COLD-FORMED STEAL SEISMIC DESIGN RECOMMENDATIONS - VOL 3 OF 3

Main Category:	Civil Engineering
Sub Category:	Structural
Course #:	STR-119
Course Content:	148 pgs
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Exam Preview:

1. Seismic design with cold-formed steel has two problems that are inherent with the material itself: (1) light gage thickness and (2) variability in material strength.
   a. True
   b. False
2. Table 11-1. Seismic importance factors based on ASCE/SEI 7-10 risk category, what is the seismic importance factor for a class 3 risk category?
   a. 1.00
   b. 1.15
   c. 1.25
   d. 1.50
3. Diaphragms are considered flexible if the maximum lateral deformation of the diaphragm exceeds ____ the average story drift of the associated story (ASCE/SEI 7-10, section 12.3.1.3).
   a. 1.5X
   b. 2X
   c. 3X
   d. 4X
4. Since the straps must be fabricated from ASTM A1003/A1003M, Type L steel (ASTM 2013b), the straps will never be rerolled, so that the strap stress will never reach maximum ultimate stress, making the maximum yield strength a reasonable upper limit.
   a. True
   b. False

5. Using Table 11-10. Maximum angle thickness based on column-to-anchor fillet weld thickness, what is the maximum angle thickness, in inches, for a weld that is ¼ in (6mm) thick?
   a. 5/16 in
   b. 1/2 in
   c. 5/8 in
   d. 3/4 in
6. According to the reference material, the minimum distance between the centers of screws shall not be less than __ times their nominal diameter (AISI 2007a, section E4.1).
   a. 2
   b. 3
   c. 4
   d. 5
7. According to the reference material, the columns may be hollow structural sections (HSS) or can be built up from studs.
   a. True
   b. False
8. According to the reference material, welded design follows AISI guidance (AISI 2007a, section E2 "Welded Connections"), which covers connections of members in which the thick-ness of the thinnest member is ___ in. or less.
   a. 1/16
   b. 1/8
   c. 3/16
   d. 1/4
9. According to the reference material, the column anchor design provisions presented later in these recommendations will create a moment connection. The primary purpose of the anchor design is to resist shear and uplift forces.
   a. True
   b. False
10. Section 11.13.2 discusses the screwed fastener connection design. When using equation 11-76, what is the value for F_{cu} when using Garde 33 steel?
    a. 45 ksi
    b. 65 ksi
    c. 58 ksi
    d. 72 ksi

Contents

11 Seismic Design Recommendations for Shear Walls (Diagonal Strap Systems)

The seismic design recommendations provided here may be used for all occupancy categories as defined in ASCE/SEI 7-10, "Minimum Design Loads for Buildings and Other Structures" (ASCE/SEI 7-10, Table 1.5-1), subject to the limitations presented in this chapter.

Seismic design with cold-formed steel has two problems that are inherent with the material itself: (1) light gage thickness and (2) variability in material strength. The objective of seismic design recommendations is to ensure ductile building system performance in the large seismic event and elastic response in the small seismic event or wind loading. Ductile building performance requires that selected ductile components yield but continue to carry loads and absorb energy through significant plastic response. At the same time, potentially brittle failure modes such as column buckling or connection failure must be prevented. The design challenge for cold-formed steel is to ensure that building components, and in particular shear panel components, be proportioned relative to each other and detailed so that the ductile response is ensured. In these design recommendations, ductile response is accomplished by ensuring that the diagonal straps yield and respond plastically through significant displacement, without risk of column buckling or damage to brittle connections.

Seismic design recommendations are provided on three levels:

1. Tabular data for prototype shear panels in terms of the maximum story shear and maximum and minimum gravity load as defined in this chapter. The shear panel configurations and data are provided in Appendix C.
2. Detailed seismic design recommendations using shear panels with diagonal straps as the primary lateral-load-resisting element. The recommendations are documented in this chapter, and an example application of the recommendations is given in Chapter 12[5].

[5] The spreadsheet program used in the example problem is available through the Network for Earthquake Engineering Simulation (NEES) Data Repository (http://nees.org/warehouse) as an engineering support tool for shear panel design.

