacity is significantly less than that of columns. The actual ductility demand on a column or pier is a complex function of a number of variableslO.llincluding the earthquake characteristics, design force level, period of the bridge, shape of the inelastic hysteresis loop of the contributions of founcolumns, elastic damping coeff~cient, dation and bearing compliance to structural flexibility, and plastic hinge length of the column. The damage potential of a column is also related to the ratio of the duration of strong motion shaking to the natural period of the bridge. This ratio will be an indicator of the number of yield excursions, and hence of the cumulative ductility. There are some grounds for considering the cumulative ductility to be a more useful index than the peak ductility level; for example, 10 cycles at a curvature ductility factor of 8 might be more damaging than one yield excursion at a curvature ductility factor of 10 or 12. However, there is little experimental evidence to support or contradict this view.
Both Service Load and Load Factor methods of design are permitted although it is recommended that the Load Factor method of design be used since it is consistent with the ultimate load capacity concept used in determining the design force levels. An increase in allowable stresses of 33%% is permitted for Service Load design. This is consistent with current Division I specifications. C7.6.2 Column Requirements The definition of a column in this section is provided as a guideline to differentiate between the additional design requirements for a pier and for a column.
This should be used as a guideline and, if a column or pier is above or below the recommended criterion, it can be considered either as a column or pier, provided the appropriate R-Factor of Article 3.7 and the appropriate requirements of either Article 7.6.2 or 7.6.3 are used. For columns with an Wd ratio less than 21/2, the forces resulting from plastic hinging will generally exceed the elastic design forces and consequently the forces of Article 7.2.2 would not be applicable. C7.6.2(A) Vertical Reinforcement This requirement is intended to apply for the full section of the columns. The lower limit on the column reinforcement reflects the traditional concern for the effect of time-dependent deformations as well as the desire to avoid a sizable difference between the flexural cracking and yield moments.
The 6% maximum ratio is to avoid congestion and to permit anchorage of the longitudinal steel. If the effectiveness of higher percentages of reinforcement is substantiated by test results, relaxation of this requirement could be considered; however, the PEP gave serious consideration to reducing the upper limit to 4% and recommends that a lower value be used when feasible. C7.6.2(B) Flexural Strength Article C3.9 indicates that bridges will be subjected to the simultaneous occurrence of ground motion in three orthogonal directions. Thus columns are required to be designed biaxially and checked for both the minimum and maximum axial forces. For columns with a maximum axial stress exceeding 0.20 f: the strength reduction factor, d,, is reduced to 0.50. A linear interpolation is to be used for the value of d, (0.90) for pure flexure and the value at 0.20 f:. This requirement was added because of the trend towards a reduction in ductility capacity as the axial load increases. Implicit in this requirement is the recommendation that design axial stresses be less than 0.20 ff. Columns with axial stresses greater than this value are not prohibited but are designed for higher fore levels (i.e., a lower factor) in lieu of the lower ductility capacity. 1998 COMMENTARY C7.6.2(C) Column Shear and Transverse Reinforcement The requirements of this section are to minimize the potential for a column shear failure.'OJ1 The design shear force is specified as either that capable of being developed by flexural yielding of the columns or the elastic design shear force. This requirement was added because of the potential for superstructurecollapse if a column fails in shear. It should be noted that a column may yield in either the longitudinal or transverse direction and that the shear force corresponding to the maximum shear developed in either direction (for non-circular columns) should be used for the determination of the transverse reinforcement. The concrete contribution to shear capacity is undependable within the plastic hinge zone, particularly at low axial load levels, because of full section cracking under load reversals. As a result, the concrete shear contribution must be decreased for axial load levels less than 0.10 f: Ag. In the absence of evidence to the contrary, this article specifies a linear reduction in v, from the value permitted in Division I (Article 8.16.6.2) at 0.10 f: compression, down to zero at zero com~ressiveload. C-69 The confinement requirements for rectangular sections of Equation (7-6) were developed from the requirements for spiral columns as follows. The confining force P provided by a spirally reinforced column shown in Figure C7.6.2A, is where: r = confining pressure s = spacing of the spiral reinforcement D = core diameter of the column A, = area of the spiral reinforcement fyh = yield strength of the spiral reinforcement. Therefore, The volumetric ratio p, of spiral reinforcement is: C7.6.2(0) Transverse Reinforcement for Conjinement at Plastic Hinges The main function of the transverse reinforcement specified in this section is to ensure that the axial load carried by the column after spalling of the concrete cover will at least equal the load carried before spalling and that buckling of the longitudinal reinforcement is prevented.lO.llThus the spacing of the confining reinforcement is also important. Equation (8-63) of Division I, Article 8.18.2 and Equation (7-4) of these Specificationsare based on the arbitrary concept that, under axial compressive loading, the maximum capacity of the helically reinforced column (spiral column) before loss of cover concrete is equal to that with the cover concrete destroyed and the helical reinforcement stressed to its useful limit. The toughness of the spiral column under axial loading is not directly relevant to its typical role in earthquake-resistant structures where toughness or ductility is likely to be related to performance of the column under large reversals of moment as well as axial load.
Nonetheless, without implicit quantitative relationships between performance criteria interpreted in terms of the quality of the confined concrete and the amount of spiral reinforcement, there has been no compelling reason to modify Equation (7-4) for earthquakeresistant construction other than by adding Equation (7-5) which provides a varying lower bound to the amount of transverse reinforcement and tends to govern for columns with large cross-sectional areas. Substituting Equation (C7-2) into Equation (C7-I), The confining force provided by a rectangular column shown in Figure C7.6.2B, is P = rah, = ~,f,, (C7-4) where a is the spacing of the hoop reinforcement, and h, is the core dimension of the column for the direction FIGURE C7.6.2A Confining Pressure Provided by a Spirally Reinforced Column C-70 HIGHWAY BRIDGES Loss of concrete cover in the plastic hinge zone, as a result of spalling, requires careful detailing of the confining steel. It is clearly inadequate simply to lap the spiral reinforcement. If the concrete cover is going to spall, the spiral will be able to unwind. Therefore, under these conditions full strength lap welds are required. Similarly, rectangular hoops must be adequately anchored by bending ends back into the core. Thus the requirement of at least a 135" bend with an extension of at least six tie bar diameters back into the core, or an equivalent welded anchorage, was specified. Asf~h A& A, f y h &frr FIGURE C7.6.2B Confining Pressure Provided by a Rectangular Reinforced Column under consideration; ZA,f,, is the force resistance capability of the hoop reinforcement crossing the section under consideration.Therefore, Thus, if the two columns provide equal confining pres,sure, from Equation (C7-3) and Equation (C7-5), then, by substituting Equation (7-4) into Equation (C7-6), The 0.225 coefficient for a rectangular column corresponds to the experimentally determined 0.45 coefficient of Equation (7-4) for a spiral column.
However, on the basis of a limited amount of experimental data, it was felt that a rectangular column was not as effective as a spiral column. So the coefficient for a rectangular column was increased from 0.225 to 0.30. Figures C7.6.2C and C7.6.2D will aid the designer in the use of Equation (7-6). It should be noted that &, the total area of hoop reinforcement, should be determined for both principal axes of a rectangular column and the maximum value should be used. Based on tests conducted during the last decade, it has been recommended to use only spirally reinforced columns. Hoops are used for the confinement of large size columns. For rectangular shaped columns, spirals or hoops are used in interlocking rings. Cross ties are important for preventing buckling of longitudinal bars, especially in plastic hinge zones. The number of cross ties gets to be prohibitive for large columns and is the reason why interlocking spirals or hoops are recommended. C7.6.2(F) Splices In construction it is desirable to lap longitudinal reinforcement with starter bars or dowels at the column base. This is undesirable for seismic performance on two counts; first, the splice occurs in a potential plastic hinge region where requirements for bond will be extremely severe.1° This appears to have been the main cause of failure of one of the bridges of the Golden-State-Foothills freeway interchange in the 1971 San Fernando earthquake.12Second, lapping the main reinforcement will tend to concentrate plastic deformation close to the base and reduce the effective plastic hinge length as a result of stiffening of the column over the lapping region. This may result in a very severe local curvature demand. Testing of this common construction detail is urgently required. C7.6.3 Pier Requirements The requirements of this section are based on limited data on the behavior of piers in the inelastic range. Consequently, the R-Factor of 2 for piers is based on the assumption of minimal inelastic behavior. It is required that the vertical reinforcement ratio be equal to or in excess of the horizontal reinforcement ratio in order to avoid the possibility of having inadequate web reinforcement in piers which are short in comparison to their height. Splices are staggered in an effort to avoid weak sections. The requirement for a minimum of two layers of reinforcement in walls carrying substantial design shears is based on the premise that two curtains of reinforcement will tend to "basket" the concrete and retain the integrity of the wall after cracking of the concrete. Also, under typical construction conditions, the likelihood of maintaining the location of a single layer of reinforcement near the middle of the pier is low.
C7.6.4 Column Connections The integrity of the column connection is important if the columns are to develop their flexural capacity. First, the longitudinal reinforcement must be capable of devel- 1998 COMMENTARY C-7 1 NQTE : AT SPLICES, CROSSTIES SHALL 8E SUPPOirrEO OR SECURED TO PREVENT DISPLACEMENT OURING CONCRETE PLACEMENT. -b a FOR A, CFIOSSWGx-x AN v 9 6" MAX. PER AASHTO WHERE ALTERNATE BARS ARE TlES. FIGURE C7.6.2C Tie Details in a Rectangular Column CROSSlIESENGAGE LONGIT. REINFORCEMENT HOOPS AND CROSSTIES CONTRIBUfETOA A S REOUIRED BY FOR%LAS (8 3 AND 84) 6" MAX. PER AASKTO WHERE ALTERNATE BARS ARE nED. FIGURE C7.6.2D Tie Details in a Square Column C-72 HIGHWAY BRIDGES oping its overstrength capacity of 1.25fY.Second, the transverse confining reinforcement of the column must be continued a sufficient distance into the joint to avoid a plane of weakness at the interface. For column connections in a column cap an evaluation of existing and new data on the strength of joints subjected to moment reversals has indicated that the strength of the joint is relatively insensitive to the amount of transverse reinforcement, provided there is a minimum amount, or unconfined and that a limiting shear stress of 12joints may be used for normal weight aggregate concrete. The allowable stress for joints made with lightweight aggregate concrete has been based on the observation that shear transfer in such concrete has been measured to be approximately 75% of that in normal-weight aggregate concrete. C7.6.5 Construction Joints in Piers and Columns This section requires that construction joints be designed and constructed to resist seismic design forces at the joint. Equation (7-9) is based on Equation (11-30) of ACI 318-71 but is restated to reflect dowel action and frictional resistance. REFERENCES 1. Priestley, M.J.N., Seible, F., Chai, Y.H., "Design Guidelines for Assessment Retrofit and Repair of Bridges for Seismic Performance," University of California, San Diego, 1992. 2. Priestley, M.J.N., Park, R., Potangaroa, R.T., "Ductility of Spirally Confined Concrete Columns," ASCE, J. Structural Div., January 1981. 3. Mander, J.B., Priestley, M.J.N., Park, R., "Theoretical Stress-Strain Model for Confined Concrete," ASCE, J. Structural Div., August 1988. 4. Mander, J.B., Priestley, M.J.N., Park, R., "Observed Stress-Strain Behavior of Confined Concrete," ASCE, J. Structural Div., August 1988. 5.
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BMCOL 76 described by Board and Matlock.18 The influence of group action on pile stiffness is a somewhat controversial subject. Solutions based on elastic theory can be misleading where yield near the pile head occurs. Experimental evidence tends to suggest that group action is not significant for pile spacings greater than 4d to 6d.
For batter pile systems, the computation of lateral pile stiffness is complicated by the stiffness of the piles in axial compression and tension. It is also important to recognize that bending deformations in batter pile groups may generate high reaction forces on the pile cap. It should be noted that while batter piles are economically attractive for resisting horizontal loads, such piles are very rigid in the lateral direction if arranged so that only axial loads are induced. Hence, large relative lateral displacements of the more flexible surrounding soil may occur during the "free-field" earthquake response of the site (particularly if large changes in soil stiffness occur over the pile length, and these relative displacements may in turn induce high pile bending moments. For this reason, more flexible vertical pile systems where lateral load is resisted by bending near the pile heads, are commended. However, such pile systems must be designed to be ductile, because large lateral displacements may be necessary to resist the lateral load. A compromise design using batter piles spaced some distance apart may provide a system which has the benefits of limited flexibility and the economy of axial load resistance to lateral load. Soil-Pile Interaction. The use of pile stiffness characteristics to determine earthquake-induced pile bending moments based on a pseudo-static approach, assumes that moments are induced only by lateral loads arising from inertial effects on the bridge structure. However, it must be remembered that the inertial loads are generated by interaction of the free-field earthquake ground motion with the piles, and that the free-field displacements themselves can influence bending moments. This is illustrated in an idealized manner in Figure CA8. The free-field earthquake displacement time histories provide input into the lateral resistance interface elements which in turn transfer mo- - 1998 COMMENTARY r (t) n F (t) C-79 INERT1 A1 MOMENTS AN0 SHEAR FORCES STRUCTURE RELATIVE D l S k I C E m E N T BENDING'MOMENTS DOMINATED BY LATERAL INTERACT ION LOADS OI SPLICEI T llME t FREE F t E L O OISPLICEMEWT I T TIYE t BENDING MOMENTS DOMINATED BY EARTHQUAKE D l SPLACEMENTS .. I FREE F I E L D D l SPLACEMENT TIME HISTORIES FIGURE CA8 Mechanism of Soil-Pile Interaction During Seismic Loading tion to the pile. Near the pile heads, bending moments will be dominated by the lateral interaction loads generated by inertial effects on the bridge structure. At greater depth (e.g., greater than 1Od) where soil stiffness progressively increases with respect to pile stiffness, the pile will be constrained to deform in a similar manner to that of the free field, and pile bending moments become a function of the curvatures induced by free-field displacements. To illustrate the nature of free-field displacements, reference is made to Figure CA9, which shows a 200-ft (60 m) deep cohesionless soil profile subjected to the El Centro Earthquake.
The free-field response was determined using a nonlinear one-dimensional response analysis.
From the displacement profiles shown at specific times, curvatures can be computed and pile bending moments calculated it if is assumed that the pile is constrained to displace in phase with the free-field response. Large curvatures could develop at interfaces between soft and rigid soils and, clearly, in such cases emphasis should be placed on using flexible ductile piles. Margasuggests that curvatures of up to 6 X in.-' (15 X mm-l) could be induced by strong earthquakes, but these should pose no problems to well-designed steel or prestressed concrete piles. Studies incorporating the complete soil-pile-structure interaction system as presented by Figure CA8, have been described by Penzien20for a bridge piling system in a deep soft clay. A similar but somewhat simpler soil-pile-structure interaction system (SPASM) to that used by Penzien, has been described by Matlock et al.2' The model used is, in effect, a dynamic version of the previously mentioned BMCOL program. C6.4.2(C)and C7.4.2(C) Special Pile Requirements The uncertainties of ground and bridge response characteristics lead to the desirability of providing tolerant pile foundation systems. Toughness under induced curvature and shears is required, and hence piles such as steel H-sections and concrete filled steel-cased piles are favored for highly seismic areas. Unreinforced concrete piles are brittle in nature, so nominal longitudinal reinforcing is specified to reduce this hazard. The reinforcingsteel should be extended into the footing to tie elements together and to assist in load transfer from the pile to the pile cap. Experience has shown that reinforced concrete piles tend to hinge or shatter immediately below the pile cap. Hence, tie spacing is reduced in this area so that the concrete is better confined.
Driven precast piles should be constructed with considerable spiral confining steel to ensure good shear strength and tolerance of yield curvatures should these be imparted by the soil or structural response. Clearly, it is desirable to ensure that piles do not fail below ground level, and that flexural yielding in the columns is forced to occur above ground level. The additional pile design requirements imposed on piles for bridges classified as SPC C and D for which earthquake loading is more severe, reflect a design philosophy aimed C-80 HIGHWAY BRIDGES FIGURE CA9 Typical Earthquake Displacement Profiles at minimizing below ground damage which is not easily inspected following a major earthquake. C6.4.3, C7.4.3, andC7.4.5 Abutments The numerous case histories of damage to, or failure of, bridges induced by abutment failure or displacement during earthquakes have clearly demonstrated the need for careful attention to abutment design and detailing in seismic areas. Damage is typically associated with fill settlement or slumping, displacements induced by high seismically induced lateral earth pressures, or the transfer of high longitudinal or transverse inertia forces from the bridge structure itself. Settlement of abutment backfill, severe abutment damage or bridge deck damage induced by the movement of abutments may cause loss of bridge access, and hence abutments must be considered as a vital link in the overall seismic design process for bridges. The nature of abutment movement or damage during past earthquakes has been well documented in the literature. Evanszz examined the abutments of 39 bridges within 30 miles (48.3 km)of the 1968 M7 Inangahua earthquake in New Zealand, of which 23 showed measurable movement and 15 were damaged. Movements of free standing abutments followed the general pattern of outward motion and rotation about the top after contact with and restraint by the superstructures. Fill settlements were observed to be 10 to 15% of the fill height. Damage effects on bridge abutments in the M7 Madang earthquake in New Guinea reported by Ellisonz3 were similar; abutment movements as much as 20 in.
(500 mm)were noted. Damage to abutments in the 1971 San Fernando earthquake is described by Fung et al." Numerous instances of abutment dis- placement and associated damage have been reported in publications on the Niigata and Alaskan earthquakes. However, these failures were primarily associated with liquefaction of foundation soils. Design features of abutments vary tremendously, and depend on the nature of the bridge site, foundation soils, bridge span length and load magnitudes. Abutment types include free-standing gravity walls, cantilever walls, tied back walls, and monolithic diaphragms. Foundation support may use spread footings, vertical piles or battered piles, while connection details to the superstructure may incorporate roller supports, elastomeric bearings or fixed bolted connections. Considering the number of potential design variables together with the complex nature of soilabutment-superstructure interaction during earthquakes, it is clear that the seismic design of abutments necessitates many simplifying assumptions. C6.4.3(A),C7.4.3(A),and C7.4.5 Free-Standing Abutments For free-standing abutmentssuch as gravity or cantilever walls, which are able to yield laterally during an earthquake (i.e., superstructuresupported by bearings which are able to slide freely) the well-establishedMononobe-Okabe pseudostatic approach outlined below, is widely used to compute earth pressures induced by earthquakes. For free-standing abutments in highly seismic areas, design of abutments to provide zero displacement under peak ground accelerations may be unrealistic, and design for an acceptable small lateral displacement may be preferable. A recently developed method for computing the magnitude of relative wall displacement during a C-81 1998 COMMENTARY given earthquake is outlined in this subsection. Based on this simplified approach, recommendations are made for the selection of a pseudo-static seismic coefficient and the corresponding displacement level for a given effective peak ground acceleration. E, = 1/2yH2(1- k,)K, where the seismic active cos2(@-0-p) K, = Mononobe-Okabe Analysis The method most frequently used for the calculation of the seismic soil forces acting on a bridge abutment is a static approach developed in the 1920's by MononobeZ5and Okabe.%The Mononobe-Okabe analysis is an extension of the Coulomb sliding-wedge theory taking into account horizontal and vertical inertia forces acting on the soil. The analysis is described in detail by Seed and WhitmanZ7and Richards and Elms.%The following assumptions are made: 1. The abutment is free to yield sufficiently to enable full soil strength or active pressure conditions to be mobilized. If the abutment is rigidly fixed and unable to move, the soil forces will be much higher than those predicted by the Mononobe-Okabeanalysis. 2.
The backfill is cohesionless, with a friction angle of 3. The backfill is unsaturated, so that liquefaction problems will not arise. +. Equilibrium considerations of the soil wedge behind the abutment (Figure CA10) then lead to a value, Em, of the active force exerted on the soil mass by the abutment (and vice versa), when the abutment is at the point of failure. EAEis given by the expression: ~JCOS~CO pcos(6+p+ S~ 0) (CA-4) and where: y = unit weight of soil H = height of soil face = angle of friction of soil 0 = arc tan (kh/l - k,) 6 = angle of friction between soil and abutment kh = horizontal acceleration coefficient k, = vertical acceleration coefficient i = backfill slope angle p = slope of soil face. + The equivalent expression for passive force if the abutment is being pushed into the backfill is: where: GRAV l TY WALL t coefficient K, is: - CANTI LEVER WALL FIGURE CAlO Active Wedge Force Diagram C-82 HIGHWAY BRIDGES As the seismic inertia angle 0 increases, the values of K, and KpEapproach each other and, for a vertical backfill, become equal when 0 = +. Despite the relative simplicity of the approach, the accuracy of Equation (CA-3) has been substantiated by model testsz7 and by back calculation from observed failures of flood channel walls.2gIn the latter case, however, the displacements were large; and this, as will be seen, can modify the effective values of khat which failure occurs. The value of H, the height at which the resultant of the soil pressure acts on the abutment, may be taken as HI3 for the static case with no earthquake effects involved. However, it becomes greater as earthquake effects increase.