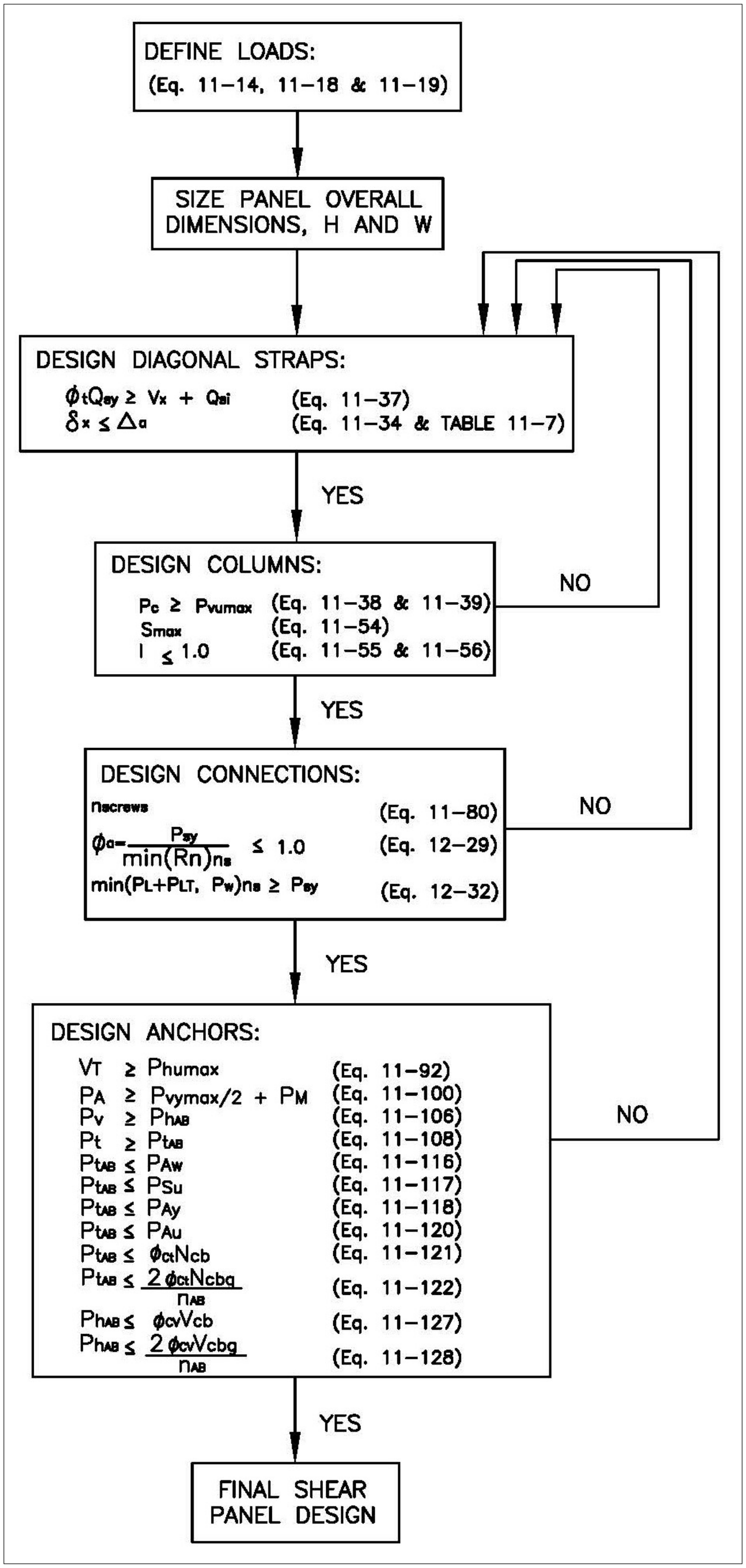
3. A test procedure and acceptance criteria for other shear panel configurations, which are provided in Appendix D.

The design recommendations presented here are directly related to ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*. The technical basis for unique CFS seismic design recommendations is the topic of Part I of this report. These recommendations also incorporate material from the references listed below:

- *AISI Manual: Cold-Formed Steel Design* (AISI 2008)
- "Cold-Formed Steel Light-Frame Construction" (ICC 2011)
- *Recommended Seismic Provisions for New Buildings and Other Structures* (NEHRP 2009)
- "Standard Specification for Steel Sheet, Carbon, Metallic- and Non-metallic-Coated for Cold-Formed Framing Members" (ASTM 2013b)
- *Steel Construction Manual* (AISC 2011)
- "Specification for Structural Steel Buildings" (AISC 2010a)
- "Seismic Provisions for Structural Steel Buildings" (AISC 2010b)
- *Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D* (ACI 2011b)

Figure 11-1 is a process flowchart for seismic design using cold-formed steel shear walls. Chapter 12 explains an example problem showing the application of the recommendations presented in this chapter.

Figure 11-1. Flowchart for cold-formed steel shear panel seismic design.



11.1 Risk category

The risk category of a building shall be determined in accordance with Table 1.5-1 of ASCE/SEI 7-10. The risk categories are based on the risk to human life, health, and welfare associated with building damage or failure by nature of their occupancy or use (ASCE/SEI 7-10, section 1.5.1). The categories vary from least critical (category I) to the most critical (category IV).

11.2 Importance factors

Seismic importance factors, I_e, are defined for buildings based on their risk category in ASCE/SEI 7-10, Table 1.5-2). These importance factors are summarized in Table 11-1 for seismic loads only.

Table 11-1. Seismic importance factors based on ASCE/SEI 7-10 risk category.

Risk Category (ASCE/SEI 7-10, Table 1.5-1)	Seismic Importance Factor, I_e
I	1.00
II	1.00
III	1.25
IV	1.50

11.3 Defining ground motion

Seismic ground motions shall be defined according to ASCE/SEI 7-10, section 11. This paragraph defines spectral response accelerations, S_s and S_1, for 0.2 and 1 second, respectively, in Figures 22-1 through 22-6 on U.S. Geological Survey (USGS) website http://earthquake.usgs.gov/hazards/designmaps/. Site classifications (A through F) shall be determined based on soil properties as defined in ASCE/SEI 7-10, Chapter 20. But if soil properties are not known, Site Class D shall be used. From the site classifications, values of site coefficients (F_a and F_v) are determined for the mapped spectral response acceleration values in ASCE/SEI 7-10, Table 11.4-1 and Table 11.4-2. These tables are reproduced below as Table 11-2 and Table 11-3.

Table 11-2. Values of F_a as a function of site class and mapped 0.2-second period maximum considered earthquake spectral response acceleration.[6]

Site Class	Mapped Risk-Targeted Maximum Considered Response (MCER) Spectral Response Acceleration Parameter at Short Period				
	SS ≤ 0.25	SS = 0.50	SS = 0.75	SS = 1.00	SS ≥ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	a[7]	a	a	A	A

Table 11-3. Values of F_v as a function of site class and mapped 1-second period maximum considered earthquake spectral response acceleration.[8]

Site Class	Mapped Risk-Targeted Maximum Considered Response (MCER) Spectral Response Acceleration Parameter at 1-s Period				
	S1 ≤ 0.1	S1 = 0.2	S1 = 0.3	S1 = 0.4	S1 ≥ 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	a[9]	a	a	a	a

The maximum considered earthquake (MCE_R) spectral response acceleration for short periods (S_{MS}) and at 1 second (S_{M1}), adjusted for site class effects, are calculated as follows (ASCE/SEI 7-10, Eq 11.4-1 and 11.4-2):

$$S_{ms} = F_a S_S \qquad \text{(Eq 11-1)}$$

and

$$S_{M1} = F_v S_1 \qquad \text{(Eq 11-2)}$$

[6] Use straight-line interpolation for intermediate values of S_S.

[7] a indicates site-specific ground motion procedure set forth in ASCE/SEI 7-10, Chapter 21, are to be used.

[8] Use straight-line interpolation for intermediate values of S_1.

[9] a here has the same meaning as for Table 11-2.

These values define the elastic spectra and are reduced to define design earthquake spectral response acceleration at short periods, S_{DS}, and at 1-second period, S_{D1}, as follows (ASCE/SEI 7-10, Eq 11.4-3 and 11.4-4):

$$S_{DS} = \frac{2}{3} S_{MS} \qquad \text{(Eq 11-3)}$$

and

$$S_{D1} = \frac{2}{3} S_{M1} \qquad \text{(Eq 11-4)}$$

From these terms, a design response spectrum is developed (ASCE/SEI 7-10, section 11.4.5 and Figure 11-4-1). (See Figure 12-1 for an example of a response spectrum plot for building in the design example.) For the natural period of the structure, T, this spectrum defines values of effective acceleration. The three regions of this spectrum are defined as follows:

For periods less than or equal to T_o, the design spectral response acceleration, S_a, shall be (ASCE/SEI 7-10, Eq 11.4-5):

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) \qquad \text{(Eq 11-5)}$$

For periods greater than or equal to T_o and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken as equal to S_{DS}.