This has been shown by tests and theoretically by Wood,3Owho found that the resultant of the dynamic pressure acted approximately at mid-height. Seed and Whitman have suggested that h could be obtained by assuming that the static component of the soil force (computed from Equation (CA-3) with (0 = k, = 0) acts at HI3 from the bottom of the abutment, while the additional dynamic effect should be taken to act at a height of 0.6H. For most purposes it is sufficient to assume h = Hl2, with a uniformly distributed pressure. Although the Mononobe-Okabe expression for active thrust is easily evaluated for any particular geometry and HORIZONTAL SEISMl C COEFFICIENT kh friction angle, the significance of the various parameters is not obvious.
Figure CAll shows the variation of K, against khfor different values of 0 and k,;K, is obviously very sensitive to the value of 0. Also, for a constant value of 0, K, doubles as khincreases from zero to 0.35 for zero vertical acceleration, and thereafter it increases more rapidly. In order to evaluate the increase in soil active pressure due to earthquake effects more easily, , K can be normalized by dividing by its static value KAto give a thrust factor: Whereas Figure CAll shows that K, is sensitive to changes in the soil friction angle the plots of FT against in Figure CAI2 indicate that the value of has little effect on the thrust factor until quite suddenly, over a short range of FTincreases rapidly and becomes infinite for specific critical values of The reason for this behavior may be determined by examining Equation (CA-4). The contents of the radical must be positive for a real solution to be possible, and for this it is necessary that: +, + ,+, + +. This condition could also be thought of as specifying a limit to the horizontal acceleration coefficient that could . HORIZONTAL S E I S M I C COEFFICIENT kh FIGURE CAll Effect of Seismic Coefficients and Soil Friction Angle on Seismic Active Pressure Coefficient 1998 COMMENTARY -20' 25' 30' 35' 40' 45 SOIL FRICTION ANGLE U SOIL FRICTION ANGLE U FIGURE CAI2 Influence of Soil Friction Angle on MagniEcation Ratio be sustained by any structure in a given soil. The limiting condition is that: For zero vertical acceleration and backfill angle and for a soil friction angle of 35", the limiting value of kh is 0.7. This is a figure of some interest in that it provides an absolute upper bound for the seismic acceleration that can be transmitted to any structure whatsoever built on soil with the given strength characteristics. Figure CAI3 shows the effect on FTof changes in the vertical acceleration coefficient k,. Positive values of k, have a significant effect for values of kh greater than 0.2. The effect is greater than 10% above and to the right of the dashed line. As is to be expected from Equation (CA-6), KAEand FT are also sensitive to variations in backfill slope, particularly for higher values of horizontal acceleration coefficient when the limit implied by Equation (CA-6) is approached. This effect is shown in Figure CA14. The effects of abutment inertia are not taken into account in the Mononobe-Okabe analysis. Many current procedures assume that the inertia forces due to the mass of the abutment itself may be neglected in considering seismic behavior and seismic design. This is not a conservative assumption, and for those abutments relying on their mass for stability it is also an unreasonable assumption, in that to neglect the mass is to neglect a major aspect of their behavior. The effects of wall inertia are discussed further by Richards and who show that wall inertia forces should not be neglected in the design of gravity retaining walls. Designfor Displacement If peak ground accelerations are used in the MononobeOkabe analysis method, the size of gravity retaining structures will often be excessively great. To provide a more economic structure, design for a small tolerable displacement rather than no displacement may be preferable. Tests have shown that a gravity retaining wall fails in an incremental manner in an earthquake. For any earthquake ground motion, the total relative displacement may be calculated using the sliding block method suggested by Ne~mark.~' The method assumes a displacement pattern VERTICAL SEISMIC COEFFICIENT k, FIGURE CAI3 Influence of Vertical Seismic Coefficient on Magnification Ratio HIGHWAY BRIDGES C-84 SLOPE ANGLE i FIGURE CAI4 Influence of Backfill Slope Angle on Magnification Ratio similar to that of a block resting on a plane rough horizontal surface subjected to an earthquake, with the block being free to move against frictional resistance in one direction only. Figure CAI5 shows how the relative displacement relates to the acceleration and velocity time histories of soil and wall. At a critical value of kh,the wall is assumed to begin sliding; relative motion will continue until wall and soil velocities are equal. Figures CA16 and CAI7 show the resultsz8 of a computation of wall displacement for kh = 0.1 for the El Centro 1940 N-S record. Newmark computed the maximum displacement response for four earthquake records, and plotted the results after scaling the earthquakes to a common maximum acceleration and velocity. Franklin and Chang32repeated the analysis for a large number of both natural and synthetic records and added their results to the sample plot. Upper bound envelopes for their results are shown in Figure CA18. AU records were scaled to a maximum acceleration coefficient of 0.5 and a maximum velocity V of 30 in./sec (760 mmlsec). The maximum resistance coefficient N is the maximum acceleration coefficient sustainable by a sliding block before it slides. In the case of a wall designed using the Mononobe-Okabe method, the maximum coefficient is, of course, kh. Figure CAI8 shows that the displacement envelopes for all the scaled records have roughly the same shape. An approximation to the curves for relatively low displacements is given by the relation, expressed in any consistent set of units, (CA-10) FIGURE CAI5 Relation Between Relative Displacement and Acceleration and Velocity Time Histories of Soil and Wall 1998 COMMENTARY TIME C-85 SECONOS -- -04 - 0 - SOIL .-.- - -..- WALL I 1 2 3 4 5 7 6 TIME 8 9 10 11 12 13 14 I I5 SECONDS FIGURE CAI6 Acceleration and Velocity Time Histories of Soil and Wall (El Centro 1940 N-S Record) where d is the total relative displacement of a wall subjected to an earthquake ground motion whose maximum acceleration coefficient and maximum velocity are A and V, respectively. This is drawn as a straight line on Figure CA18. Note that as this expression has been derived from envelope curves, it will overestimate d for most eaahquakes. One possible design procedure would be to choose a desired value of maximum wall displacement d together with appropriate earthquake parameters, and to use Equation (CA-10) to derive a value of the seismic acceleration coefficient for which the wall should be designed. The wall connections, if any, could then be detailed to allow for this displacement. T I M E . SECONOS FIGURE CAI7 Relative Displacement of Wall (El Centro 1940 N-S Record) C-86 HIGHWAY BRIDGES I VALUES OF T = MAX. RESISTANCE COEFFICIENT MAX. EARTHPUAKE ACCELERATION FIGURE CAI8 Upper Bound Envelope Curves of Permanent Displacements for All Natural and Synthetic Records Analyzed by Franklin and Chang (1 in. = 25.4 mm) By applying the above procedure to several simplified examples, Elms and Martin33have shown that a value of kh = A12 is adequate for most design purposes, provided that allowanceis made for an outward displacementof the abutment of up to 10Ain. (254A mm). For bridges classified as SPC C and D, more detailed consideration of the mechanism of transfer of structural inertia forces through bridge bearings to free-standing abutments is required, particularly for bridges classified as SPC D where continued bridge accessibility after a major earthquake is required. For sliding steel bearings or pot bearings, force diagrams describing limiting equilibrium conditions for a simple abutment are shown in Figure CA19. Where bearings comprise unconfined elastomeric pads, the nature of the forces transferred to the abutment becomes more complex, since such bearings are capable of transferring significant force. The magnitude of the force initially depends on the relative movement between the superstructure and the abutment, and force magnitudes can become quite large before slip will occur. For bridges classified as SPC D, additional consideration should be given to the use of linkage bolts and buffers to minimize damage. A typical abutment support detail used by the New Zealand Ministry of Works is shown in Figure CA20. It may be seen that linkage bolts are incorporated to prevent spans dropping off supports. The rubber rings act as buffers to prevent impact damage in the event that the lateral displacement clearance provided is inadequate. The knock-off backwall accommodates differential displacement between the abutment and superstructure, with minimum structural damage. A more typical design provision in United States practice is to seal the gap between superstructure and abutment with bitumen 1998 COMMENTARY C-87 FIGURE CAI9 Force Diagrams Including Bearing Friction to minimize impact damage. It must be recognized, however, that in this case some damage and possible abutment rotation will occur in strong earthquakes. The use of a settlement or approach slab in Figures CA20 and CA21 which has the effect of providing bridge access in the event of backfill settlement is also noted. The slab also provides an additional abutment friction anchorage against lateral movement.
Nonyielding Abutments As previously noted, the Mononobe-Okabe analysis assumes that the abutment is free to yield laterally a sufficient amount to mobilize peak soil strengths in the soil backfill. For granular soils, peak strengths can be assumed to be mobilized if deflections at the top of the wall are about 0.5% of the abutment height. For abutments which are restrained against lateral movement by tie backs or batter piles, lateral pressures induced by inertia forces in the backfill will be greater than those given by a Mononobe-Okabe analysis. Simplified elastic solutions FRI C T I O N O R / SETTLEMENT SLAB C6.4.3(B), C7.4.3(B)and C7.4.5 Monolithic Abutments Monolithic or end diaphragm abutments such as shown in Figure CA21 are commonly used for single and for twospan bridges in California. As shown, the end diaphragm is cast monolithically with the superstructure and may be directly supported on piles, or provision may be made for beam shortening during post-tensioning. The diaphragm acts as a retaining wall with the superstructure acting as a prop between abutments. Such abutments have performed well during earthquakes and avoid problems such as backwall and bearing damage associated with yielding abutments, and reduce oj EXPANSI ON, JOINT BACKWALL WITH ' KNOCK-OFF DEVl CE presented by Wood30for rigid non-yielding walls, also indicate that pressures are greater than those given by Mononobe-Okabe. The use of a factor of 1.5 in conjunction with peak ground accelerations is suggested for design where doubt exists that an abutment can yield sufficiently to mobilize soil strengths. /RUBBER RING BOLT I I \BEARING w FIGURE CA20 Seat-Type Abutment Showing Details Used in New Zealand C-88 HIGHWAY BRIDGES ! t. ABUTMENT . SETTLEMENT SLAB ' 1i I I NEOPRENE STRIP FIGURE CA21 Monolithic Abutments Showing Details Used in California the lateral load taken by columns or piers. On the other hand, higher longitudinal and transverse superstructure inertia forces are transmitted directly into the backfill and provision must be made for adequate passive resistance to avoid excessive relative displacements. Whereas free-standing or seat-type abutments allow the engineer more control over development of soil forces, the added joint introduces a potential collapse mechanism into the structure. To avoid this collapse mechanism, monolithic abutments are particularly recommended for bridges classified as SPC D. Whereas damage may be heavier than that for free-standing abutments because of the higher forces transferred to backfill soils, with adequate abutment reinforcement the collapse potential is low. In making estimates of monolithic abutment stiffness and associated longitudinal displacements during transfer of peak earthquake forces from the structure, it is recommended that abutments be proportioned to restrict displacements to 0.3 ft (90 mm) or less in order to minimize damage. REFERENCES 1. Ferritto, J.M. and Forest, J.B., "Determination of Seismically Induced Soil Liquefaction Potential at Proposed Bridge Sites," Federal Highway Administration Offices of Research and Development, Washington, DC, 1977. 2. Martin, Geoffrey R., "Seismic Design Considerations for Bridge Foundations and Site Liquefaction Potential," Proceedings, Workshop on Seismic Problems 3. 4. 5.
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Smooth blocks at low, confining stresses (i.e., blocks located near the top of the wall) will tend to have connection failures controlled by pullout. At higher confining stresses (i.e., blocks near the middle and bottom of the wall), connection failures will likely be controlled by rupture of the reinforcement at the connection. The connection failure mode is determined from the laboratory connection test and the anticipated confining pressure applied to the connection based on the facing block location within the wall. C5.8 MECHANICALLY STABILIZED EARTH WALL DESIGN C5.8.6 Reinforcement Strength Design C5.8.6.1 Design L i e Requirements C5.8.6.1.1 This definition of nonaggressive soil is currently provided in Division 11, Article 7.3.6.3. Since this definition really functions as a design criteria, the definition of nonaggressive soil should be located in Division I. This revision HIGHWAY BRIDGES moves the definition of nonaggressive soil to Division I.
The content of the criteria has not been changed. References in Division I to Division II for the definition of nonaggressive soil were removed since the definition of nonaggressive soil is moved to Division I through this revision.
This definition of nonaggressive soil is currently provided in Division 11, Article 7.3.6.3. Since this definition really functions as a design criteria, it should be located in Division I. This revision moves the definition to Division I.
The content of the criteria have not been changed, other than for temporary geosynthetic structures the maximum allowed pH was changed from "1 1" to "10". The reason for the more restrictive pH range for temporary geosynthetic structures is that for some geosynthetic polymers, in particular polyester, degradation can occur very rapidly at a pH of 11 based on laboratory studies.
This change will also make the AASHTO specifications more consistent with the FKWA Demonstration Project 82 manual. References in Division I to Division IIfor the definition of nonaggressive soil were removed since the definition of nonaggressive soil is moved to Division I through this revision. C5.8.7 Soil Reinforcement /Facing Connection Strength Design C5.8.7.2 Connection Strength for Geosynthetic Reinforcements This revision clarifies that one must calculate connection strength only for the connection failure mode which controls, i.e., reinforcement rupture or reinforcement pullout. Smooth blocks at low, confining stresses (i.e., blocks located near the top of the wall) will tend to have connection failures controlled by pullout. At higher confining stresses (i.e., blocks near the middle and bottom of the wall), connection failures will likely be controlled by rupture of the reinforcement at the connection. The comection failure mode is determined from the laboratory connection test and the anticipated confining pressure applied to the connection based on the facing block location within the wall. Changes to Article 5.8.7.2 of the 1997Znterim Specifications were balloted and approved for inclusion in the 1998 Interim Specijications. See also C5.8.7.2 (1998). C5.8.12 Special Loading Conditions use these specifications, and at times designers have selected the wrong distribution for design. This article was based on the results from a full-scale traffic barrier test conducted by the Reinforced Earth Company (see Reinforced Earth Company Technical Bulletin MSE-8, October 1995 for additional information.) This revision makes Article 5.8.12.2 consistent with the results of this full-scale crash barrier research and attempts to clarify the current specifications regarding this issue. See also C5.8.12.2 (1998). COMMENTARY TO SECTION 1OSTRUCTURAL STEEL (OMNIBUS REVISIONS) INTRODUCTION The miscellaneous revisions to AASHTO Section 10 have been prepared to: 1) allow the engineer the option to compute fatigue stress ranges (in both ASD and LFD) and overload flange stresses (in LFD) for composite sections assuming the concrete deck to be fully effective for both positive and negative moment if certain conditions are met, and 2) allow the engineer to compute the maximum strength of the compression flange for a braced noncompact section in LFD based on the actual compressionflange slenderness ratio, with a practical upper limit specified for the ratio. Other revisions (primarily editorial) have also been prepared, which are intended to clarify and enhance certain existing provisions. Reference to AASHTO M 270 Grade 70W and ASTM A 709 Grade 70W steel has been replaced with reference to AASHTO M 270 Grade HPS70W and ASTM A 709 Grade HPS70W steel to encourage the use of HPS over conventional 70W bridge steel due to its enhanced properties. Grade 70W steel is still available at this writing, but should only be used with the approval of the Owner. Table C10.2A Minimum Material PropertiesStructural Steel Reference to AASHTO M 270 Grade 70W and ASTM A 709 Grade 70W steel has been replaced with reference to AASHTO M 270 Grade HPS70W and ASTM A 709 Grade HPS70W steel to encourage the use of HPS over conventional 70W bridge steel due to its enhanced properties. Grade 70W steel is still available at this writing, but should only be used with the approval of the Owner. C10.3 REPETITIVE LOADING AND TOUGHNESS CONSIDERATIONS C5.8.12.2 lkaffic Loads and Barriers C10.3.1 Allowable Fatigue Stress The reason for the two different impact load distributions in Article 5.8.12.2 and how the load distributions are to be used for design is not provided in the current specifications. This has created confusion for designers who The revision to this article allows the engineer the option to compute the range of stress in ASD using the full composite section assuming the concrete deck to be fully effec- 199912000 COMMENTARY tive for both positive and negative moment if certain conditions are met. Those conditions are that shear connectors must be provided along the entire length of the girder and the longitudinal reinforcement must satisfy the revised provisions of Article 10.38.4.3. The revised Article 10.38.4.3 states that the minimum longitudinal reinforcement in the concrete deck must equal or exceed 1 percent of the crosssectional area of the concrete slab whenever the longitudinal tensile stress in the slab due to either the construction loads or the design loads exceeds the allowable tensile stress for the concrete f, specified in Article 8.15.2.1 .l.In addition, the required longitudinal reinforcement is to be No. 6 bars or smaller spaced at not more than 12 inches. Concrete can provide significant resistance to tensile stress at service load levels. Recognizing this behavior will have a significantly beneficial effect on the computation of fatigue stress ranges in top flanges in regions of stress reversal and in negative moment regions. By utilizing shear connectors in these regions to ensure composite action and properly placed longitudinal reinforcement at locations wherever the longitudinal tensile stress in the deck exceeds the tensile strength of the concrete, crack length and width can be controlled so that full-depth cracks should not occur. When a crack does occur, the stress in the longitudinal reinforcementincreases until the crack is arrested. Ultimately, the cracked concrete and the reinforcement reach equilibrium. Thus, the deck may contain a small number of staggered cracks at any given section. Properly placed longitudinal reinforcement prevents coalescence of these cracks. Reference 1 addresses the effects of slip and crack size on both the strength and stiffness of concrete in tension. Field data presented in Reference 2 substantiate that stresses in the composite section are best predicted based on section properties computed assuming an uncracked composite section up to the overload level. C10.3.2 Load Cycles quirements given in the latest version of the ASTM A 709 specification. C10.6 DEFLECTION This revision states that the live load used to check deflection preferably shall not exceed HS20 loading. This revision parallels a similar revision that was made to Article 10.3.2 (see the preceding discussion in C10.3.2.1). Live load deflection is a serviceability issue and not a strength issue. Experience with bridges designed according to previous versions of these specifications indicates no adverse effects attributed directly to live load deflection.Therefore, there would appear to be little reason to check live load deflection according to the suggested criteria for a loading heavier than the standard HS20 loading given in these specifications. C10.12 FLEXURAL MEMBERS The revision toArticle 10.12, "Flexural Members," defines an effective flange area& for tension flanges of flexural members. The effective flange area is given by Equation (10-4g) in Article 10.18.2.2.4. The effective flange area is to be used to compute the elastic section properties at sections with holes. At splices in areas of stress reversal, & should only be computed for the flange subject to tension under the loading condition being investigated. Also, for reasons to be discussed in Article 10.18.2.2.4, the strength of compact sections with holes in the tension flange is not to be taken greater than the moment capacity at first yield in the case of the strength design method. C10.15 HEAT-CURVED ROLLED BEAMS AND WELDED PLATE GIRDERS C10.15.1 Scope This revision states that the live load used to check fatigue preferably shall not exceed HS20 loading. Many states are now designing for HS25 live loading. However, the existing AASHTO fatigue provisions in the Standard Specifications were initially developed assuming an HS20 design loading. The use of HS25 loading for fatigue design can yield results that are inconsistent with the calibration used to develop the specifications and can unduly penalize the design. Heat curving of rolled beams and welded girders is extended to Grade HPS70W (high-performance)steels with a specified minimum yield strength not exceeding 70,000 psi. This revision is being made in Division I for consistency with the revision permitting heat curving of these steels that is being made to Article 11.4.12.2.1 of Division II under AASHTO 1999 Agenda Item 12. C10.3.3 Charpy V-Notch Impact Requirements C10.18 SPLICES This article is removed because the statement in this article is no longer valid based on the fracture toughness re- INTRODUCTION These revisions have been prepared to: 1) ensure a more consistent interpretation of the provisions for the design of splices in flexural members at all limit states, C-94 HIGHWAY BRIDGES 2) better handle the design of splices for composite flexural members, especially in areas of stress reversal, 3) provide a more consistent and reasonable design shear for splices in flexural members, and 4) clarify the intent and application of the 15% rule to determine the effective flange area for flexural members with holes. Organizational changes have also been made to separate and group provisions related specifically to the service load design and strength design methods. These revisions concentrateprimarily on the basic provisions for the design of bolted field splices for flexural members, which require the greatest amount of clarification. The revision does not directly address the effects of lateral flange bending on flange splices in curved I girders and the effects of torsional shear on bottom-flange splices of curved box girders at this time. Although not currently addressed in the Standard Specifications, designers should at least be cognizantof these effects and should consider their relative importance in splice designs for these members. pression members has been moved to Article 10.18.3, which covers the design of bolted splices for compression members. The previous language in this article regarding the 15% rule has been eliminated and is now covered in the following: 1) a new Article 10.18.2.2.4 (for application to the design of bolted flange splices), 2) a revised Article 10.18.4.1 (for application to the design of splices for tension members), and 3) a revised Article 10.12 (for application to the computation of elastic section properties for flexural members at sections with holes).
These articles