For periods greater than T_S and less than or equal to T_L, the design spectral response acceleration, S_a, shall be (ASCE/SEI 7-10, Eq 11.4-6):

$$S_a = \frac{S_{D1}}{T} \qquad \text{(Eq 11-6)}$$

For periods greater than T_L, S_a shall be (ASCE/SEI 7-10, Eq 11.4-7):

$$S_a = \frac{S_{D1}T_L}{T^2} \qquad \text{(Eq 11-7)}$$

where:

T = the fundamental period of the structure in seconds,
T_o = $0.2S_{D1}/S_{DS}$, and
T_S = S_{D1}/S_{DS}.

T_L = long-period transition period, in seconds, shown in ASCE/SEI 7-10, Figure 22-12 through Figure 22-16.

11.4 Seismic design category

Each structure shall be assigned a seismic design category based on its risk category and design earthquake spectral response accelerations, S_{DS} and S_{D1}, as indicated in Table 11-4 and Table 11-5 (reproduced from ASCE/SEI 7-10, Tables 11.6-1 and 11.6-2).

Table 11-4. Seismic design category based on short period response acceleration parameter.

Value of S_{DS}	Risk Category	
	I or II or III	IV
$S_{DS} < 0.167g$	A	A
$0.167g \le S_{DS} < 0.33g$	B	C
$0.33g \le S_{DS} < 0.50g$	C	D
$0.50g \le S_{DS}$	D	D

Table 11-5. Seismic design category based on 1-second-period response acceleration parameter.

Value of S_{DS}	Risk Category	
	I or II or III	IV
$S_{D1} < 0.067g$	A	A
$0.067g \le S_{D1} < 0.133g$	B	C
$0.133g \le S_{D1} < 0.20g$	C	D
$0.20g \le S_{D1}$	D	D

11.5 Structural design criteria

The basic lateral and vertical seismic-force-resisting systems considered here are diagonal-strap configurations (Panels A3, C1, and D2) shown in Appendix B. These are considered bearing-wall systems. The format of Table 12.2-1 (ASCE/SEI 7-10) is used in Table 11-6 to present the response modification coefficient, R, and deflection amplification factor, C_d. These values are used to calculate the base shear and design story drift. The overstrength factor, Ω_o, used in ASCE/SEI 7-10 is not included here because shear panel overstrength is accounted for by $\Omega_o Q_E$ (in Equation 11-

17). This is the maximum lateral capacity of the shear panel based on the maximum estimated ultimate stress of the panel diagonal straps.

Table 11-6. Design coefficients and factors for seismic-force-resisting systems.

Seismic-Force-Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d	Structural System Limitations Including Structural Height, h_n, (ft) Limits[10]				
			A&B	C	D	E	F
Bearing Wall System							
Light-gage cold-formed steel wall systems using flat strap bracing	4	3.5	NL	NL	65	65	65

The response modification coefficient, R, in the direction under consideration at any story shall not exceed the lowest value for the seismic-force-resisting system in the same direction considered above that story, excluding penthouses. Other structural systems (dual systems) may be used in combination with these cold-formed steel panels, but then the smallest R value for all systems in the direction under consideration must be used for determining the loads applied to the entire structure in that direction (ASCE/SEI 7-10, section 12.2.3), and the design shall comply with other requirements of ASCE/SEI 7-10, section 12.2.3. Another structural system may be used in the orthogonal direction with different R values, and the lowest R value for that direction only shall be used in determining loads in that orthogonal direction (ASCE/SEI 7-10, section 12.2.2).

11.6 Structural configuration and redundancy

ASCE/SEI 7-10 presents recommendations on diaphragm flexibility, configuration irregularities, and redundancy (ASCE/SEI 7-10, section 12.3). Diaphragms are considered flexible if the maximum lateral deformation of the diaphragm exceeds twice the average story drift of the associated story (ASCE/SEI 7-10, section 12.3.1.3).

A redundancy factor, ρ, shall be defined for all structures in each of the two orthogonal directions based on the extent of structural redundancy in the lateral-force-resisting system. For structures in seismic design categories B and C, the value for ρ shall be taken as 1.0. For structures in catego-

[10] NL = not limited. For metric units, use 20 m for 65 ft. Heights are measured from the base of the structure, which is the level at which the horizontal seismic ground motions are considered to be imparted to the structure.

ries D, E, and F, values for ρ shall be taken as 1.3 unless one of the following conditions is met, in which case ρ shall be taken as 1.0 (ASCE/SEI 7-10, section 12.3.4.2):

1. Each story resists more than 35% of the base shear in the direction of interest and the removal of an individual strap or connection thereto would not result in more than a 33% reduction in story strength, nor would the removal result in an extreme torsional irregularity Type 1b as defined in ASCE/SEI 7-10 (see ASCE/SEI 7-10, Table 12.3-3).
2. The structure is regular in plan at all levels and the seismic-force-resisting systems consist of at least two bays of seismic-force-resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35% of the base shear.

11.7 Load combinations

Consideration of combinations of loads in the two orthogonal directions is not needed.

The effects of gravity loads and seismic forces shall be combined in accordance with the factored load combinations as indicated below (ASCE/SEI 7-10, section 2.3.2).

$$1.2D + 1.0E + L + 0.2S \qquad \text{(Eq 11-8)}$$

and

$$0.9D + 1.0E \qquad \text{(Eq 11-9)}$$

where

D = the dead load
E = the earthquake load
L = the live load; the load factor on L in Equation 11-8 shall equal 0.5 for all occupancies in which L_o in ASCE/SEI 7-10 (ASCE/SEI 7-10, Table 4-1) is less than or equal to 100 psf (4.79 kN/m^2), with the exception of garages or areas occupied as places of public assembly
S = the snow load, which shall be taken as either the flat roof snow load (p_f) or the sloped roof snow load (p_s).

The earthquake load or seismic load effect for use in Equation 11-8 shall be determined in accordance with Equation 11-10 (ASCE/SEI 7-10, section 12.4.2):

$$E = E_h + E_v \qquad \text{(Eq 11-10)}$$

The seismic load effect for use in Equation 11-9 shall be determined in accordance with Equation 11-11 (ASCE/SEI 7-10, section 12.4.2):

$$E = E_h - E_v \qquad \text{(Eq 11-11)}$$

The effect of horizontal seismic loads, E_h, shall be defined as follows (ASCE/SEI 7-10, section 12.4.2.1):

$$E_h = \rho Q_E \qquad \text{(Eq 11-12)}$$

The effect of vertical seismic loads, E_v, shall be defined as follows (ASCE/SEI 7-10, section 12.4.2.2):

$$E_v = 0.2 S_{DS} D \qquad \text{(Eq 11-13)}$$

where

ρQ_E = the maximum horizontal force that could be resisted by the bracing
ρ = the redundancy factor
Q_E = the effect of horizontal seismic forces
$0.2 S_{DS} D$ = the vertical spectral acceleration effect of the seismic load
S_{DS} = the design spectral response acceleration at short periods
D = the effect of dead load.

The effects of gravity load (dead, live, and snow load) and seismic forces shall be combined as follows when the effect of gravity and vertical seismic loads are additive, by combining Equations 11-8, 11-10, 11-12, and 11-13 (ASCE/SEI 7-10, section 12.4.2.3):

$$(1.2 + 0.2 S_{DS}) D + \rho Q_E + L + 0.2 S \qquad \text{(Eq 11-14)}$$

The effects of gravity load and seismic forces shall be combined as follows when the effect of gravity and seismic loads counteract each other, by combining Equations 11-9, 11-11, 11-12, and 11-13 (ASCE/SEI 7-10, sections 2.2, 2.3.2, and 12.4.2.3):

$$(0.9 - 0.2S_{DS})D + \rho Q_E + 1.6H \qquad \text{(Eq 11-15)}$$

The load factor on *H* shall be set equal to zero in Equation 11-15 if the structural action due to *H* counteracts that due to seismic loading. The term *H* in Equation 11-15 is the load due to lateral earth pressure, ground water pressure, or pressure of bulk materials when these pressures add to the effects of horizontal earthquake forces (ASCE/SEI 7-10, sections 2.2, 2.3.2, and 12.4.2.3).