in shear and in bearing at the bolt holes according to Table 10.32.3B. Since an overload is not defined in the service load design method, high strength bolted connnections are also to be proportioned to prevent slip under D + (L + I). The slip resistance is determined as specified in Article 10.32.3.2.1. The second sentence in previous versions of Article 10.18.1.1 stated that "For members primarily in bending, the gross section shall also be used, except that if more than 15% of eachflange area is removed, that amount removed in excess of 15% shall be deductedffom the gross area." This so-called 15% rule is intended to ensure that a flange subject to tension will not fracture at its net section. The above rule is re-written as Equation (10-4g) in a new Article 10.18.2.2.4 for checking the strength of flange splices (i.e., the flange and associated splice plates) subject to tension. The equation defines an effective A, based on the above rule. If yielding on the effective area given by Equation (10-4g) is prevented in a flange or splice plate subject to tension, then fracture on the net section should theoretically not occur and need not be explicitly checked. For flanges and splice plates subject to compression, net section fracture is not a concern, and the effective area is simply equal to the gross area as stated in Equation (10-4h). The effective area given by Equation (10-4g) is equal to the net area of the flange or splice plate plus a factor P times the gross area of the flange or splice plate. The sum must not exceed the gross area. The factor P can be defined by the following equation: where A, is the net area, A, is the gross area, and F, is the specified minimum tensile strength of the steel. Based on the above formula, for plates with a specified minimum yield strength Fyof 70 ksi and below, and with a ratio of net area to gross area greater than or equal to approximately 0.5, P can be taken equal to 0.15. In most all practical cases, the ratio of net area to gross area will exceed 0.5 based on current AASHTO bolt spacing and edge distance requirements, which indicates that Equation (10-4g) in its current form will theoretically prevent fracture at the net section of a flange or splice plate for those cases. As a result, the existing check on fracture at the net section in this article (accomplished by limiting the design tensile stress on the net section to a specified percentage of F,), is considered to be redundant for most all practical cases and has therefore been eliminated. For the rare case where the ratio of the net area to the gross area of a flange or splice plate of 70-ksi steel or below might be below 0.5, the designer may wish to make an adjustment in P using the above formula, or else make an explicit check for fracture on the net section. For 100-ksi yield-strength steels, the effective area of the flange or splice plates is conservatively limited to the net area. Thus, the factor P is set equal to 0.0 for M 270 Grades 1001100W steels. P is also set equal to 0.0 when holes exceed 1-114 inch in diameter (see Table 10.32.1A in the Standard Specifications). For all other steels and when holes do not exceed 1-114 inch in diameter, P is equal to 0.15 (or 15%). The net area is given as the net width W,of the flange times the flange thickness in order to accommodate the possibility of staggered holes in the flange and splice plate, where several chains of holes may need to be investigated to determine the minimum net width (and area). C10.18.2.3 Web Splices In general, web splice plates and their connections are to be proportioned for a combination of the following: 1) a design shear, 2) a moment due to the eccentricity of the design shear, 3) the portion of the flexural moment as- sumed to be resisted by the web (applied at the mid-depth of the web), and 4) for sections where the neutral axis is not located at the mid-depth of the web, a horizontal design force resultant (applied at the mid-depth of the web). Each of these force effects is discussed in more detail later on under the description of subsequent articles. Web splice plates are to be symmetrical on each side of the web and are to extend as near as practical to the full depth of the web between flanges without impinging on bolt assembly clearances. The required bolt assembly clearances are available in the AISC Manual of Steel Construction. In this article, a minimum design shear V, is defined for the case of the strength design method. There have been many different interpretations of the current provisions for the design shear. Some designers have used the larger of 75% of the web shear capacity at the splice V,, or the average of the maximum shear V at the point of splice and V,. Others have used a notional design shear previously defined in Article 10.18.2.3 as V multiplied by the ratio of the splice design moment to the actual moment at the splice.

As discussed earlier, for a composite section in an area of stress reversal, the maximum bending strength of the section at the splice is different in positive and negative flexure; thus, it is not always clear which section should be used to compute the splice design moment. Also, the notional design shear previously given in Article 10.18.2.3 was originally intended to be used only for designing splices in rolled flexural members in order to provide a more reasonable value of the design shear. However, many have since applied this notional shear to the design of web splices for larger girder sections in an attempt to determine a more reasonable design shear for those sections. In general, V, can be as much as 4 to 5 times greater than V for members such as short stocky rolled beams. If V, is assumed to be equal to 5V and the traditional 75% and average rules are applied to determine the design shear for the splice, then 0.75(5V)>3.75V (governs) It would seem to be overly conservative and impractical to design the web splice in this case for 3.75 times the maximum applied shear. Thus, to provide a more consistent design shear to be used for designing web splices for all types of flexural members and to prevent having to design the web splice C-100 HIGHWAY BRIDGES for unreasonably large increases in the applied shear in certain cases (such as the case illustrated above), Equation (10-4i) of the revised provisions arbitrarily limits the increase in the shear to 50% of V when V is less than 50% of the shear capacity V,.

This would represent the region where the 75% rule would normally govern.

The increase in the shear is limited to 50% of V because the opportunities for V to change from its calculated value are smaller than for moment; large unintended shifts in the shear at the splice are unlikely. In addition, the maximum shear is usually not concurrent with the maximum moment at the splice; thus, the use of a lower value of the design shear in regions where the applied shear is low seems reasonable. For cases where Vis greater than 50% of V,, the average rule [Equation (10-4j)I is applied to determine the design shear. In this article, a minimum design moment M, due to the eccentricity of the design shear V, is defined for the case of the strength design method. The eccentricity of V, is explicitly defined as the distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration.

Some designers have defined the eccentricity as the distance between the centroids of the connection on each side of the joint, but recent tests on bolted splices conducted at the University of Texas have indicated that the eccentricity should instead be defined from the centerline of the splice (5). In this article, the portion of the flexural moment that is assumed to be resisted by the web M, is defined for the case of the strength design method. M, is assumed to be applied at the mid-depth of the web, which means that for sections where the neutral axis is not at the mid-depth of the web, a horizontal force resultant H, must also be applied at the mid-depth of the web in order to maintain equilibrium. M, and H, applied together yield a combined stress distribution equivalent to the unsymmetrical stress distribution in the web. For sections with equal compressive and tensile stresses at the top and bottom of the web (i.e. with the neutral axis located at the mid-depth of the web), H, will equal zero. The determination of the proportion of the total moment carried by the web is not necessarily straightforward for an unsymmetrical composite girder. Many different approaches have been used, which have not always led to consistent results. In addition, in designing the web-splice bolt group according to the traditional elastic vector method for the effects of this moment plus the moment due to the eccentric shear, many designers have computed the polar moment of inertia of the bolt group about the neutral axis of the composite section (which is typically not at the mid-depth of the web). Such an approach may not yield the correct result unless the neutral axis is computed from the summation of the stresses due to the appropriate loadings acting on the respective cross sections supporting the loadings. Simply shifting the polar moment of inertia of the bolts to the geometric neutral axis of the composite section may cause the bolt forces to be underestimated. Thus, to simplify the overall computations and to avoid possible errors, the provisions require that all actions (moment and horizontal force resultant) be applied at the mid-depth of the web. As a result, when applying the elastic vector method to determine the critical bolt forces in the web splice, the polar moment of inertia of the bolt group should be taken about the centroid of the connection. To further reduce any ambiguities, explicit equations are given in the provisions, which may be used to determine M, and H, to be applied at the mid-depth of the web. M, and H, are computed by conservatively using the elastic flexural stresses at the mid-thickness of the top and bottom flange. These stresses are computed considering the application of the moments due to the appropriate loadings to the respective cross sections supporting those loadings. By using the stresses at the mid-thickness of the flanges, the same stress values can be used in the design of both the flange and web splices, which simplifies the calculations. As required in Article 10.18.2.1.6, M, and H, are to be computed for both positive and negative flexure in areas of stress reversal. Each loading condition is to be considered independently to determine the governing condition. For the case of a composite girder in positive flexure, the controlling flange is typically the bottom flange; thus, the top of the web is usually in compression and the neutral axis is usually near the top flange. To compute minimum design values of M, and H, for this case, the stress at the mid-thickness of the bottom flange is assumed to be equal to its design stress Fcudefined by Equation (10-4b) times the hybrid girder reduction factor R. As shown in the following figure, the stress f,,, at the mid-thickness of the other flange (the top flange in this case), which is to be taken as the flexural stress concurrent with the maximum applied flexural stress fcuat the mid-thickness of the bottom flange, is then assumed to be factored up by the ratio Ru. For this loading condition, %, is taken as the ratio of Fcu to fcufor the bottom (controlling) flange. In essence, the stresses in the web are being factored up by the same amount as the stresses in the controlling flange so that the web splice is designed in a consistent fashion. By integrating these stresses over the depth of the web, Equation (10-41) can then be derived to com- 199912000 COMMENTARY C-101 Positive Flexure Case pute M, to be applied at the mid-depth of the web. H,,, given by Equation (10-4m), is simply taken as the average of the factored-up stresses at the mid-thickness of the top and bottom flange. The stresses in Equations (10-41) and (10-4m) are to be taken as signed quantities (positive for tension; negative for compression). For convenience, absolute value signs are applied to the resulting difference of the stresses in the equation for M., In actuality, the sign of M, corresponds to the sign of the vertical bending moment for the loading condition under consideration.

The computed value of H,is taken as a a signed quantity (positive for tension; negative for compression).

To incorporate the hybrid girder reduction factor R, the equation for M, was originally written as follows: Performing the algebra and rearranging yields Equation (10-41).

Equation (10-4m) can be derived similarly. For the case of negative flexure in an area of stress reversal, the controlling flange can be either the top or bottom flange, with the maximum stress caused by the sum of the dead-load plus the negative live-load moments; thus, the top of the web is usually in tension and the neutral axis is usually at or just slightly above the mid-depth of the web.

To compute minimum design values of M, and H, for this case, the stress at the mid-thickness of the controlling flange is again assumed to be equal to its design stress Fa defined by Equation (10-4b) times R. If the top flange is assumed to be the controlling flange, the stress fa, at the mid-thickness of the other flange (the bottom flange in this case), which is to be taken as the flexural stress concurrent with the flexural stress fcuat the mid-thickness of the top flange, is then assumed to be factored up by the ratio R,as shown in the following figure. For this case KUis taken as the ratio of Fcuto fcufor the top flange. M, is again given by Equation (10-41) and H, is again given by Equation (10-4m). For the case of web splices not in areas of stress reversal, M,, and H, need only be computed from Equations (10-41) and (10-4m) for the loading condition caus- Negative Flexure Curve C-102 HIGHWAY BRIDGES ing the maximum stress in the controlling flange at the point of splice. Thus, only a single loading condition need be checked. An alternative approach for compact steel sections, whereby all the flexural moment is assumed to be resisted by the flange splices, provided the flanges are capable of resisting the design moment, is referred to by footnote in Article 10.18.2.3.1. This method is only to be applied when checking the strength of the connection; slip of the bolts should still be checked using the conventional approach. Should the flanges not be capable of resisting the full design moment, the web splice is assumed to resist the additional flexural moment in addition to the design shear and the moment due to the eccentricity of the design shear. This article specifies that web splice plates and their connections in the case of the strength design method are to be proportioned as a minimum to develop the most critical combination of V, M,, M, and H,. The connections are to be designed as eccentrically loaded connections to develop the resultant bolt force in shear and in bearing at the bolt holes according to the provisions of Article 10.56.1.3.2. The traditional elastic vector method is the most common approach used to design the connection and is the recommended approach. M,, M,, and H,, are again to be applied at the mid-depth of the web and the polar moment of inertia of the connection should be computed about the centroid of the connection. The following formula can be used to compute the polar moment of inertia I, of the bolt group about the centroid of the connection: where: m= number of vertical rows of bolts n = number of bolts in one vertical row s = the vertical pitch g = the horizontal pitch Hwucan be assumed distributed equally to all the bolts and is simply added to the horizontal components of M, and Mwu. When checking the bearing strength at bolt holes in the web splice, the strength of an outermost hole can be conservatively checked against the maximum force (vector resultant) acting on the extreme bolt in the connection; this check is conservative since the components of this force parallel to the failure surfaces are smaller than the maximum force. Should the bearing strength be exceeded, it is recommended that the edge distance be increased slightly in lieu of increasing the number of bolts or thickening the web. Another option would be to calculate the bearing strength based on the inclined distance, or else resolve the resultant force in the direction parallel to the edge distance. The provisions also require that as a minimum, in the case of the strength design method, high-strength bolted connections for web splices be proportioned as eccentrically loaded connectionsto prevent slip under the most critical combination of: 1) an overload design shear V,, 2) an overload design moment Mvodue to the eccentricity of Vvo, 3) an overload design moment Mwoapplied at the mid-depth of the web representing the portion of the flexural moment that is assumed to be resisted by the web, and 4) for sections where the neutral axis is not located at the mid-depth of the web, an overload horizontal design force resultant Hwoapplied at the mid-depth of the web. The maximum resultant bolt force on the eccentrically load c o ~ e c t i o nshould not exceed the slip resistance of the connection computed from Equation (10-172) with the number of bolts Nbtaken equal to 1.0. Again, a Class B surface condition should be assumed for the faying surface wherever possible. The overload design shear M, given by Equation (10-4n) is simply taken as the maximum shear in the web due to D+PL(L+I)at the point of the splice, where PL is defined in Article 3.22.

The overload design moment Mvogiven by Equation (10-40) is taken as as the overload design shear V,, times the eccentricity e defined previously. The overload design moment M,, and horizontal force resultant H,, are computed using an approach similar to that described above for computing M, and H,. For splices in areas of stress reversal, both positive and negative flexure must again be considered. First, the loading condition causing the maximum flexural stress fo at the mid-thickness of the bottom flange for the smaller section at the splice due to D+PL(L+I)is considered. Then, the loading condition causing fo in the top flange is considered (it is not necessary to determine a controlling and non-controlling flange when checking slip). Each loading condition is considered independently to determine the governing condition. Equations (10-4p) and (10-4q) for computing Mwoand Hwoare similar in format to Equations (10-41) and (10-4m) for computing M, and H, with the following substitutions: 1) F,, is replaced by the maximum flexural stress f, due to D+PL(L+I)at the mid-thickness of the flange under consideration at the point of splice, 2) f,,, is replaced by fof,which is the flexural stress at the mid-thickness of the other flange due to D+PL(L+I)concurrent with foin the flange under consideration, and 3) R, and R are not required. Stresses at the mid-thickness of the flanges are again used in the equations in order to simplify the calculations.

1999/2000 COMMENTARY For the case of the web splices not in areas of stress reversal, M,, and H,, need only be computed from Equations (10-4p) and (10-4q) for the loading condition causing f, due to D+PL(L+I)in the flange with the maximum stress at the point of splice. C-103 method, high-strength bolted connections are also to be proportioned as eccentrically loaded connections to prevent slip under the most critical combination of shear, moment and horizontal force due to D+ (L+I) at the point of splice. The slip resistance is determined as specified in Article 10.32.3.2.1, with Nb again taken equal to 1.O. C10.18.2.3.6 through C10.18.2.3.9 For proportioning web splices and their connections in the case of the service load design method, these articles specify: 1) a design shear stress Fw (Article 10.18.2.3.6), 2) a design moment M, due to the eccentricity of the design shear (Article 10.18.2.3.7), 3) a design moment M, applied at mid-depth of the web representing the portion of the flexural moment that is assumed to be resisted by the web (Article 10.1 8.2.3.8), and 4) for sections where the neutral axis is not located at the mid-depth of the web, a horizontal design force resultant H, applied at middepth of the web (Article 10.18.2.3.8). The derivations of these design force effects are similar to the derivations for these force effects discussed previously for the case of the strength design method and will not be repeated here. Article 10.18.2.3.9 specifies that web splice plates and their connections in the case of service load design method are to be proportioned as a minimum to develop the most critical combination of F,Df, M,, M,, and H,. As in the case of the strength design method, the connections are to be designed as eccentrically loaded connections to develop the resultant bolt force in shear and in bearing at the bolt holes according to Table 10.32.3B. M,, M,, and Hw are again to be applied at the mid-depth of the web. Since an overload is not defined in the service load design C10.18.3 Compression Members The title of this article has been changed from Columns to Compression Members so as to better indicate that the provisions apply to splices in all types of compression members. The provisions apply to splices made with highstrength bolted connections. C10.18.6 Fillers The language in this article is now encompassed in a new revised Article 10.18.1.2. Therefore, this article has been eliminated. Note: Following is a brief example illustrating the basic application of some of the proposed provisions for the design of flange and web splices for flexural members. The example is incomplete, but it does illustrate some of the basic computations involved in computing the design forces, moments, and shears that would be used to design the splice plates and their connections according to the proposed provisions. More complete illustrative examples should be available from the industry in the near future.