For both expressions in Equations 11-14 and 11-15, the total horizontal force is ρQ_E. This force alone defines the total lateral load that must be resisted by the shear panel diagonal straps, and these elements should be sized based on this force.

The effect of horizontal seismic loads, E_{mh}, shall be defined as follows to account for diagonal strap overstrength:

$$E_{mh} = \Omega_o Q_E \qquad \text{(Eq 11-16)}$$

where

Ω_o = the system overstrength.

The term $\Omega_o Q_E$ calculated in Equation 11-16 (ASCE/SEI 7-10, section 12.4.3.1) need not exceed the maximum force that can be developed in the diagonal straps based on the maximum estimated ultimate strength of these elements. This is expressed as follows:

$$\Omega_0 Q_E \le Q_u = F_{sumax} n_s b_s t_s \frac{W}{\sqrt{H^2+W^2}} \qquad \text{(Eq 11-17)}$$

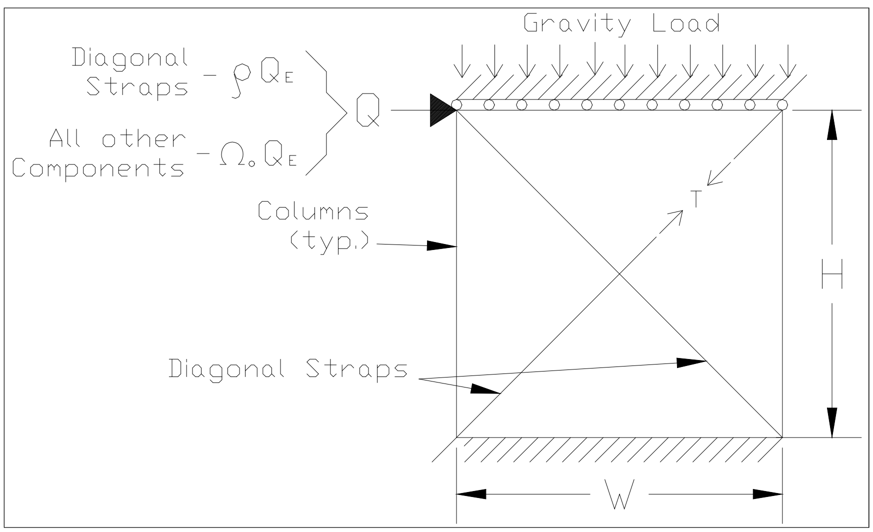
where

F_{sumax} = the maximum ultimate stress of the diagonal straps, which equals 1.5 F_{su} for ASTM A1003/A 1003M Structural Grade 33 Type H (ST33H), Structural Grade 230 Type H [ST230H] steel (F_{su} = 310 MPa and 45 ksi), and 1.25 F_{su} for ASTM A1003/A

1003M Structural Grade 50 Type H (ST50H), Structural Grade 340 Type H [ST340H] steel (F_{su} = 448 MPa and 65 ksi)

n_s = the number of diagonal straps
b_s = the width of the diagonal straps
t_s = the thickness of the diagonal straps
W = the overall panel width
H = the overall panel height (see Figure 11-2 for a schematic panel drawing showing W and H).

Figure 11-2. Schematic of cold-formed steel shear panel model.



The effects of gravity load (dead, live, and snow load) and seismic forces shall be combined as follows to account for diagonal strap overstrength when the effect of gravity and seismic loads are additive by combining Equations 11-8, 11-10, 11-13, and 11-16:

$$(1.2 + 0.2S_{DS})D + \Omega_o Q_E + L + 0.2S \qquad \text{(Eq 11-18)}$$

The effects of gravity load and seismic forces shall be combined as follows to account for diagonal strap overstrength when the effect of gravity and seismic loads counteract each other by combining Equations 11-9, 11-11, 11-13, and 11-16:

$$(0.9 - 0.2S_{DS})D + \Omega_o Q_E + 1.6H \qquad \text{(Eq 11-19)}$$

For both expressions in Equations 11-18 and 11-19, the total horizontal force is $\Omega_o Q_E$. Except for H in Equation 11-19, every other term in these equations represents vertical loads. The shear panel systems should be analyzed based on the most critical load combination defined by either Equation 11-18 or 11-19. Each panel component (including all connections), other than the diagonal straps, should be designed based on these loads.