HIGHWAY BRIDGES C-104 EXAMPLE For the smaller section at the splice: A. Dead Note: Consider provisions of Article 10.12 when Bottom Flange computing section properhrhegStresses load positive live load: areatthemid-thickness of the flanges.

= 22.75 ksi + Controlling flange for positive flexure B. Dead load + negative live load: = -15.24 ksi Controlling flange for negative flexure TOPFlange Note: Future wearing surface conservative& ignored in Case B. Dead load load factor also A. Dead load + positive live load: = -6.75 ksi B. Dead load + negative live load: = 12.46 ksi 1.0. cOmeNative&takenm 1999/2000COMMENTARY C-105 Example: Flange Splices (cont'd) CASE 1: For positive flexure: = 36.38 ksi (Bottom Flange) or: -.- Fcu= 0.75aF9 = 0.75(1.0)50 = 37.50 ksi (governs) -~ FnCu= Rc,(lfncJRl) r 0.75aFfl (Top Flange) is taken as 1.0 for flanges in compression at sections where Mu 2 My. a Fncu= 1.65(1-6.7511.01) = 11 .

I 4 ksi r 0.75(1.0)50 = 37.50 ksi .. Fncu= 37.50 ksi HIGHWAY BRIDGES Example: Flange Splices fcont'd) CASE 1: For positive flexure: PC, = Fcu& (Bottom Flange) For flanges in tension: where: p = 0.15 Assume A, = $ = 22.5 in2 (i.e. holes < 15% of 4) .. PC. = (37.50)(22.5) = 844 kips (Top Flange) For flanges in compression, A, = 4. Assume A, = 14.0 inP .' Pn,, = (37.50)(14.0) = 525 kips 199912000 COMMENTARY C-107 Example: Flange Splices (cont'dl CASE 2: For negative flexure: a may be taken as (MJM,) for flanges in compression at sections where Mu is less than My. For simplicity, assume a = 1.0 for I-girders.

FCu= l12(lfculRI+ aF#) ((Bottom Flange) = 112((-15.2411 .O) + 1.0(50)) = 32.62 ksi or: Fcu = 0.75aFfl = 0.75(1.0)50 = 37.50 ksi (governs) --- -- ---- --- .- FncU = Rcu(lfncJRI) 2 0.75aFfl (Top Flange) Fncu= 2.36(112.46/1.01) = 29.41 ksi 2 0.75(1.0)50 = 37.50 ksi ... Fncu = 37.50 ksi HIGHWAY BRIDGES C-108 Example: Flange Splices (cont'dl CASE 2: For negative flexure: PC, = FC,& (Bottom Flange) For flanges in compression, A, = 4. .. PC, = (37.50)(22.5)= 844 kips Pncu = FncuAB (Top Flange) For flanges in tension: & = w " t + p A , s A, where: p = 0.15 Assume A, = 0.95A, = 13.3 in2 (i.e. holes a 20% of 4) -'- P,,, = (37.50)(13.3) = 499 kips 199912000 COMMENTARY Example: Flange Splices (cont'd) Bottom-Flange Splice: Assume inner & outer splice plate areas will be within 10 percent of each other. :. use PcJ2 = 84412 = 422 kips to design inner & outer plates (Art. 10.1 8.1.3) Note: if actual areas ~f inner & outer splice plates are not within 10 percent of each other, proportion PC. to the inner & outer plates by the ratio of the splice plate areas and repeat (Article 10.1 8.1.3). Check shear on bolts (double shear) and bearing at bolt holes for 844 kips. In lieu of extending the filler, reduce the shear strength of the bolts by [(l+y)l(l+2y)] on the side of the splice with the filler plate (Article 10.18.1.2.1). C-109 C-110 HIGHWAY BRIDGES Example: Flange Splices (cont'dl Top-Flange Splice: Assume inner & outer splice plate areas will be within 10 percent of each other. :. use Pncu/2= 52512 = 263 kips (A&,. = A, = 263150 = 5.26 in.2(comp.) use PncJ2 = 49912 = 250 kips & (,) = 250150 = 5.00 in.2 (tension) Note: if actual areas of inner & outer splice plates are not within 10 percent of each other, proportion P,, to the inner 8 outer plates by the ratio of the splice plate areas and repeat (Art. 10.18.1.3). Check shear on bolts (double shear) and bearing at bolt holes for 525 kips. In lieu of extending the filler, reduce the shear strength of the bolts by [(l+y)l(l+2y)] on the side of the splice with the filler plate (Article 10.1 8.1.2.1). 199912000 COMMENTARY SPLICE-DESIGN REQUIREMENTS Flanqe Splices (cont'd) Overload Desiqn Force (LFD onlv): where: f, = maximum flexural stress due to D + DL (L + I) at the mid-thickness of the flange at the point of splice. A, = smaller gross flange area on either side of the splice Used only to check for slip of the bolts. Recommendation: Use Class B surface condition if possible. C-111 HIGHWAY BRTDGES C-112 Example: Web Note: Future wearing surface conservatively ignored Dead load load factor Splice akocome,,,a~e~ taken as 1.0. Desian Shear: V = 1.3[1.0(-72-21) + 1.67(-98)] = -334 kips (governs) V = 1 .O(-72-9) + 1.3(1.67)(18) = -42 kips O.SV, = 0.5(464) = 232 kips V = 1-3341 kips = 334 kips > 232 kips :. v,, = 1/2(V + V,) V,, = 1/2((-334(+ 464) = 399 kips 1999/2000 COMMENTARY SPLICE-DESIGN REQUIREMENTS Moment Due to Eccentricitv of Vw; where: e = distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration Example - Web Splice (cont'd) C-113 HIGHWAY BRIDGES C-114 - E x a m ~ l e Web S ~ l i c e(cont'd) Since splice is in an area of stress reversal: CASE 1: For positive flexure: -11.14 ksi fCu(bot. flange) = 22.75 ksi fnCu(topflange) = -6.75 ksi F, (bot. flange) = 37.50 ksi Rcu = IFCU~fCUl = (37.50122.751 = 1.65 Rcufnw= (1.65)(-6.75) = -11.I4 ksi Mwu=: 0.5(69m0)~)(1.0)37~50 - (-1 1 .14)(12 =: 9,649 kip-in. = 804.1 kip-ft 37.50 ksi Hwu=: 0.5(69.0)((1 m0)37.50 + (-1 1.14)) 2 : : 454.7 kips 1999/2000COMMENTARY C-115 Example - Web Splice (cont'd) CASE 2: For negative flexure: fcu(bot. flange) = -15.24 ksi fnCdtopflange) = 12.46 ksi Fcu(bot. flange) = 37.50 ksi - . 30.65 ksi Rcu = IFC"~fCUl = (37.501-15.241 = 2.46 Rcufncu = (2.46)(12.46) = 30.65 ksi ~,~=0.5(69.0)~~(1.0)(-37.50) - 30.651 12 = 13,519 kip-in. -37.50 ksi = 1,127 kip-ft H,,=O.5(69.0)((1 .0)(-37.50) + 30.65) 2 = -118.2 kips HIGHWAY BRIDGES C-116 SPLICE-DESIGN REQUIREMENTS Use V,, Mvu,Mwuand Hwuto check: 1) shear resistance of the bolts 2) bearing resistance at bolt holes Ps= VwuINb(N, = no. of bolts) pH= Hwu/Nb where: m = number of vertical rows of bolts n = number of bolts in one vertical row s = the vertical pitch g = the horizontal pitch 1999/2000COMMENTARY SPLICE-DESIGN REQUIREMENTS Use Mvu,Mwuand Hwuto check: I) flexural yielding on the gross section of the web splice plates Use Vwuto check: 1) shear yielding on the gross section of the web splice plates C-117 HIGHWAY BRIDGES C-118 SPLICE-DESIGN REQUIREMENTS Check slip resistance of the web bolts in a similar fashion by: 1) Replacing V, with Vwo. 2) Replacing Mvuwith M,, = Vwoe. 3) Replacing Mwuwith Mwoand Hwuwith Hwo computed from the previous formulas only using the actual stresses at the midthickness of the flanges due to D + PL(L+I) in place of the design stresses. 1999l2000 COMMENTARY C10.21 LATERAL BRACING The word preferably has ben added to the fist sentence because there may be special instances where it may be desirable to include lateral bracing in the interior bays as well.

C10.23 WELDING C10.23.2 Effective Size of Fillet Welds C10.23.2.2 Minimum She of Fillet Welds In the table of minimum fillet weld sizes in this article, the metric thickness of the base metal of the thicker part joined (T) is changed from 19 mm to 20 mm.This revision brings the table into conformance with the requirements of the AWS D1.5 Bridge Welding Code and also with a similar table in the SI Units version of the 2ndEdition of the AASHTO LRFD Bridge Design Specijications (Table 6.13.3.4-1). C10.30.8 Stay-in-PlaceDeck Forms C10.30.8.2 Metal Stay-in-Place Forms Editorial revisions are made to this article.

A clarification is made to indicate that the deflection limit of Ll180 or % inch applies to form work spans of 10 feet or less and the deflection limit of U240 or 3/4 inch applies to form work spans exceeding 10 feet. C10.32 ALLOWABLE STRESSES C-119 buckling modes are given in the AISC Manual of Steel Construction, Ninth Edition, 1989. Reference to Grade HPS70W steel has been added.

Clarifications have also been made to the allowable stresses for axial tension in members with and without holes. For members without holes, yielding on the gross section is checked (for Grade 1001100Wsteels, gross section yielding is conservatively checked against 0.46Fu, which is less than 0.55Fy).For members with holes, both yielding on the gross section and fracture on the net section must be checked. Fracture on the net section is conservatively checked using 0.46Fu(which represents 0.55 times the ratio of the AISC resistance factor of 0.75 for net section fracture divided by the AISC resistance factor of 0.90 for gross section yielding). The net section check based on 0.50f, is eliminated. Because yielding on the gross section and fracture on the net section are to be explicitly checked, the former footnote d referring to the use of the 15% rule for the gross section check is redundant and is eliminated here. Also, the reference to open holes larger than 0/4 inches is removed because fracture is to now to be checked on the net section in all cases for members with holes, regardless of the hole size. Table C1032.3A Allowable Stresses for Low-Carbon Steel Bolts and Power Driven Rivets Footnote d has been added to indicate that the joint length correction factor also applies when determining the shear strength of ASTM A 307 bolts (Note: footnotes in all tables in Section 10 have been generally re-ordered in order to place them in a more logical sequence). Table C10.32.3B Allowable Stresses on High-Strength Bolts or Connected Material C10.32.1 Steel 'Ihble C1032.1A Allowable Stresses--Structural Steel (Inpounds per square inch) Language is added to footnote c of Table 10.32.1A regarding the computation of the allowable stress in ASD for concentrically loaded columns. The language indicates that singly symmetric and unsymmetric compression members, such as angles or tees, and doubly symmetric compression members, such as cruciform or builtup members with very thin walls, may be governed by the modes of flexural-torsional buckling or torsional buckling rather than the conventional flexural buckling mode reflected in the equations given in the table. It is further indicated that procedures to check these members for these Language has been added at the end of footnote e in order to clarify the definition of the 50-inch length used in determining whether or not to apply the joint-length correction factor when calculating the shear strength of highstrength bolts in flange splices. C10.32.3.3 Applied Tension, combined Tension, and Shear Equation (10-18) was replaced by Equations (10-16) and (10-17) in previous interim specifications and is no longer required. A note has been added to indicate the removal of this equation to prevent having to renumber d subsequent equations in section 10. C-120 HIGHWAY BRIDGES C10.32.4 Pins, Rollers, and Expansion Rockers An editorial correction is indicated in this article. The reference to Table 10.32.4.2A should be to Table 10.32.4.3A instead. There is no Table 10.32.4.2A in the specification.

C10.34 PLATE GIRDERS C10.34.2 Flanges C10.34.2.1 Welded Girders The indicated revision to this ASD article specifies recommended minimum flange proportions for fabricated I-shaped girders. Compression-flange widths are preferably not to be less than 0.2 times the web depth, but in no case less than 0.15 times the web depth. Compressionflange thicknesses are preferably not to be less than 1.5 times the web thickness. If the compression flange of the girder is smaller than the tension flange, the minimum flange width may be based on two times the depth of the web in compression, D,, rather than the web depth. These proportions are recommended to help ensure that the web is adequately restrained by the flanges to control web bend-buckling. The recommended proportions are based on a study by Zureick and Shih (Reference 6) on doubly symmetric tangent girders, which clearly showed that the web buckling capacity was dramatically reduced when the compression flange buckled prior to the web. Although the study was limited to doubly symmetric girders, the recommended minimum flange proportions are deemed to be adequate for reasonably proportioned singly symmetric I-girders. The advent of composite design has led to a significantreduction in the size of compression flanges in positive moment regions. These smaller flanges are most likely to be governed by the recommended limits.

Providing minimum compression-flange widths that satisfy the recommended limit in these regions will help to ensure a more stable girder that is easier to handle. In addition, the blt of tension flanges be limited to a practical upper limit of 24 to ensure the flanges will not distort excessively when welded to the web. Also, an upper limit on the blt for a tension flange covers the case where the flange may be subject to an unanticipated stress reversal. The AASHTO ASD compression-flangelocal buckling check specified in this article for the top flange during construction implicitly assumes that a load factor of approximately 1.82 (110.55) is applied to the unfactored dead loads. The corresponding LFD compression-flange local buckling check (Article 10.61.4) is made using a load factor of 1.3 applied to the unfactoreddead loads. Thus, the current ASD constructibility check applies 1.4 (1.8211.3) times more dead load. When the original ASD code was developed, the constructibility check for dead load alone was not explicitly considered. However, recognition of this significant discrepancy in safety margin for the case of dead load acting alone was apparently made at some point in time since the revised equation did appear in earlier versions of the Standard Specifications. Therefore, to once again reduce this significant inherent conservatism in the ASD constructibility check and make it more equivalent to the LFD check, the current ASD width-to-thickness requirement for the case of dead load acting alone is divided by resulting in the revised Equation (10-20). C10.34.2.2 Riveted or Bolted Girders The width-to-thickness requirement for unsupported outstanding legs of top flange angles in compression in composite girders under the noncomposite dead load [Equation (10-22)] is revised to be consistent with the revision made to Equation (10-20) of Article 10.34.2.1.5, as described below.

C10.34.3 Thickness of Web Plates C10.34.3.2 Girders Stiffened Longitudinally A longitudinally stiffened web must be investigated for the stress conditions at different limit states, as well as along the girder. The stiffener is often located at an inefficient location for a particular condition resulting in a very low bend-buckling web capacity (reflected in a small value of the bend-buckling coefficient k). Because simply-supported boundary conditions are assumed in the development of the equations for k, it is conceivable that the computed web bend-buckling capacity for the longitudinally stiffened web may be less than that computed for a web without longitudinal stiffeners where some rotational restraint from the flanges has been assumed. To prevent this anomaly, this revision requires that the k value for a longitudinally stiffened web for the case where dSDc 2 0.4 equal or exceed a value of 9(D/Dc)2,which is the k value for a web without longitudinal stiffeners computed assuming partial rotational restraint from the flanges. 199912000 COMMENTARY Also, near points of dead-load contraflexure, both edges of the web may be in compression when stresses in the steel and composite sections due to moments of opposite sign are accumulated. In this case, the neutral axis lies outside the web. Thus, this revision also limits the minimum value of k to 7.2, which is approximately equal to the theoretical bend-buckling coefficient for a web plate under uniform compression assuming fixed boundary conditions at the flanges (Reference 7).

See also C10.34.3.2.1 (1997). C10.34.4 Transverse Intermediate Stiffeners An editorial revision is indicated to clanfy the definition of the handling requirement (referred to in following articles). A subsequent article explicitly indicates that the handling requirement need not be applied to longitudinally stiffened girders. Therefore, it is no longer necessary to repeat that statement in this article. The word tensile is added to the definition of the bending stress, F,, for use in Equation (10-30) to agree with the definition of the same term in this same equation given in the AISC ASD Specifications. An editorial revision is indicated to clarify that the moment of inertia of a transverse stiffener(s) is to be taken about the plane that is explicitly defined in Article 10.34.4.8. The mid-plane of the web is to only be used when there is a pair of stiffeners. The definition of the transverse stiffener spacing is modified to remove the word actual in front of the words distance between stiflener.

Earlier versions of the Standard Specifications indicated that the required stiffener spacing was to be used in calculating the term J given by Equation (10-32).

When the required spacing (which must be greater than or equal to the actual spacing) is used to compute J, the smallest possible required moment of inertia results. However, in situations where the actual stiffener spacing is used to compute J and I, the stiffener moment of inertia that is provided may not be suficient if the stiffener was originally designed based on the earlier criteria. Therefore, to avoid potential problems, the word actual is removed. Reference to the use of the maximum subpanel depth in designing transverse stiffeners on longitudinally stiff- C-121 ened girders is eliminated in the definition of D in Equation (10-32) for consistency with the revision to Article 10.34.5.6 discussed below. Finally, the local buckling capacity of a transverse stiffener is combined with the area requirement for the stiffener in a new Equation (10-32a). The stiffener area requirement is based upon the load that the stiffener must support. In many cases, the required stiffener area is zero indicating that the stiffener is not required to support any axial compression. In these cases, the lightly loaded stiffener can be more slender without concern for local buckling of the stiffener. The local buckling capacity of the stiffener can be tied to the required load the stiffener must support by setting the local buckling capacity equal to the vertical tension field load, which yields the new Equation (10-32a).

The local buckling capacity of the stiffener, Fa, is given by Equation (10-32b). The upper limit on blt of 16 currently specified in Article 10.34.4.10 is retained for lightly loaded stiffeners. C10.34.5 Longitudinal Stiffeners Equation (10-34) is modified to use the yield strength of the longitudinal stiffener in determining the required thickness of the longitudinal stiffener. The revised Equation (10-34) is equivalent to the LFD requirement. The stress in the longitudinal stiffener is controlled directly by the provisions of Article 10.34.5.3, and therefore, need not be indirectly controlled through the width-to-thickness requirement, as is currently the case. This revision states that the maximum spacing of transverse stiffeners on longitudinally stiffened girders be limited to 1.5 times the web depth rather than 1.5 times the maximum subpanel depth (both for intermediate stiffeners and at end panels). There is no known theoretical reason for using the subpanel depth in this requirement. Using the subpanel depth unnecessarily complicates the provision. This revision eliminates the requirement to use the maximum subpanel depth instead of the total panel depth when designing the transverse stiffeners on longitudinally stiffened girders. There is no known theoretical reason for using the subpanel depth in these requirements. The effect of the longitudinal stiffener is not considered in determining the shear capacity of a girder and it has not been C-122 HIGHWAY BRIDGES studied in enough detail to do so. Using the subpanel depth in this requirement may lead to confusion and unintentional design errors. C10.36 COMBINED STRESSES Table C10.36A Bending-Compression Interaction Coefficients The current lower limit of 0.4 on the C, coefficient contained in the amplification factor for members under combined bending and axial force comes into effect for end moment ratios less than or equal to -0.5. The C, factor with the lower limit of 0.4 was originally adopted from the work of Austin (Reference 8), who originally intended the factor to apply to lateral-torsional buckling of beams, and not to the determination of second-order in-plane bending strength of beam columns. Unfortunately, the work of Austin was misinterpreted and his factor was applied to approximate the results of more exact in-plane second-order analyses of beam-columns. AISC then introduced a Cb moment-gradient correction factor for handling lateraltorsional buckling of beams, which happens to approximately equal the inverse of the C, factor presented by Austin with the lower limit of 0.4. Zandonini (Reference 9) subsequently pointed out that the C, factor could indeed be used effectively to determine the second-order bending strength of beam columns if the 0.4 limit was eliminated. Subsequently, AISC removed the lower limit of 0.4 in the fist edition of the AISC LRFD Specijications (Reference 10). Thus, it is recommended that the lower limit be eliminated in the Standard Specifications as well. A similar revision has been implemented in Article 4.5.3.2.2b of the AASHTO LRFD Bridge Design Specjications. C10.38 COMPOSITE GIRDERS C10.38.1 General Language is added to indicate that concrete on the tension side of the neutral axis can also be considered for computing fatigue stress ranges and fatigue shear ranges in ASD as permitted under the revised provisions of Articles 10.3.1 and 10.38.5.1 (see the Commentary discussion related to those articles). The AASHTO ASD lateral-torsional buckling check for constructibility in this article implicitly assumes that a load factor of approximately1.82 (110.55) is applied to the unfactored dead loads. The corresponding LFD lateraltorsional buckling check (Article 10.61.3) is made using a load factor of 1.3 applied to the unfactored dead loads. Thus, the current ASD constructibility check applies 1.4 (1.8211.3) times more dead load. To reduce this significant inherent conservatism in the ASD constructibility check and make it more equivalent to the LFD check (for reasons discussed previously under the Commentary to the revision to Article 10.34.2.1.5), the current ASD equation for the lateral-torsional buckling capacity in Table 10.32.1A should be multiplied by 1.4 when making this check. Similarly, the ASD web shear buckling check for constructibility in this article implicitly assumes that a load factor of approximately 1.75 (0.5810.33) is applied to the unfactored dead loads. The corresponding LFD shear buckling check (Article 10.61.2) is made using a load factor of 1.3 applied to the unfactored dead loads. Thus, the current ASD constructibility check applies 1.35 (1.7511.3) times more dead load. To reduce this significant inherent conservatism in the ASD constructibility check and make it more equivalent to the LFD check, the current ASD equation for the shear buckling capacity should be multiplied by 1.35 when making this check, which results in the revised Equation (10-57a). It is also specified that the sum of the noncomposite and composite dead-load shears be used in making this check. Both the noncomposite and composite dead-load shears are critical in checking the stability of the web during construction. See also C10.38.1.7 (1997). C10.38.4 Stresses This article specifies the ASD requirement for minimum longitudinal reinforcement in the concrete deck. Because of the effect of moving live loads, points of deadload contraflexure have little meaning in continuous bridges.

requirements for the design of the bracing. Reference 11 provides some guidance in this regard. Generally, a larger number of parallel girders requires stronger bracing than would a fewer number of girders. The required bracing strength is a function of the force in the compression flange being braced. Since bracing is essentially resisting the tendency of the compression flange to move, it is most effective when attached as close as possible to the flange. The restraining force must be applied to the flange along some path between its point of connection and that flange. It should also be mentioned that Reference 11 can provide guidance on unusual cases of partially braced members not handled directly by current specification equations. The yield stress F, is replaced with the factored bending stress in the compression flange fb in determining whether or not the load-shedding factor Rbisequal to 1.0 for a longitudinally stiffened girder. As in ASD, the compression-flange stress is used in checking for local web bend-bucklingsince web bend-buckling is controlled by flange strain. Since this limit does not represent the maximum permitted web slenderness, but is only used to determine if local web bend-buckling has occurred, an upper limit on the web slenderness is not specified. In addition, lower limits are placed on the bendbuckling coefficient k for a longitudinally stiffened girder for reasons discussed previously (see commentary on revisions to Article 10.34.3.2.1). The values of the constant X given in the article reflect different assumptions of support provided to the web by the flanges to prevent local web bend-buckling. For composite sections in positive moment regions, using the area of the steel top flange by itself (which is typically smaller than the bottom flange) to determine which value A to use, is too conservative because of the support offered to the web by the top flange and concrete deck acting together. Thus, it is indicated that the depth of the web in compression D, relative to Dl2 instead be used to determine which value of h should be used to better handle composite sections. Language is also inserted at the end of this article to indicate that sections of partially braced members must satisfy the web thickness limits given by Equations (10-104) or (10-log), as applicable, subject to the requirements of Article 10.49.2 or 10.49.3 (with the exception noted below for constructibility-seethe commentary to the revisions to Article 10.61.1). As a result, the upper limit on web slenderness in the statement immediately above Equation (10103d)is redundant and need not be specified. Because this web slenderness limit is removed, footnote b to Article 10.48.4.1 is no longer required and the lateral-torsional buckling equations in this article can be applied to any general case (including the constructibility case). Language similar to the language in the existing paragraph at the end ofArticle 10.48.4.1, which r e f e d to footnote b, has been inserted in Articles 10.48.5.1 and 10.49.2 instead. Sections of partially braced members must also satisfy the compression-flange slenderness requirement given by the revised Article 10.48.2.1(a). The language regarding violation of the web thickness requirement in Article 10.48.2.1(b) is eliminated in this article because this condition is now handled sufficiently and more clearly by the direct incorporation of the Rbfactor in new Equation (10-99) in Article 10.48.2 (see above). It is indicated in this article that the web thickness of transversely stiffened girders is also subject to the thickness requirement specified in Article 10.49.2, which applies to singly symmetric transversely stiffened sections where D,exceeds Dl2. C10.48.5 Transversely Stiffened Girders C-128 HIGHWAY BRIDGES Language is also added to indicate that if the web slenderness Dlt, of a symmetric transversely stiffened girder exceeds the upper limit given by Equation (10-104), either the section must be modified to comply with the limit or longitudinal stiffeners must be provided. Similar language was formerly located at the end of Article 10.48.4.1. The yield stress F, in the denominator of Equation (10-104) is not replaced with the factored bending stress in the compression flange fb because the current slenderness limit (based on F,) defines a somewhat arbitrary upper bound below which fatigue due to excessive lateral web deflections is not considered to be a concern. To exceed this upper bound, it is felt that additional specification requirements would need to be inserted to directly control local web bend-buckling under the fatigue loading. It was decided not to include these additional specification requirements at this time. For completeness, reference to Articles 10.50 (Composite Sections), 10.51 (Composite Box Girders), and 10.53 (Hybrid Girders) is added for the computation of the maximum bending strength of transversely stiffened girders. The indicated revisions to this LFD article parallel the revisions to ASD Article 10.34.4.7 (see the earlier discussion on the revisions to Article 10.34.4.7). A definition of d,is added without making a distinction between the actual and required spacing, for reasons discussed previously.

As a result of the revisions to the area requirement for transverse stiffeners in the new Equation (10-106a) (see the earlier discussions on the revisionsto Article 10.34.4.7), the previous Equation (10-104) has been replaced with a revised upper limit of 16 on the slenderness ratio in new Equation (10-105). The previous Equation (10-104) was intended to ensure that local buckling of the stiffener would not occur if the stiffener were loaded to its yield load.

However, in many cases, the stiffener is not required to support any axial compression. Therefore, the local buckling capacity is now tied to the required load the stiffener must support through the uses of the new Equations (10-106a) and (10-106b). C10.48.6 Longitudinally Stiffened Girders The existing language in this article refers to the requirements for symmetrical girders only. Therefore, language is added at the end of this article to indicate that singly symmetric sections are subject to the requirements of Article 10.49.3. As for transversely stiffened girders without longitudinal stiffeners, the yield stress F, in the denominator of Equation (10-109) is not replaced with the factored bending stress in the compression flange fb,because the current slenderness limit (based on F,) defines a somewhat arbitrary upper bound below which fatigue due to excessive lateral web deflections is not considered to be a concern. To exceed this upper bound, it is felt that additional specification requirements would need to be inserted to directly control local web bend-buckling under the fatigue loading. It was decided not to include these additional specification requirements at this time. See also C10.48.6.1 (1997). For completeness, reference to Articles 10.50.1.2 (Noncompact Composite Sections in Positive Bending), 10.50.2.2 (Noncompact Composite Sections in Negative Bending), 10.51 (Composite Box Girders), and 10.53 (Hybrid Girders) is added for the computation of the maximum bending strength of longitudinally stiffened girders. The existing reference to Article 10.48.8.1 is replaced with the correct reference to Article 10.48.8.2 (see similar reference given in Article 10.48.5.2). The current reference in this article to Article 10.48.8.1 is changed to the more correct reference to Article 10.48.8, which parallels a similar reference given in Article 10.48.5.3. The indicated revision to this article clarifies that the width-to-thickness ratio for a longitudinal stiffener is to be checked using the yield strength of the longitudinal stiffener in Equation (10-105). Also, a provision is added that the factored bending stress in the longitudinal stiffener is not to exceed the yield strength of the longitudinal stiffener, which parallels a similar requirement given in ASD Article 10.34.5.3. A definition has been added to clanfy that the moment of inertia of the longitudinal stiffener'is to be taken about the edge of the stiffener in contact with the web plate. This revision eliminates the requirement to use the maximum subpanel depth instead of the total panel depth when designing the transverse stiffeners on longitudinally stiffened girders. There is no known theoretical reason for 199912000 COMMENTARY using the subpanel depth in these requirements. The effect of the longitudinal stiffener is not considered in determining the shear capacity of a girder and it has not been studied in enough detail to do so. Using the subpanel depth in this requirement may lead to confusion and unintentional design errors. Also, the words 'at D15' at the end of this requirement are considered superlluous and are removed since the longitudinal stiffener does not necessarily have to be located at Dl5. A modificationis made to indicate that only the radius of gyration, r, and not the moment of inertia, I, of the longitudinal stiffener is to be computed including a web strip up to 18t, in width. The additional web strip contributes little to the moment of inertia of the stiffener. Also, the allowable stress design provisions, which do not include a radius of gyration requirement, do not permit the inclusion of the web strip when calculating the moment of inertia of the stiffener. C-129 C10.49.2 Singly Symmetric Sections with Transverse Stiffeners The word 'unsymmetrical' is replaced with the words 'singly symmetric' in the heading to this article. Language is also added at the end of this article to indicate that if the web slenderness DJt, for the singly symmetric section exceeds the upper limit given by Equation (10- 120), either the section must be modified to comply with the limit or else longitudinal stiffeners must be provided. Sirnilar language was formerly located at the end of Article 10.48.4.1. C10.49.3 Longitudinally Stiffened Singly Symmetric Sections The word 'unsymmetrical' is replaced with the words 'singly symmetric' in the heading to this article. C10.48.8 Shear Editorial revisions are indicated in this article to be consistent with the editorial revisions to previous articles discussed above. The word 'I-shaped' is removed since the shear provisions in this article also apply to box-girders. Other revisions are made to clarify the existing provisions. Equation (10-118a) is added to this article to better accommodate composite non-compact sections. The maximum bending strength of these sections is now expressed in terms of the maximum strength F,, of the compression and tension flanges, expressed in terms of stress rather than moment, in revised Articles 10.50.1.2 and 10.50.2.2. The moment-shear interaction relationship for these sections is revised accordingly. The word 'unsymmetrical' is replaced with the words 'singly symmetric'. The word 'unsymmetrical' is replaced with the words 'singly symmetric'. See also C10.49.3.2 (1997). C10.49.4 Singly Symmetric Braced Noncompact Sections Editorial revisions are made to the heading and to the wording in this article for consistency with revisions to preceding articles. The current reference in this article to Article 10.48.2.1 is changed to the more correct reference to Article 10.48.2. C10.49.5 Partially Braced Members with Singly Symmetric Sections The indicated revisions to this LFD article parallel the revisions to ASD Articles 10.34.4.2 and 10.34.5.5 (see the earlier discussion on the proposed revisions to Articles 10.34.4.2 and 10.34.5.5). Editorial revisions are made to the heading and wording in this article for consistency with revisions to preceding articles. C10.50 COMPOSITE SECTIONS C10.49 SINGLY SYMMETRIC SECTIONS The heading for this LFD article is renamed from 'UNSYMMETRIC BEAMS AND GIRDERS' to 'SINGLY SYMMETRIC SECTIONS' to more appropriately reflect the fact that the provisions under this heading refer to 'sections' that are symmetric about one axis of the cross section. The heading for this LFD article is changed from 'COMPOSITE BEAMS AND GIRDERS' to the more appropriate heading of 'COMPOSITE SECTIONS' since all the provisions in this article apply to 'sections'. The words 'beams and girders' are changed to the word 'sections' in the first sentence of this article for consistency. C-130 C10.50.1 HIGHWA.Y BRIDGES Positive Moment Sections The words 'of Composite Beams and Girders' are considered redundant and are removed form the heading for this article for consistency with the revision to the heading for Article 10.50. C10.50.1.1 Compact Sections The words 'beams and girders' are changed to the word 'sections' in the first sentence of this article for consistency. Also, AASHTO M 270 Grade HPS70W (ASTM A 709 Grade HPS70W) steel has been added to the list of steels that have the demonstrated ability to reach the plastic moment capacity M,. Revisions to this article clarify that only composite sections of constant-depth flexural members without longitudinal web stiffeners and without holes in the tension flange (refer to the commentary to Article 10.18.2.1.5) can qualify as compact sections for positive bending. Sufficient research has not yet been conducted on composite sections of variable-depth members with or without longitudinal web stiffeners, composite sections of constantdepth members with longitudinal web stiffeners or on composite sections of variable- or constant-depth members with holes in the tension flange to determine if these sections can achieve the full plastic moment capacity in positive bending. The maximum bending strength of composite sections in positive flexure of variable-depth members, or with longitudinal web stiffeners, or with holes in the tension flange is to be determined from the provisions of Article 10.50.1.2 (see below). The words 'beams and girders' are changed to the word 'sections' throughout this article for consistency. An editorial change is also indicated immediately above Equation (10-129d). The former Article 10.50(f) is now Article 10.50(c). Finally, the P factor of 0.7 in the definition of D' for Equation (10-129a) has been extended to include Grade HPS70W and 70W steels based on research at the University of Nebraska at Lincoln. C10.50.1.2 to moments whereas it does apply to stresses. As a result, it becomes more convenient and more correct to express the maximum strength in terms of stress. For tension flanges, the sum of the accumulated factored stresses is not to exceed the maximum strength, F,, of the flange taken equal to F,. For compression flanges, the maximum strength, F,, of the flange is taken equal to FyRb.The flange-stress reduction factor Rbaccounts for the increase in compression-flange stress that results due to local web bend-buckling and is to be computed according to the provisions of Article 10.48.4.1. To provide some additional relief at more lightly loaded sections, Rbis to be computed using the actual factored compression-flange bending stress fbin place of the term MJS,, when Rbis computed using Equation (10-103b) in Article 10.48.4.1; fb cannot exceed F,. In addition, for composite sections in positive moment regions, the revised article states that the area of the compression flange Afcin Equation (10-103b) for the computation of Rbis to be taken as the transformed area of the top flange and concrete deck that yields the depth of the web in compression D, calculated in accordance with Article 10.50(b).

The effective transformed Arc, can be derived as follows: where: A,, AR A, A, = total area of section = area of tension flange = area of web = effective transformed area of compression flange and concrete deck Using the web depth D for simplicity instead of the distance between the centerline of A* and Arcto compute the distance to the neutral axis from the effective top flange, which is equivalent to Dc in this case, gives (referring to Figure 1): Noncompact Sections This article is revised to express the maximum strength of non-compact composite sections in terms of stress rather than moment. For a composite noncompact girder, dead- and live-load moments due to the factored loads are applied to different sections and should not be directly summed when at elastic stress levels (up to and including F,); that is, the principle of superposition does not apply D -AW 1 t Aft FIGURE 1 199912000 COMMENTARY Rearranging Equation (2) yields: Substituting Equation (3) into Equation (1) and solving for the effective transformed Arcgives: The use of this effective Af, in Equation (10-103b) for Rbis more appropriate for composite sections in positive bending and is more consistent with the original derivation of Rb which results in a less critical value of Rbfor these sections. The revised Article 10.50.1.2.1 also states that the resulting Rbfactor be distributed to the top flange and concrete deck in proportion to their relative stiffness. When the top flange is composite, the stresses that are shed from the web to the flange are resisted in proportion to the relative stiffness of the steel flange and the concrete deck. The Rb factor is to be applied only to the stresses in the steel flange. Thus, whenever Equation (10-103b) is applicable to a composite section under positive moment, a modified Rbfactor for the top flange (termed Rb) can be computed as follows: C-131 Essentially, the calculated load-shedding factor to be applied to the effective transformed area is first proportioned to the steel flange and is then used to compute a modified (smaller) load-shedding factor for the flange. It should be noted that in most cases, the above procedure will only need to be implemented for composite noncompact sections in positive Jlexure with longitudinally stiffened webs that have relatively large values of D, For composite sections in positive bending without longitudinal web stiffeners, either the section will typically qualijj as compact, or should the section be noncompact, the Rb factor calculated from Equation (10-103b) will typically exceed 1.0 (and must therefore be set equal to 1.0) unless D, is unusually large. Lastly, the revised article states that the revised web thickness requirement of Article 10.48.2.1(b) shall apply. When conventional cast-in-place composite decks are used, the lateral bracing requirement of Article 10.48.2.1(c) and the compression-flange slenderness requirement of Article 10.48.2.1(a) need not be checked.

However, when precast decks are used with the stud shear connectors clustered in pockets several feet apart, the Engineer may wish to limit the maximum bending strength of the top (compression) flange according to Equation (10-99) in Article 10.48.2 and check the limiting blt ratio specified in Article 10.48.2.1(a) in order to ensure that local buckling of the flange will not occur in the regions between the shear connectors. C10.50.2 Negative Moment Sections The words 'of Composite Beams and Girders' are considered redundant and are removed from the heading for this article for consistency with the revision to the heading for Article 10.50.

The current references to Articles 10.48 and 10.49 are replaced with the more correct references to Articles 10.50.2.1 or 10.50.2.2, as applicable, for the computation of the maximum bending strength. Articles 10.50.2.1 and 10.50.2.2 refer back to the appropriate provisions of Article 10.48 where necessary. C10.50.2.1 Compact Sections where: Rb = factor computed from Equation (10-103b) using the effective transformed AfcfromEquation (4) A~ = area of the top flange AfC = transformed area of the top flange and concrete deck from Equation (4) Revisions are proposed to clarify that compact composite sections of constant-depth flexural members without longitudinal web stiffeners and without holes in the tension flange (refer to the commentary to Article 10.18.2.1.5) can qualify as compact sections for negative bending. Sufficient research has not yet been conducted on composite sections of variable-depth members with or without longitudinal web stiffeners, composite sections of constantdepth members with longitudinalweb stiffeners, or on composite sections of variable- or constant-depthmembers with C-132 HIGHWAY BRIDGES holes in the tension flange to determine if these sections can achieve the full plastic moment capacity in negative bending.