11.8 Deflection, drift limits, and building separation

The design story drift, Δ, shall not exceed the allowable story drift, Δ_a, as obtained from Table 11-7 (ASCE/SEI 7-10, Table 12.12-1), for any story. The design story drift shall be computed as the difference of deflections at the center of mass at the top and bottom of the story under consideration, as determined by Equation 11-34 (ASCE/SEI 7-10, section 12.8.6). For structures with significant torsional deflections, the maximum drift shall include torsional effects. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact under total deflection as defined in ASCE/SEI 7-10, section 12.12.3.

Table 11-7. Allowable story drift, Δ_a (in. or mm).

Structure	Risk Category		
	I or II	III	IV
Light-framed wall systems using flat strap bracing (diagonal strap shear walls)	$0.020h_{sx}$[11]	$0.015h_{sx}$	$0.010h_{sx}$

11.9 Equivalent lateral force procedure

ASCE/SEI 7-10 presents three permitted analytical procedures for defining the structural response (ASCE/SEI 7-10, Table 12.6-1), but only the equivalent lateral analysis (ASCE/SEI 7-10, section 12.8) is presented here. Only this procedure is presented because of the following: (1) its simplicity; (2) it can be used for all light-frame construction in all seismic design categories; and (3) typical cold-formed steel structures will likely be low-rise construction so that first mode response will dominate the seismic response of the structures. However, if deemed beneficial, the modal response spectrum analysis procedure presented in ASCE/SEI 7-10, section 12.9) could be used. The next three sections of this chapter present the determination of base shear, period, and vertical distribution of lateral forces using the *equivalent lateral force procedure*.

[11] h_{sx} is the story height below Level x.

11.9.1 Seismic base shear

Using the equivalent lateral force procedure, the seismic base shear, V, in a given direction shall be determined according to the following equation (ASCE/SEI 7-10, Eq 12.8-1):

$$V = C_s W \quad \text{(Eq 11-20)}$$

where:

C_s = the seismic response coefficient
W = the effective seismic weight (ASCE/SEI 7-10, section 12.7.2).

The seismic response coefficient, C_s, shall be determined according to the following equation (ASCE/SEI 7-10, Eq 12.8-2):

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \quad \text{(Eq 11-21)}$$

The value for C_s is calculated according to Equation 11-21, and need not exceed the following (ASCE/SEI 7-10, Eq 12.8-3 and 12.8-4):

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \text{ for } T \le T_L \quad \text{(Eq 11-22)}$$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \text{ for } T > T_L \quad \text{(Eq 11-23)}$$

where

T = the fundamental period of the structure determined below.
T_L = the long-period transition period, shown in ASCE/SEI 7-10 (ASCE/SEI 7-10, Figures 22-12 through Figure 22-16) but shall not be less than (ASCE/SEI 7-10, Eq 12.8-5):

$$C_s = 0.044 S_{DS} I_e \ge 0.01 \quad \text{(Eq 11-24)}$$

Also, for structures located where S_1 is greater than or equal to 0.6g (ASCE/SEI 7-10, Eq 12.8-6):

$$C_s = \frac{0.5 S_1}{\left(\frac{R}{I_e}\right)} \quad \text{(Eq 11-25)}$$

11.9.2 Period determination

The fundamental period of the building, T, in the direction under consideration shall be defined using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis (ASCE/SEI 7-10, section 12.8.2). Alternatively, T is permitted to be taken as the approximate fundamental period, T_a, determined in accordance with the following requirements. The fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period, C_u, from Table 11-8 (ASCE/SEI 7-10, Table 12.8-1) and the approximate fundamental period, T_a, determined as follows:

$$T_a = C_t h_n^x \tag{Eq 11-26}$$

where

C_t = constant = 0.02 (English) or 0.0488 (metric) for cold-formed steel shear panels with diagonal straps (ASCE/SEI 7-10, Table 12.8-2, for all other structural systems)
h_n = the structural height in feet (English) or meters (metric) which is the vertical distance from the base to the highest level of the seismic-force-resisting system of a structure. For pitched or sloped roofs, the structural height is from the base to the average height of the roof
x = 0.75 (ASCE/SEI 7-10, Table 12.8-2, for all other structural systems).

Table 11-8. Coefficient for upper limit on calculated period.