Also, AASHTO M 270 Grade HPS70W (ASTM A 709 Grade HPS70W) steel has been added to the list of steels that have the demonstrated ability to reach the plastic moment capacity M,. C10.50.2.2 Noncompact Sections This article is revised to correspond with the above revisions to Article 10.48.2 for braced noncompact sections, except that the maximum strength is specified separately for the tension and compression flange and is expressed in terms of stress rather than moment for reasons discussed previously. When all requirements of Article 10.48.2.1 are satisfied (including the lateral bracing requirement), the maximum strength, F,, of the tension flange is taken equal to F, and the maximum strength, F,, of the compression flange is taken equal to Fc,Rb. F,, represents a critical compression-flange local buckling stress, which is determined based on the actual slenderness of the compression flange as specified in Article 10.48.2 and cannot exceed Fy.Therefore, a compression flange with a larger slenderness (up to the limiting value of 24 specified in Article 10.48.2.1(a)) can be used at more lightly loaded sections. To achieve F,, equal to Fy at critical sections, the compression flange slenderness (based on the full flange width b) cannot exceed the limiting values indicated in revised Table 10.48.2.1A. The compressionflange capacity is also modified by the flange-stress reduction factor Rb. The flange-stress reduction factor Rb accounts for the increase in compression-flange stress that results due to local web bend-buckling and is to be computed according to the provisions of Article 10.48.4.1. To provide some additional relief at more lightly loaded sections, Rbis to be computed using the actual factored compression-flange bending stress fb in place of the term M,./Sxcwhen Rbis computed using Equation (10-103b) in Article 10.48.4.1; fbcannot exceed F,.

When all requirements of Article 10.48.2.1 are satisfied, except for the lateral bracing requirement given by Equation (10-101), the maximum strength, F,, of the compression flange is again taken equal to FcrRb.However, in this case the calculated maximum strength of the compression flange cannot exceed the maximum strength for the limit state of lateral-torsional buckling, which is to be calculated as the limiting stress Mu/Sxc,where Mu and S,, are determined according to the provisions of Article 10.48.4.1 for partially braced members.

Mu in Article 10.48.4.1 includes the flange-stress reduction factor Rb.For consistency, when computing the moment- gradient correction factor Cb in Article 10.48.4.1, the smaller and larger factored compression-flange bending stresses, fb, at each end of the unbraced segment of the beam are to be substituted for the smaller and larger end moments MI and M2, respectively. This article specifies the LFD requirement for minimum longitudinal reinforcement in the concrete deck. To address at least some of the issues discussed above under ASD Article 10.38.4.3, it is proposed that the minimum one-percent longitudinal reinforcement be placed wherever the longitudinal tensile stress in the deck due to either the construction loads or the overload specified in Article 10.57 exceeds 0.9fr, where f, is the modulus of rupture for the concrete specified in Article 8.15.2.1.1. The factor 0.9 represents a conservative resistance factor applied to the modulus of rupture to provide additional assurance against concrete cracking. In addition, the required longitudinal reinforcement is to be No. 6 bars or smaller spaced at not more than 12 inches to ensure adequate distribution of the reinforcement to control the crack size.

By controlling the crack size, the concrete deck can be considered to be effective in tension for serviceability checks (e.g. fatigue and overload) as long as adequate shear connection between the deck and the girders is also provided (see discussion on Article 10.58.1). As a result of this requirement, the minimum longitudinal reinforcement will likely need to be extended beyond the dead-load points of contraflexure. The area of the concrete slab to be used in this requirement is also defined in the specification as the structural thickness times the entire width of the deck for reasons discussed previously (see the earlier commentary on the revisions to Article 10.38.4.3).

C10.51 COMPOSITE BOX GIRDERS C10.51.5 Compression Flanges The equation for the buckling coefficient for a longitudinally stiffened bottom flange plate in the current specifications assumes that the plate and stiffeners are infinitely long and ignores the effect of any transverse bracing or stiffening. As a result, when the number of stiffeners exceeds two, the moment of inertia of the stiffeners required to achieve the desired k value increases dramatically so as 199912000 COMMENTARY to become impractical. In new designs where an exceptionally wide box flange is required, it may indeed become necessary to provide more than two longitudinal stiffeners. Rating of older bridges with more than two longitudinal stiffeners becomes problematic if the current requirements are employed because the longitudinal stiffeners are not likely to provide enough moment of inertia to satisfy the unrealistically high requirement. Thus, the revision to this article indicates that the number of longitudinal flange stiffeners preferably shall not exceed two. For cases where the number of longitudinal stiffeners exceeds two, it is suggested that additional transverse stiffeners (beyond the recommended transverse stiffeners placed near points of dead load contraflexure) be added to reduce the required size of the longitudinal stiffeners to a more practical value. Current ASD specifications contain provisions for the design of flanges stiffened both longitudinally and transversely in Article 10.39.4.4, which can be modified for use with the strength design method. Included are requirements related to the spacing and stiffness of the transverse stiffeners. The bottom strut of the transverse interior bracing in the box can be considered to act as a transverse stiffener for this purpose if the strut satisfies the appropriate stiffness requirements. C-133 states that Fyis to be taken as the yield strength of the longitudinal stiffener. The revision to Article 10.48.6.3(a) further states the factored bending stress in the longitudinal stiffener is not to exceed the yield strength of the stiffener, which eliminates the need to limit the stress in the stiffener indirectly by using Fy of the adjacent flange in checking the stiffener width-to-thickness and radius of gyration requirements. An additional revision regarding the placement of longitudinal web stiffeners in yielded portions of the web parallels a similar revision given in a new ASD Article 10.40.2.1.4 (see the earlier discussion on the new Article 10.40.2.1.4).

C10.53.1 Noncomposite Hybrid Sections The word 'girders' is replaced with the more appropriate word 'sections' in the heading for this article. C10.53.1.1 Compact Sections An editorial revision is made to clarify the definition of Fyf.

C10.53.1.2 Braced Noncompact Sections The indicated revisions in this new LFD article parallel the revisions to ASD Article 10.39 (see the earlier discussion on the proposed revisions to Article 10.39). C10.51.7 Design of Flange to Web Welds This new LFD article on design of flange-to-web welds for box girders parallels the existing ASD Article 10.39.5. The same requirements should be applied to box girders designed by ASD or LFD. C10.53 HYBRID GIRDERS This article states that for hybrid girders, Fyis to be taken as the specified minimum yield strength of the element under consideration with the exceptions listed. The exceptions listed under item (1) are revised to remove the reference to Article 10.48.2.1 (b) (since the current equation in that article has been removed) and to add a reference to Article 10.50.1.1.2, which contains a web slenderness requirement with Fyin the denominator. In these cases Fyof the compression flange is to be used in calculating the web slenderness requirement. The first sentence under item (2) in this article is eliminated since the above revision to Article 10.48.6.3(a) Existing Equation (10-146) is revised to correspond with the revised Equation (10-98) in Article 10.48.2. A new Equation (10-146a) is also added to this article, which represents the new Equation (10-99) in Article 10.48.2 with the hybrid factor R added. In addition, language is added to indicate that the hybrid factor R is to be taken as 1.0 at sections where the stress in both flanges caused by the maximum design load does not exceed the specified minimum yield strength of the web since web yielding is assumed not to occur in this case. C10.53.1.3 Partially Braced Members The heading for this article is revised to correspond to the revised heading for Article 10.48.4. The language in this article is also revised for consistency with the revised language of Article 10.48.2 that refers to the requirements of Article 10.48.4.1 for computing the maximum permissible compression-flange capacity for a partially braced member. C10.53.2 Composite Hybrid Sections The word 'girders' is replaced with the more appropriate word 'sections' in the heading for this article. Language is also added to differentiate the computation of the maximum strength for compact and noncompact C-134 HIGHWAY BRIDGES composite hybrid sections in both positive and negative bending. The current article does not include provisions for the design of compact composite hybrid sections. Therefore, the appropriate language is added to permit their use. C10.53.3 Shear This new article represents the previous Article 10.53.1.4 (Transversely Stiffened Girders), which has been moved here to improve the overall flow of the specification. C10.56.1 Connectors C10.56.1.3 Bolts and Rivets This editorial revision eliminates the reference to Table 10.57A in the definition for the design shear strength of a rivet or bolt, F,. Table 10.57Aprovides slip resistances for bolts. Under combined tension and shear, slip resistance is computed according to the provisions of Article 10.57.3.4. Article 10.56.1.3.3 computes the tensile strength of a bolt or rivet under combined tension and shear. C10.54 COMPRESSION MEMBERS C10.56.1.4 Slip-Critical Joints C10.54.1 Axial Loading C10.54.1.1 Maximum Capacity (Axial Load) A footnote to this article is added regarding the computation of the maximum capacity of concentrically loaded columns in LFD. The language in this footnote is parallel to the language added in a similar footnote to ASD Table 10.321A (see earlier discussion of the revisions to Table 10.32.1A). C10.54.1.2 Effective Length The reference to the existing footnote in this article is revised to accommodate the addition of the new footnote discussed under Article 10.54.1.1. C10.54.2 Combined Axial Load and Bending C10.54.2.2 Equivalent Moment Factar C The current lower limit of 0.4 on the C coefficient contained in the amplification factor for members under combined bending and axial force (in LFD) is eliminated for consistency with the revision to Table 10.36A discussed earlier. C10.56 SPLICES, CONNECTIONS, AND DETAILS Table ClQJ6A Design Strength of Connectors Footnote d has been applied to the shear strength of ASTM A 307 bolts to indicate that the joint length correction factor also applies when determining the shear strength of these bolts. Also, language has been added at the end of footnote d in order to clarify the definition of the 50-inch length used in determining whether or not to apply the joint-length correction factor when calculating the shear strength of high-strength bolts in flange splices. Language has been added to clarify that in addition to checking slip at overload, the bolts in slip-critical connections must also satisfy the shear and bearing strength requirements of Article 10.56.1.3 under the maximum design loads in Load Factor Design. C10.57 OVERLOAD A new paragraph is added to clanfy the definition of overload when consideringAASHTO Group I, Group IA, or Group I1 load combinations. The existing language regarding moment redistribution is moved into this paragraph so it applies to both noncomposite and composite sections. A provision to check web bend-buckling at overload is added. Equation (10-173) in Article 10.61.1 is used to make the check. For composite sections, D, is to be calculated considering the accumulated bending stresses, as specified in Article 10.50(b). Revised Article 10.57.2 (see below) will allow the option to compute overload flange stresses caused by loads acting on the appropriate composite section assuming the concrete deck to be fully effective for both positive and negative moment if certain conditions are met. If the concrete deck is assumed to be fully effective in negative moment regions, more than half of the web will typically be in compression increasing the susceptibility of the web to bend-buckling. Since the design checks at overload are considered to be serviceability checks, web bend-buckling at overload should be limited. Sections that do not comply with Equation (10-173) should be modified to comply with the requirement; longitudinal web stiffeners should not be added to satisfy this serviceability requirement. C10.57.1 Noncomposite Sections This revised article limits the maximum overload flange stress at noncomposite sections (versus girders) to 0.8F,. 199912000 COMMENTARY The hybrid factor R is eliminated because web yielding, should it occur, is limited at overload. C10.61.1 C-135 Web Bend-Buckling The indicated revision is made in LFD to be consistent with a similar revision made in ASD (see the previous discussion on the revisions to ASD Article 10.3.1). Language is added to indicate that if a longitudinal stiffener is used to comply with the web bend-buckling check for constructibility, it must be placed at a location on the web that satisfies Equation (10-173) for constructibility and that also satisfies the strength criteria for the maximum design loads.

The revised language also indicates that this location may not necessarily correspond to the recommended optimum location of the stiffener specified in Article 10.49.3.2(a). The recommended optimum location can serve as an initial trial location, but the stiffener may have to be moved vertically up or down from the optimum location in order to satisfy both the constructibility and strength criteriaparticularly in positive bending regions of composite girders and in areas of stress reversal. By judicious placement of the longitudinal stiffener in regions of stress reversal, it may be possible to place only one stiffener on the web (rather than two) such that all design criteria are adequately satisfied with either edge of the web in compression. The existing language indicating that the longitudinally stiffened girder must meet the requirements of Articles 10.48.6 and 10.49.3 is considered redundant and is removed (see also the next paragraph below). These requirements must be satisfied when the girder is in the final condition. A paragraph is added to the end of this article indicating that the web thickness requirements specified in Articles 10.48.5.1,10.48.6.1,10.49.2, and 10.49.3.2(b)arenot to be applied to the constructibility load case. Local web bend-buckling is explicitly checked for the constructibility load case according to Equation (10-173). The requirements in the above articles are intended to apply only when the girder is in the final condition. The use of these requirements (which have the yield stress Fy in the denominator) is too conservativefor the constructibilityload case since compression stresses in the web are typically below F, during construction. Checking these requirements using the factored noncomposite dead load compression flange stress fb in place of Fy is redundant since web bend-buckling is already explicitly checked, as mentioned earlier. Finally, an editorial revision is made to insert the lower limits for the bend-buckling coefficient for longitudinally stiffened girders (see earlier discussion on the revisions to ASD Article 10.34.3.2.1). C10.61 CONSTRUCTIBILITY C10.61.2 An editorial change is made to change the load factor 'y' to the load factor ' 7 ' . See also C10.61 (1997). It is specified that the sum of the factored noncomposite and composite dead-load shears be used in checking for shear buckling of the web during construc- C10.57.2 Composite Sections This revised article limits the maximum overload flange stress at composite sections (versus girders) to 0.95FYThe hybrid factor R is eliminated because web yielding, should it occur, is limited at overload. For consistency with other serviceability checks (e.g. fatigue-see the discussion on revised Article 10.58.1), overload flange stresses caused by loads acting on the appropriate composite section may be computed assuming the concrete deck to be fully effective for both positive and negative moment if: 1) shear connectors are provided along the entire length, and 2) the longitudinal reinforcement satisfies the provisions of Article 10.50.2.3. By providing shear connectors to ensure composite action and by controlling the crack size at overload with the minimum longitudinal reinforcement, it is logical to consider the concrete deck to be effective in tension at overload for loads acting on the appropriate composite section for reasons discussed previously (see discussion on revisions to ASD Article 10.3.1). Should the concrete deck be considered effective in tension, for consistency, the resulting stresses due to loads acting on the appropriate composite section are to be combined with the stresses due to loads acting on the noncomposite section to calculate D, for checking web bend-buckling. C10.57.3 Slip-Critical Joints The words for H or HS rmck load only have been removed.

There is no known theoretical reason for this requirement. The design slip force should not be exceeded in connections subject to either H or HS truck or lane loading. C10.58 FATIGUE C10.58.1 General Web Shear Buckling C-136 HIGHWAY BRIDGES tion.

Both the non-composite and composite dead-load shears are critical in checking the stability of the web during construction. C10.61.4 Compression-FlangeLocal Buckling The blt requirement for the compression flange in Equation (10-174) is rewritten in terms of the full flange width b rather than the projecting flange width byfor consistency with previous revisions.

A practical upper limit of 24 is placed on the compression-flange slenderness limit for constructibility, which corresponds to the upper limit of 24 specified in ASD. Should the load-shedding factor Rb be less than 1.0, the compression-flange stress is theoretically increased. Thus, the revised article also requires that fdlbe taken as the factored non-composite dead load compression-flange stress divided by Rb, but not to exceed F,. REFERENCES 1. Barzegar, F. and S.

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Guide to Stability Design Criteria for Metal Structures, Fifth Edition, Structural Stability Research Council, Edited by Theodore V. Galambos, 1998. COMMENTARY TO SECTION 17--SOIL-REINFORCED CONCRETE STRUCTURES INTERACTION SYSTEMS C17.6.4.7, C17.7.4.7, and C17.8.5.7 Consideration of thrust in determining flexural stresses under service load conditions can have a significant effect on reinforcing requirements to meet the provisions of Section 17; however, the equations to make this calculation are not commonly available. As a result, excessive reinforcement areas are often specified. The proposed revisions incorporate equations taken from ACI SP-3, 1965 and make them readily available to design engineers. The proposed changes will reduce, sometimes substantially, the amount of reinforcement in reinforced concrete sections compared to those that ignore the benefit of compressive thrust. See also C17.6.4.7 (1997). COMMENTARY TO SECTION 1843OIL-THERMOPLASTIC PIPE INTERACTION SYSTEMS This change is recommended as a result of work done under NCHRP Project 4-24 as reported in NCHRP Report 429 to address environmental stress cracking in AASHTO M 294 polyethylene culverts. Approval of this change is made provisionally pending approval by the AASHTO Subcommittee on Materials of those changes made to AASHTO M 294 that are recommended in NCHRP Report 429. The change in cell class is made to reflect changes in the Slow Crack Resistance (SCR) tests. The current cell class number for the ESCR is "2." This number should be changed to "0" if the SP-NCTL test is adopted by the AASHTO Subcommittee on Materials in August 2000. The cell class "0"in ASTM D335 is referred to "unspecified." Instructions for the SP-NCTL test procedure and requirements will be incorporated into the appropriate sections of the Material Specification to guide the user. See also C18.4.3.1.2 (1997). 1999/2000 COMMENTARY DIVISION I1 COMMENTARY TO SECTION 7-EARTH RETAINING SYSTEMS C7.3 MATEIUALS C-137 C11.3.2.1 Material References to the AASHTO M 292 (ASTM A 194) Grades 2 and 2H nuts have been eliminated. These nuts are primarily for pressure-vessel applications and are not widely used for bridges. C11.3.2.5 Alternative Fasteners Reference to the ASTM F 1852 Specification has been added. As of this writing, there is no equivalent metric specification. The current specifications do not provide a clear criteria for determining whether or not a given block has adequate freeze-thaw resistance. Furthermore, ASTM C 666 has more than one testing protocol, neither of which have an identified acceptance criteria. ASTM C 1262 is a newly developed protocol specifically developed for dry-cast concrete blocks, and only just recently has information been available to identify what the acceptance criteria should be when using this protocol. ASTM C 1372 contains the acceptance criteria for dry-cast concrete blocks, but is not as stringent as desired. Hence, the limit of 1% weight loss after 150 cycles is provided in this revision. Dry-cast concrete block durability in a freeze-thaw environment is potentially a significant problem, as evidenced by the recent experience of the Minnesota DOT. Clarifying the protocol and using updated testing methods will help to minimize this problem. The requirements of this article have been revised to correspond with the requirements given in the ANSV AASHTO/AWS D1.5Bridge Welding Code. C7.3.6 Structure Backfill Material C11.4.7 Straightening Material C7.3.6.3 Mechanically Stabilized Earth Walls These revisions allow the definition of nonaggressive soil to be moved to Division I, since the definition of nonaggressive soil is needed for design purposes, and is not intended for the development of construction specifications. COMMENTARY TO SECTION 11CTEEL STRUCTURES C11.3.1.1, C11.3.1.4, C11.4.1, C11.4.3.3.3, C11.4.7, and C11.4.12.2.1 ASTM and the AASHTO Subcommittee on Materials have adopted a specification for HPS70W steels. Numerous highway bridges have been successfully fabricated using AWS D1.5, supplemented by the provisions in the AASHTO Guide Specijications for Highway Bridge Fabrication with HPS70W Steel. AASHTO M 270 (ASTM A 709) Grade HPS70W steels have been tested by the New York State Thruway Authority (NYSTA) up to 1245°F.Acopy of the report of the work done by High Steel Structures for NYSTA is available from FHWA. C11.4.1 Identification of Steels During Fabrication Color codes for steels as noted in the AASHTO M 160 (ASTM A 6) Specifications may also be used for identification purposes. This method is being eliminated by many owners due to the complexity of the code with many new material grades.

Hence, Table 11.4 has been deleted. C11.4.3.3.2 Cold Bending The requirements of this article have been revised to correspond with the requirements given in the ANSV AASHTO/AWS D1.5Bridge Welding Code. C11.4.11 Annealing and Stress Relieving Requirements for Grade HPS7OW steel have been added. C11.4.12.2.3 Temperature The requirements of this article have been revised to correspond with the requirements given in the ANSV AASHTO/AWS D1.5Bridge Welding Code. REFERENCES 1. American Concrete Institute, Publication SP-3, "Reinforced Concrete Design Handbook, Working Stress Method," 1965. 2. NCHRP Report 429, HDPE Pipe: Recommended Material Specifications and Design Requirements, Y. G. Husan, T. J. McGrath, 1999. 5/19/2018 AASHTO Standard Specifications 17th Edition 1/86 Section 10STRUCTURAL STEEL Part AGENERAL REQUIREMENTS AND MATERIAI.S 10.1 APPLICATION 10.1.1 Notations = total area of longitud

1.2.l) = load factor equal to 1.3 (Article 10.61)= F,,/Fu (Article l 11.53. J .2) angle of inclination of the web plate to thevertical (Articles 1 0.39.3. l and 10.51.3) ratio of total cross-sectional area to the cross-sectional area of both flanges (Article 10.15.2) distance from the outer edge of the tensionflange to the neutral axis divided by the depthof the steel section (Articles 10.40.2 and10.53.1.2) = amount of camber, in. (Article 10. 15.3) = dead load camber in inches at any point (Ar- ticle 10.15.3)= maximum value of ADL. in. (Article 10.15.3)= reduction factor (Articles 10.38.5.J .2, 10.56.1.1, and 10.56. 1.3) longitudinal stiffener coefficient (Articles 10.39.4.3 anl 10.51.5.4)= slip coefficient in a slip-critical joint (Article 10.57.3) 10.2 MATERIALS 10.2.1 General These specifications recognize steels listed in the fol-lowing subparagraphs. Other steels may be usecl; how-ever, theii properties, strengths, allowable stresses, andworkability must be established and specified. 10.2.2 Structural Steels Structural steels shall conform to the material desig-nated in Table 10.2A. (The stresses in this table are in pounds per square inch.) The modulus of elasticity of agrades of structural steel shall be assumed to b29,000,000 psi and the coefficient of linear expansio0.0000065 per degree Fahrenheit. 10.2.3 Steels for Pins, Rollers, and ExpansionRoclers Steels for pins, iollers, and expansion rockers shaconform to one of the designations listed in Tables 10.2Aand 10.2B, or shall be stainless steel conforming to ASTMA 240 or ASTM A 276 HNS 21800. 10.2.4 FastenersRivets and Bolts Fasteners may be carbon steel bolts (ASTM A 307)power-driven rivets, AASHTO M 228 Grades l or (ASTM A 502 Grades 1 or 2); or huh-strength bolts AASHTO M 164 (ASTM A 325) or AASHTO M 253(ASTM A 490). 10.2.5 Weld Metal Weld metal shall conform to the current require ments of the ANSI/AASHTO/AWS DI.5 Bridge Welding Code. 10.2.6 Cast Steel, Ductile Iron Castings, MalleableCastings, and Cast Iron 10.2.6.1 Cast Steel and Ductile Iron Cast steel shall conform to specifications for SteeCastings for Highway Bridges, AASHTO M 192 (ASTMA 486); Mild-to-Medium-Strength Carbon-Steel Castings for General Application, AASHTO M 103 (ASTMA 27); and Corrosion-Resistant Iron-Chromium, IronChromium-Nickel and Nickel-Based Alloy Castings foGeneral Application, AASHTO M 163 (ASTIVI A743)Ductile iron castings shall conform to ASTM A536. 10.2.6.2 Malleable Castings Malleable castings shall conform to specifications foMalleable Lon Castings, ASTM A 47, Gradc 35018 (minimum yield point 35,000 psi). 10.2.6.3 Cast Iron Cast iron castings shall conform to specifications forGray Lon Castings, AASHTO M 105, Class 30. 5/19/2018 AASHTO Standard Specifications 17th Edition 8/86 258 HIGHWAY BRIDGET TABI.E 10.2A Minimum Material PropertiesStructural Steel AASHTO Designation M 270Grade 36 M 270Grade 50 M 270Grade 50W M 270Grade HPS70W' M 270Grdes 100/100W Equivalent ASTMDesignation A 709 A 709 A 709 A 709 A 709Grade 36 Grade 50 Grade 50W Grade HPS70W Grades l00/ l00W Thickness of Pll1tC:s Up to 4 in.incl. Up to 4 in.incl. Up to 4 in.incl. Up to 2'/i in. Over 2'/i in. toincl. 4 in. incl. Shapes ' All group All groups All groups Not applicable Not applicable Not applicable Minimum TnsileStrength, Fa 58,000 65,000 70,000 90,000 1 10,000 100,000 Minimum Yield Pointor Minimum YieldStrength, F, 36,00o so,000 50,000 70,000 100,000 90,000 Except for the mandatory notch toughness and weldability requirements, the ASTM designations are similar to the AASHTO designations. Steels meeting the AASHTO requirements are prequalified for use in welded bridges. M 270 Gr. 36 and A 709 Gr. 36 are equivalent to M 183 and A 36. M 270 Gr. 50 and A 709 Gr. 50 are equivalent to M 223 Gr. 50 and A 572 Gr. 50.M 270 Gr. 50W and A 709 Gr. 50W are equivalent to M 222 and A 588. M 270 Gr. 70W and A 709 Gr. 70W are equivalent to A 852. M 270 Gr. 100/100W and A 709 Gr. 100/100W are equivalent to M 244 and A 514. AASHTOM 270 Grade HPS70W replaces AASHTO M 270 Grade 70W. The intent of this replacement is to encourage the use of high-performancesteel (HPS) over convenonal bridge steels due to its enhanced properties. AASHTO M 270 Grade 70W steel is still availab le, but should be used with the owners approval.Quenched and tempered aHoy steel structural shapes and seamless mechanical tubing meeting all mechanical and chemical requirements of A 709 GfdS 100/100W, except that the specified maximum tensile strength may be 140,000 psi for structural shapes and 145,000 psi for seamless mechan-ical tubing, shall be considered as A 709 Grades 100/l00W For nonstructural applications or bearing assembly components over 4thick, use AASHTO M 270 Gr. 36 (ASTM A 709 Gr. 36).f roups l and 2 include all shapes except those in Groups 3, 4, and 5. Group 3 includes L-shapes over 3/4 inch in thickness. HP shapes over 102 pounds/foot, and the following W shapes:Designation: W36 K 230 to 300 incl.W33 5 200 to 240 incl.W14 X 142 to 211 incl.W12 X 120 to 190 incl. Group 4 includes the following W shapes: W14 219 to 550 incl.Group 5 includes the following W shapes: W14 605 to 730 incl.For breakdewn of Groups 1 and 2, see ASTM A 6. TABLE 10.2B Minimum Material PropertiesPins, Rolleis, and Rockers Expansion Rollers Shall be Not less Than 4 Inches in Diameter AASHTO Designation with Size Limitations M 169 4 in. in dia. or M 102 to 20 in. in dia. M 102 to 20 in. in dia.
M 102 to 10 in, in dia. M 102 to 20 in. in dia.less ASTM Designation A 108 A 668 A 668 A 668 A 668Grade or Class Grades 1016 to 1030 incl. Class C Class D Class F Class G Minimum Yield Point, psiF, 36,000 37,500 50,000 May substitute rolled material of the same properties. For design purpose only. Not a part of the A 108 specificat ions. Supplementary material requirements should provide guarantee that material will meet these values. 5/19/2018 AASHTO Standard Specifications 17th Edition 9/86 DIVISION IDESIGN 25 Part B DESIGN DETAILS 10.3 REPETITIVE LOADING AND TOUGHNESSCONSIDERATIONS 10.3.1 Allowable Fatigue Stress Ranges Members and fasteners subject to repeated variations or reversals of stress shall be designed so that the maxi-mum stress does not exceed the basic allowable stressesgwen in Article 10.32 and that the actual range of stress does not exceed the allowablc fatigue stiess range gwenin Table 1 0.3. l A for the appropriate type and locationof material gwen in Table 10.3.IB and shown in Fig-ure l 0.3. l C. For mcmbers with shear connectors providcdthroughout their entire length that also satisfy the provi-sions of Article 10.38.4.3, the range of stress may be com-puted using the composite section assuming the concrete deck to be fully cffective for both positive and negative moment. For unpainted weathering steel, A709, all grades, thevalues of allowable fatigue stress range, Table 10.3.IA, asinodified by footnote d, are valid only when the designand details are in accordance with the FHWA Technic ulAdvso on Uncoated Weatherng Steel in Structurvs, dated October 3, 1989.Main load carrying components subjected to tensile stresses that may be considered nonredundant load pathmembersthat is, where failure of a single element couldcause collapseshall be designed for the allowable stress ranges indicate1in Table 10.3.IA for Nonredundant LoadPath Structures. Examples of nonredundant load pathmembers are flange and web plates in one or two girderbridges, main one-element truss members, hanger plates,and caps at single or two-column bents.
10.3.2 Load Cycles 10.3.2.1 The number of cycles of maximum stressrange to be considered in the design shall be selected fromTable 10.3.2A unless traffic and loadometer surveys orother considerations indicate otherwise. The fatigue live load preferably shall not exceed HS 20 loading.
10.3.2.2 Allowable fatigue stress ranges shall applyto those Group Loadings that include live load or windload. 10.3.2.3 The number of cycles of stress range to beconsidered for wind loads in combination with dead loads, except for sti uctuies wlieie other considei ations indicata substantially diffeient number of cycles, shall b1011,000 cycles. 10.3.3 Charpy V-Notch Impact Requirements 10.3.3.1 Main load carrying member component subjected to tensile stress iequire supplemental impacproperties as described in the Material Specificatins.* 10.3.3.2 These impact requirements vary dependin on the type of steel, type of construction, welded or mechanically fastened, and the average minimum servictemperature to which the structure may be subjected. *'Table 10.3.3A contains the temperature zone designations 10.3.3.3 Components requiring inaiidatory impac properties shall be dcsignated on the drawings and the appropriate zone shall be designated in the contract documents. 10.3.4 Shear 10.3.4.1 When longitudinal beam or girder member in bridges designed for Case I roadways are investigatcfor over 2 million stress cycles produced by placing single trucl on the bridge (see footnote c of Tabl10.3.2A), the total shear force in the beam or gircler undethis single-ti ucl loading shall be limited to 0.58 F,Dt,CThe constant C, the ratio of the buckling shear stress to thshear yield stress is defined in Article 10.34.4.2 or Articl10.48.8.1. 10.4 EFFECTIVE LENGTH OF SPAN For the calculation ol stresses, span lengths shall be assumed as the distance between centers of bearings or othe points of support. *AASHTO Standard Spcc ifico ti ne fcr Trcinsportcition Matericils anMethods of Scimplii g and Te.stng. **The basis anct philosophy used to develop these requiieineiits tiegwen in a paper entitled The Development of AASH"IO FractuieToughness Requirements for Bridge Steels by John M. Barsoni, Februai y 1975, availahle roni the American Iron and 5tecl Institute, Wash-ington, D.C. 5/19/2018 AASHTO Standard Specifications 17th Edition 10/86 Category For For For For over(See Table 100,000 500,000 2,000,000 2,000,00010.3. lB) Cycles Cycles Cycles Cycles D 28 16E 22 13E' 16 9. 2F 15 12 Category For For For For over(See Table 100,000 500,000 2,000,000 2,000,00010.3. lB) Cycles Cycles Cycles Cycles HIGHWAY BRIDGES TABLE 10.3.IA Allowable Fatigue Stress Range Redundant Load Path Structures Allowable Range of Stress, F,, (ksi) A 63 (49) 37 (29) 24 (18) 24 (16)B 49 29 18 16B' 39 23 14. 5 12C 35.5 21 13 10 10 78 4.
55.8 2.69 8 Nonredundant Load Path Structures Allowable Range of Stress, F,, (ksi)b A 50 (39) 29 (23) 24 (16)' 24 (16)B 39 23 16 16B' 31 18 11 11C 28 16 10 9 12d 1D 22 13 8 5E 17 10 6 2.3E' 12 7 4 1.3F 12 9 7 6 Structure types with multi-load paths where a single fra cture in a member cannot lead to the collapse. For example, a simply supportedsingle span multi-beam bridge or a multi-element eye bar truss member has redundant load paths. The range of stress is defined as the algebraic difference between the maximum stress and the minimum stress. Tension stress isconsidered to have the opposite algebraic sign from compressionstress. For unpainted weathering steel, A 709, all grades, when used in conforma rice with the F-HWA Technical Advisor y on UncoatedWeathering Steel in Structures , dated October 3, 1989. d For transverse stiffener welds on girder webs or flanges. Partial length welded cover plates shall not be used on flanges morethan 0.8 inches thick for nonredundant load path structures. 10.5 DEPTH RATIOS 10.5.1 For beams or girders, the ratio of depth to lengthof span preferably should not be less than " 10.5.2 For composite girders, the ratio of the overall depth of girder (concrete slab plus steel girder) to thelength of span preferably should not be less than , andthe ratio of depth of steel girder alone to length of spanpreferably should not be less than 10.5.3 For trusses the ratio of depth to length of spanpreferably should nitbe less than i 10.5.4 For continuous span depth ratios the span lengthshall be considered as the distance between the dead load points of contraflexure. 111.5.5 The foregoing Requirements as they relate tobeam or girder bridget may be exceeded at the discretion of the dcsigner. '* 10.6 DEFLECTION 10.6.1 The term deflection as used herein shall bethe cleflection computed in accordance with the assump-tien madc for loading when computing the stress in themember. 10.6.2 Members having simple or continuous spanspreferably should be designed so that the deflection due toservice live load plus impact shall not exceed u of thespan, except on bridges in urban areas used in part bypedestrians whereon the ratio preferably shall not exceed?i. For checking deflection, the service live load prefer-ably shall not exceed HS 20 loading.
10.6.3 The deflection of cantilever arms due to servicelive load plus impact preferably should be limited to ?of the cantilever arre except for the case including pedes-tri:in use, where the ratio preferably should be ?
10.fi.4 When spans have cross-bracing or diaphragmssufficient in depth or stiength to ensure lateral distribu-tion of loads, the deflection may be computed for thestandard H or HS loading (M or MS) considering all beams or stringers as acting together and having equaldeflection. 10.6.5 The moment of inertia of the gross cross-sec-tional area shall be used for computing the deflections ofbeams and girdcrs. When the beam or girder is a part of acomposite member, the service live load may be consid-ered as acting upon the composite section.
10.6.6 The gross area of each truss member shallbe used in computing defleetions of trusses. If per-forated plates are used, the effective area shall be the net *For considerations to be taken into Account when exceeding theselimitations, iefercnce is wade to Bulletin No. 19, Criteiia forthe De-flection of Steel Bridget, availablc from the American kon anti SteelInstitute, Washington, D.C. 5/19/2018 AASHTO Standard Specifications 17th Edition 11/86 10.6.6 DIVISIIN IDESIGN 26 TABLE 10.3.1B Stress IllustrativeCategory Example Kind of (See Table (See FigureGeneral Condition Situacion Stress 10.3. IA) 10.3. IC) Plain Member Base metal with rolled or cleaned surface. Flame-cut edges T or Rev A 1,2with ANSI smoothness of 1,O00 or less. Built-Up Members Base rrietal and weld metal in members of built-up plates orshapes (without attachments) connected by continuous full T or Rev B 3,4,5,7 penetration groove welds (with backing bars removed) or bycontinuous fillet welds parallel to the direction of applicd stress. Base metal and weld metal in members of built-up plates or T or Rev B' 3,4,5,7shapes (without attachments) connected by continuous fullpenetration groove welds with backing bars not removed, orby continuous partial penetration groove welds parallel to thedirection of applied stress. Calculated flexural stress at the toe of transverse stiffener T or Rev 6welds on girder webs or flanges. Base metal at ends of partial length welded coverplates with - -b T or Rev B 22 Groove WeldedConnections Base metal at ends of partial length welded coverplatesnarrower than the flange having square or tapered ends, withor without welds across the ends, or wider than flange with welds across the ends: (a) Flange thickness 0. 8 in. T or Rev(b) Flange thickness V 0. 8 in. T or ReV Base metal at ends of partial length welded coverplates wider T or Revthan the flange without welds across the ends. Base metal and weld metal in or adjacent to full penetration T or Revgroove weld splices of rolled or welded sections having similarprofiles when welds are ground flush with grinding in thedirection of applied stress and weld soundness established by nondestructive inspection. Base metal and weld metal in or adjacent to full penetration T or Revgroove weld splices with 2 ft radius transitions in width, when welds are ground flush with grinding in the directionof applied stress and weld soundness established bynondestructive inspection.
Base metal and weld metal in or adjacent to full penetrationgroove weld splices at transitions in width or thickness, withwelds ground to provide slopes no steeper than 1 to 2?c, withgrinding in the direction of the applied stress, and weldsoundness established by nondestructive inspection: EE' 7 E B 8, 10 B (a) AASHTO M 270 Grades 100/100W (ASTM A 709)base metal T or Rev B' (b) Other base iiietals T or Rev B Base metal and weld metal in or adjacent to full penetration T or Revgroove weld splices, with or without transitions having slopes no greater than 1 to 2'Ze, when the reinforcement is notremoved and weld soundness is established by nondestructiveinspection. 11, 12 8, 10, 11,12 Groove WeldedAttachmentsLongitudinallyLoaded Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in thedirection of stress, is less than 2 in. Base metal adjacent to details attached by full or partialpenetration groove welds when the detail length, L, in thedirection of stress, is between 2 in. and 12 times the platethickness but less than 4 in. T or Rev T or Rev 6,15 D 5/19/2018 AASHTO Standard Specifications 17th Edition 12/86 262 HIGHWAY BRIDGES 10.6.6 TABLE 10.3.IB (Continued) Stress IllustrativeCategory Example Kind of (See Table (See FigureGeneral Condition Situation Stress 10.3. IA) 10.3. IC) Base metal adjacent to details attached by full or partialpenetration groove welds when the detail length, L, in the direction of stress, is greater than 12 times the plate thicknessor greater than 4 in.: (a) Detail thickness < 1.0 in. T or Rev E(b) Detail thickness 1.0 in. T or Rev E' Base metal adjacent to details attached by full or partialpenetration groove welds with a transition radius, R,regardless of the detail length: With the end welds ground smooth T or Rev 16(a) Transition radius fi 24 in.(b) 24 in. > Transition radius fi 6 in.(c) 6 in. > Jransition radius 2 in.(d) 2 in. > Transition radius fi 0 in. For all transition radii without end welds ground smooth.
T or Rev E 16 Groove welded Detail base metal attached by full penetration groove weldsAttachments with a transition radius, R, regardless of the detail length andTransversely with weld soundness transverse to the direction of stressLoaded established by nondestructive inspection: With equal plate thickness and reinforcement removed T or Rev 16(a) Transition radius 24 in.(b) 24 in. W Transition radius fi 6 in.(c) 6 in. > Transition radius 2 in.(d) 2 in. > Transition radius 0 in. BC D E With equal plate thickness and rcintorcement not removed T or Rev 16(a) Transition radius 6 in.(b) 6 in. > Transition radius 2 in.(c) 2 in. > Transition radius 0 in. DE With unequal plate thickness and reinforcement removed T or Rev 16(a) Transition radius 2 in. D(b) 2 in.
+ Transition radius 0 in.
E For all transition radii with unequal plate thickness and T or Rev E 16reinforcement not removed. Fillct Welded Base metal at details connected with transversely loadedConnections welds, with the welds perpendicular to the direction of stress: (a) Detail thickness 0. 5 in.
T or Rev C 14(b) Detail thickness > 0.
5 in.
T or Rev See Note Base metal at intermittent fillet welds. T or Rev E Shear stress onthroat of fillet welds. Shear F 9 Fillet Welded Base metal adjacent to details attached by fillet welds with T or Rev C IE, 17,18,20 Attachments length, L, in the direction of stress, is less than 2 in. andLongitudinally stud-type shear connectors.Loaded ,d Base metal adjacent to details attached by fillet welds with T or Rev Dlength, L, in the dection of stress, between 2 in. and 12 times the plate thickness but less than 4 in. Base metal adjacent to details attached by fillet welds withlength, L, in the direction of stress greater than 12 times the plate thickness or greater than 4 in.. 15,17 (a) Detail thickness < 1.0 in. (b) Detail thickness l .0 in. 1or RevT or Rev E 7,9,15,17E' 7,9,15 5/19/2018 AASHTO Standard Specifications 17th Edition 13/86 10.6.6 DIVISION IDESIGN General Condition Fillet WeldedAttachmentsTransversely Loadedwith the Weld inthe Direction ofPrincipal Stress"' TABLE 10.3.IB (Continued) Situacion Base metal adjacent to details attached by fillet welds with a transition radius, R, regardless of the detail length: With the end welds ground smooth (a) Transition radius 2 in.(b) 2 in.
> Transition radius 0 in. For all transition radii without the end welds ground smooth. Detail base metal attached by fillet welds with a transitionradius, R, regardless of the detail length (shear stress on thethroat of fillet welds governed by Category F): With the end welds ground smooth(a) Tlansition radius 2 in.(b) 2 in. W Transition radius 0 in. Stress IllustrativeCategory Example Kind of (See Table (See FigureStress 10.3. IA) 10.3. lC) T or Rev 16DE T or Rev E 16 T or Rev 16DE MechanicallyFastenedConnections Eyebar or Pin Plates For all transition radii without the end welds ground smooth.Base metal at gross section of high-strength bolted slipresistant connections, except axially loaded joints whichinduce out-of-plane bending in connecting materials. Base metal at net section of high-strength boltedbearing-type connections. Base metal at net section of riveted connections. Base metal at the net section of eyebar head, or pinplateBase metal in the shank of eyebars, or through the grosssection of pin plates with: T or Rev T or Rev T or Rev T or Rev T E 16 B D 21 E 23, 24 (a) rolled or smoothly ground surfaces T A 23, 24(b) llame-cut edges T B 23, 24 Tsignifies range in tensile stress only, Revsignifies a range of stress involving both tension and compression during a stress cycle.See Wattar, Albrecht and Sahli, Journal of structural Engineering, ASCF:, Vol. III, No. 6, June 1985, pp. 12351249. where S is equal to the allowable stress range for Category C gwen in Table 10.3. lA. This assumes no penetration at the weld root. 'Gusset plates attached to girder flange surfaces with only transverse fillet welds are prohibited.
volume divided by the length from center to center ofperforations. 10.6.7 The foregoing requirements as they relate tobeam or girder bridges may be exceeded at the cliscretionof the designer. * *For considerations to be takeii inte account when exceeding theseliinitations, reference is wade to Bulletiii No. 19, Criteria for the De-flection of Steel Bridges, available fiom the American Iron and SteelInstitute, Washington, D.C. 10.7 LIMITING LENGTHS OF MEMBERS 10.7.1 For compression members, the slendernesratio, KL/r, shall not exceed 120 or main members, othose in which the majoi stresses result fi om dead or liveload, o both; and shall not excecd 140 for secondarymembers, or those whose primary purpose is to brace thestructure against lateral or longitudinal force, or to bracor reduce the unbraced length of other members, main osecondary. 5/19/2018 AASHTO Standard Specifications 17th Edition 14/86 10.7.1264 HIGHWAY BRIDGES 1 fin b 9 At End of Weld, Has No Length 10 End of Weld FIGURE 10.3.TC Illusti-ative Examples 5/19/2018 AASHTO Standard Specifications 17th Edition 15/86 10.7.2 DIVISION IDESIGN 26 TABLE 10.3.2A Stress Cycles Main (Longitudinal) Load Carrying Members TABLE 10.3.3A Tempei-ature Zone Designations forCharpy V-Notch Impact Requirements pe of Road Case ADTTTruck LoadingLane Loading MinimumService Temperature TemperatureZone Designation Freeways, Expressways, I 2,500 or 2,000,000 500,000 0F and above 1Major Highways, and more 1Fto 30 F 2Streets 3lFto 60 F 3 Fre ewa s , E x re sswa s , II less than 500,000 100,000Major Highways, and 2,500 length between panel point intersections or centers o braced points or centers of end conncctions; for secondary members, the length between the ccnters of thend connections of sitch members or centers of bracepoints. 10.7.5 For tension members, except rods, eyebars, cables, and plates, the ratio of unbraced length to radius o Other Highways and IIIStreets not included inCase I or II Transverse Members and Details Subjected to Wheel Loads Freeways, Expressways, I 2,500 or overMajor Highways, and more 2,000,000Streets Freeways, Expressways, II less than 2,000,000Major Highways, and 2,500Streets gyration shall not exceed 200 for main members, shall noexceed 240 for bracing iiiembers. and shall not excee140 for main members subject to a reversal of stress. 10.8 MINIMUM THICKNESS OF METALOther Highways and IIIStreets Average Daily Truck Traffic (one direction). 500,00010.8.1 Structural steel (including bracing, cross framesand all types of gusset plates), except for webs of certain Longitudinal members should also be checked for truck loading.Members shall also be investigated for over 2 million stress cycles produced by placing a single truck on the bridge distributed to the girders as designated in Article 3.23.2 for one traffic lane loading. The shear in steel girder webs shall not exceed 0.58 F,DC for thissingle truck loading. 10.7.2 In determining the radius of gyration, r, for the purpose of applying the limitations of the KL/r ratio, thearea of any portion of a member may be neglected pro- vided that the strength of the member as calculated with- out using the area thus neglected and the strength of themember as computed for the entirc section with the KL/riatio applicable thereto, both equal or exceed the com-puted total force that the member must sustain. 10.7.3 The radius of gyration and the effective area forcarrying stress of a member containing perforated coverplates shall be computed for a transverse section throughthe maximum width of perforation. When perforations arestaggered in opposite cover plates, the cross-sectionalarea of the member shall be considered the same as for asection having perforations in the same transverse plane. 10.7.4 Actual unbraced length, L, shall be assumed asfollows: For the top chords of half-through trusses, the lengthbetween panel points laterally supperted as indicatedunder Article 10.16.12; for other main members, the rolled shapes, closed ribs in orthotropic decks, fillers, andin railings, shall be not less than ? inch in thickness.
Thweb thiclness of rolled beams or channels shall not bless than 0.23 inches. The thickness of closed ribs in othotropic decks shall not be less than inch. 10.8.2 Where the metal will be exposed to marked cor rosive influences, it shall be increased in thickness or specially protected against corrosion. 10.8.3 It should be noted that there are other provisionin this section pertaining to thickness for fillers, segmentof compression members, gusset plates, etc.
As stateabove, fillers need not be inch minimum. 10.8.4 For compression members, refer to Trusses(Article 10.16). 10.8.5 For stiffeners and other plates, refer to PlatGirders (Article 10.34). 10.8.6 For stiffeners and outstanding legs of angles, etcrefer to Article 10.10. 10.9 EFFECTIVE AREA OF ANGLES ANDTEE SECTIONS IN TENSION 10.9.1 The effective area of a single angle tension member, a tee section tension member, or each angle of a ilou 5/19/2018 AASHTO Standard Specifications 17th Edition 16/86 10.9.1266 HIGHWAY BRIDGES ble angle tension member in which the shapes are con-nected back to back on the same side of a gusset plate shallbe assumed as the net area of the connected leg or flangeplus one-half of the area of the outstanding leg. 10.9.2 If a double angle or tee section tension memberis connected with the angles or flanges back to back on op- posite sides of a gusset plate, the full net area of the shapcsshall be considered effective. 10.9.3 When angles connect to separate gusset plates, as in the case of a double-webbed truss, and the angles areconnected by stay plates located as near the gusset as prac- ticable, or by other adequate means, the full net area of the angles shall be considered effective. If the angles are notso connected, only 809r of the net areas shall be consid-ered effective. 10.9.4 Lug angles may be considered as effective in transmitting stress, provided they are connected with atleast one-third more fasteners than required by the stressto be carried by the lug angle. 10.10 OUTSTANDING LEGS OF ANGLES The widths of outstanding legs of angles in compres-sion (except where reinforced by plates) shall not exceedthe following: In main members carrying axial stress, 12 times thethickness.In bracing and other secondary members, 16 times thethickness. For other limitations, see Article 10.35.2. 10.11 EXPANSION AND CONTRACTION In all bridges, provisions shallbe wade in the design to resist thermal stresses induced, or means shall be providedfor movement caused by temperature changes. Provisionsshall be wade for changes in length of span resulting fromlive load stresses.
In spans more than 300 feet long, al- lowance shall be wade for expansion and contraction inthe floor. The expansion end shall be secured against lat-eral movement.
10.12 FLEXURAL MEMBERS Flexural members shall be designed using the elasticsection modulus except when utilizing compact sections under Strength Design as specified in Articles 10.48.1,10.50. 1.1, and 10.50.2.1. When computing the strength ofa flexural member at a section with holes in the tensionflange, an effective flange area, A,, specified by Equation(10-4g) shall be used for that flange in computing the elas-tic section properties. The diameter of the holes shall betaken as specified in Article 10.16.14.6. In the case of the strength design method, the strength of compact sectionswith holes in the tension flange shall not be taken greaterthan the moment capacity at first yield.
10.13 COVER PLATES 10.13.1 The length of any cover plate added to a rolledbeam shall be not less than (2d+ 3) feet, where (d) is thedepth of the beam in feet. 10.13.2 Partial length welded cover plates shall not be used on flanges more than 0.8 inches thick for nonredun-dant load path structures subjected to repetitive loadingsthat produce tension or reversal of stress in the member.
10.13.3 The maximum thickness of a single cover plateon a flange shall not be greater than two times the thick-ness of the flange to which the cover plate is attached. Thetotal thickness of all cover plates should not be greaterthan 2?
times the flange thickness. 10.13.4 Any partial length welded cover plate shall ex- tend beyond the theoretical end by the terminal distance,and it shall extend to a section where the stress range in the beam flange is equal to the allowable fatigue stressrange for base metal adjacent to or connected by fillet welds. The theoretical end of the cover plate, when usingservice load design methods, is the section at which thestress in the flange without that cover plate equals the al-lowable service load stress, exclusive of fatigue consider-ations. When using strength design methods, the theoret-ical end of the covei plate is the section at which the flange strength without that cover plate equals the requiredstrength for the design loads, exclusive of fatigue require-ments. The terminal distance is two times the nominalcover plate width for cover plates not welded across their ends, and lu times for cover plates welded across theirends. The width at ends of tapered cover plates shall benot less than 3 inches.
The weld connecting the coverplate to the flange in its terminal distance shall be contin-uous and of sufficient size to develop a total stress of notless than the computed stress in the cover plate at its the-oretical end. All welds connecting cover plates to beamflanges shall be continuous and shall not be smaller thanthe minimum size permitted by Article 10.23.2. 5/19/2018 AASHTO Standard Specifications 17th Edition 17/86 DIVISION IDESIGN 10.13.5 Any partial length end-bolted cover plate shallextend beyond the theoretical end by a terminal distanceequal io the length of the end-bolted portion, and the coverplate shall extend to a section where the stress range in thebeam flange is equal to the allowable fatigue stress rangefor base metal at ends of partial length welded cover plates R= R 00b (10 - 2 with high-strength bolted, slip-critical end connections(Table 10.3.IB). Beams with end-bolted cover plates shallbe fabricated in the following sequence: drill holes; cleanfaying surfaces; install bolts; weld. The theoretical end ofthe end-bolted cover plate is determined in the same man-ner as that of a welded cover plate, as is specified in Arti-cte 10.13.4. The bolts in the slip-critical connections ofthe cover plate ends to the flange, shall be of sufficientnumbers to develop a total force of not less than the com-puted force in the cover plate at the theoretical end. Theslip resistance of the end-bolted connection shall be de-termined in accordance with Article 10.32.3.2 for serviceload design, and Article 10.56.1.4 for load factor design.The longitudinal welds connecting the cover plate to thebeam flange shall be continuous and stop a distance equalto one bolt spacing before the first row of bolts in the end-bolted portion. 10.14 CAMBER Girders should be cambered to compensate for dead load deflections and vertical curvature required by profilegrade. 10.15 HEAT-CURVED ROLLED BEAMS ANDWELDED PI=ATE GIRDERS 10.15.1 Scope This section pertains to rolled beams and welded I-sec-tion plate girders heat-curved to obtain a horizontal cur-vature.
Steels that are manufactured to a specified mini-mum yield point greater than 50,000 psi, except for GradeHPS70W steel, shall not be heat-cuived. 10.15.2 Minimum Radius of Curvature 10.15.2.1 For heat-curved beams and girders, the horizontal radius of curvature measured to the center lineof the girder web shall not be less than 150 feet and shallnot be less than the larger of the values calculated (at anyand all cross sections throughout the length of thc girder)from the following two equations: In these equations, F, is the specified minimum yieldpoint in kips per square inch of steel in the girder web, iis the ratio of the total cross-sectional area to the ci osssectional area of both flanges, b is the widest flange widthin inches, D is the clear distance between flanges ininches, tq is the web thiclness in inches, and R is the radius in inches. 10.15.2.2 In addition to the above requircments, theradius shall not be less than 1,000 feet when the flangethickness exceeds 3 inches or the flange width exceeds30 inches. 10.15.3 Camber To compensate for possible loss of camber of heat-curved girders in service as residual stresses dissipate, theamount of camber in inches, at any section along thelength L of the girder shall be equal to: _ DL _ 0.02 L' 1, 000 R EY, 850 Ay = 0 for iadii greater than l, 000 where ADis the camber in inches at any point along thelength L calculated by usual procedures to compensate fordeflection due to dead loads or any other specified loads is the maximum value of A DL in inches withiii thelength L; E is the modulus of elasticity in ksi; F , is thespecified minimum yield point in ksi of the girder flangeY, is the distance from the neutral axis to the extremeouter fiber in inches (maximum distance for nonsymine-trical sections); R is the radius of curvatura in feet; and L is the span length for simple spans or for continuousspans, the distance between a simple end support and thedead load contraflexure point, or the distance betweenpoints of dead load contraflexure. (L is measured ininches.) Camber loss bctween dead load contraflexurepoints adjacent to piers is small and may be neglected. Note: Part of the camber loss is attributable to construc-on loads and will occur during construction of the 5/19/2018 AASHTO Standard Specifications 17th Edition 18/86 268 HIGHWAY BRIDGES bridge; total camber loss will be complete afterseveral months of in-service loads. Therefore, aportion of the camber increase (approximately509r) should be included in the bridge profile.Camber losses of this nature (but generally smallerin magnitude) ire also known to occur in straightbcums and girders. 10.16 TRUSSES 10.16.1 Cieneral 10.16.1.1 Coinpcnentparts of individual truss mem-bers may be connected by welds, rivets, or high-strengthbolts. 10.16.1.2 Preference sheuld be gwen to trusses withsingle intersection web systems. Members shall be sym-meti ical about the central plane of the truss. 10.TG.1.3 Trusses preferably shall have inclined endposts. Latei ally unsupported hip joints shall be avoided. 10.1fi.1.4 iVain trusses shall be spaced a sufficientdistance apart, center to center, to be secure against overturiiiiig by the assumed lateral forces. 10.16.1.5 For the calciilation of stresses, effectivedepths shall be assumed as follows: Riveted and bolted trusscs, distance between centers ofgravity of the chords. Pin-connected trusses, distance between centers ofchord pins. 10.16.2 Truss Members 10.16.2.1 Chord and web truss members shall usu-ally be wade in the following shapes: Hsections, wade with two side segments (composedof angles or plates) with solid web, perforated web, orweb of stay plates and lacing.Channel sections, wade with two angle segmento, withsolid web, per forated web, or web of stay plates andlacing.Single Box sections, wade with side channels, beams,angles, and plates or side segmento of plates only, con-nected top and bottom with pei foi ated plates ni stayplates and lacing. Single Box sections, wade with side channels, beams,angles and plates only, connected at top with solid cover plates and at the bottom with perforated plates orstay plates and lacing.Double Box sections, wade with side channels, beams,angles and plates or side segments of plates only, con- nected with a conventional solid web, together with tpand bottom perforated cover plates or stay plates andlacing. 10.16.2.2 If the shape of the truss permits, compres-sion chords shall be continuoiis.
10.16.2.3 In chords composed of angles in channel-shaped member s, the vertical legs of the angles pr eferablyshall extend downward. 10.16.2.4 If web members are subject to reversal ofstiess, their end connections shall not be pinned. Coiintei s preferably shall be rigid. Adjustable counters, if used,shall have open turnbuckles, and in the design of these members an itllowance of 10,000 pounds per square inch shall be wade for initial stress. Only one set of diagonaJ sin any panel shall be adjustable. Sleeve nuts and loop b:u s shall not be used. 10.16.3 Secondary Stresses The design and details shall be such that secondarystresses will be as small in praeticable. Secondary sti esscsdue to truss distortion or floor beam deflection usuull yneed not be considei ed in any member, the width ofwhich, measured parallel to the plane of distortion, is less than one-tenth of its length. If the secondary stress ex-ceeds 4,0011 pounds per square inch for tension membcrsand 3,000 for compression members, the excess shall betreated es a primary stress. Stresses due lo the flexuraldead load moment of the member shall be considered asadditional secondary stress. 10.1ti.4 Diaphragms 10.16.4.1 There shall be diaphragms in the trusses atthe end connections of floor beams. 10.1ti.4.2 The gusset plates engaging the pedestal pinat the end of the truss shall be connected by a diaphragm.Similarly, the webs of the pedestal shall, if practicable, beconnectelby a diapliragm. 10.16.4.3 There shall be a diaphragm between gussetplates engaging main members if the end tie plate is 4 feeto inoie from the point of intei section ot the members.
5/19/2018 AASHTO Standard Specifications 17th Edition 19/86 DIVISION IDESIGN 10.16.5 Camber The length of the truss members shall be such that thecamber will be equal to er greater than the deflection pro-duced by the dead load.
10.16.f Working Lines and Gravity Axes 10.16.6.1 Main members shall be proportioned sothat their gravity axes will be as nearly as practicable inthe center of the section. 10.16.6.2 In compression members of unsymmetri-cal section, such as chord sections formed of side seg-ments and a covei plate, the gravity axis of the sectionshall coincide as nearly as practicable with the workingline, except that eccentricity may be introduced to coun-teiact dead load bending. In twoangle bottom chord or di-agonal members, the working line may be taken as the gage line nearest the baclof the angle or at the center ofgravity for welded trusses. 10.16.7 Portal and Sway Bracing 10.16.7.1 Through truss spans shall have portal brac-ing, preferably, of the two-plane or box type, rigidly con- nected to the end post and the top chord flanges, and asdeep as the clearance will allow. If a single plane portal isu sed, it shall be located, prefeiably, in the central hans-verse plane of the end posts, with diaphragms between thewebs of the posts to provide for a distribution of the por tal sti esses. The portal bracing shall be designed to takethe full end ieaction of the top chord latci al system, andthe end posts shall be designed to transfer this reaction to the ngs. IO.1ti.7.2 Through truss spans shall have sway brac-ing 5 feet or more deep at each intermediate panel point.Top lateral struts shall be at least as deep as the top chord. 10.16.7.3 Deck truss spans shall ha.le sway bracing in the plane of the end posts and at all intermediate p:inelpoints. This bracing shall extend the full depth of the trusscs below the Poor system. The end sway liracing shallbe proportioned to carry the entire upper lateral stress to the supports through the end posts of the truss.
10.16.8 Perforated Cover Plates When ierfoiatcd