Design Spectral Response Acceleration at 1 second, S_{D1}	Coefficient C_u
$S_{D1} < 0.1g$	1.7
$0.1g \le S_{D1} < 0.15g$	1.7
$0.15g \le S_{D1} < 0.2g$	1.6
$0.2g \le S_{D1} < 0.3g$	1.5
$0.3g \le S_{D1} < 0.4g$	1.4
$0.4g \le S_{D1}$	1.4

11.9.3 Vertical distribution of lateral seismic forces

The vertical distribution of lateral seismic force, F_x (kip or kN), induced at any level shall be determined from the following equations (ASCE/SEI 7-10, section 12.8.3, Eqs 12.8-11 and 12.8-12):

$$F_x = C_{vx}V \quad \text{(Eq 11-27)}$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \text{(Eq 11-28)}$$

where

C_{vx} = the vertical distribution factor
V = the total design lateral force or shear at the base of the structure (kip or kN)
w_i and w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to Level i or x
h_i and h_x = the height (ft or m) from the base to Level i or x
k = an exponent related to the structure period as follows:
for structures having a period of 0.5 sec or less, k = 1
for structures having a period of 2.5 sec or more, k = 2
for structures having a period between 0.5 and 2.5 seconds, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

The horizontal distribution of seismic story shear in any story, V_x (kip or kN), shall be determined from the following equation (ASCE/SEI 7-10, section 12.8.4, Eq 12.8-13):

$$V_x = \sum_{i=x}^{n} F_i \quad \text{(Eq 11-29)}$$

where

F_i = the portion of the seismic base shear, V (kip or kN) induced at Level i.

The seismic design story shear, V_x (kip or kN), shall be distributed to the various shear panels in the story under consideration based on the relative lateral stiffness of the panels and the diaphragm.

11.9.4 Torsion

For buildings with flexible diaphragms (ASCE/SEI 7-10, section 12.3.1.1), the distribution of forces to the vertical elements (shear panels) shall account for the position and distribution of masses supported (i.e., distribute forces based on tributary area of the shear panels. For diaphragms that are not flexible (ASCE/SEI 7-10, sections 12.3.1.2 and 12.3.1.3), the distribution of lateral seismic forces shall take into account the effects of torsional moment, M_t, resulting from the location of masses relative to the center of rigidity (inherent torsional moment) of the lateral-force-resisting frame in both orthogonal directions (ASCE/SEI 7-10, section 12.8.4.1). This torsional moment shall include the effects of accidental torsional moment, M_{ta}, caused by an assumed offset of the mass. This offset shall equal 5% of the dimension of the structure orthogonal to the direction of the applied seismic force (ASCE/SEI 7-10, section 12.8.4.2). Structures assigned to seismic design category C, D, E, or F, which have Type 1a or 1b torsional irregularity or extreme torsional irregularity (ASCE/SEI 7-10, Table 12.3-1), shall include an amplification of accidental torsion as defined in ASCE/SEI 7-10 (ASCE/SEI 7-10, section 12.8.4.3). Similar to the lateral seismic forces, the torsional moments, M_t, are distributed along the floors of the building according to the vertical distribution factor given in Equation 11-28.

The torsional resistance comes from each of the shear panels, and the resistance from each panel is proportional to the square of the distance from the center of resistance to the plane of the panel. For a given panel, the additional shear force due to torsion, Q_{si}, can be expressed as:

$$Q_{si} = k_{si}\Delta_{si} = k_{si}\rho_i\theta \qquad \text{(Eq 11-30)}$$

where

k_{si} = the shear stiffness of shear panel i, and is defined as follows:

$$k_{si} = E n_s b_s t_s \left(\frac{W^2}{(H^2+W^2)^{3/2}}\right) \qquad \text{(Eq 11-31)}$$

Δsi = the additional lateral in-plane shear deflection due to torsion of panel i
ρi = the distance from the center of resistance to panel i, perpendicular to the plane of the panel
θ = the torsional rotation of the building at the floor level above the panel
E = the modulus of elasticity of steel, which is 29,000 ksi (200,000 MPa)
n_s = the number of diagonal straps in each direction
b_s = the width of the diagonal straps
t_s = the thickness of the diagonal straps
W = the overall panel width
H = the overall panel height (see Figure 11-2 for a schematic panel drawing showing W and H).

The torsional moment resistance, M_{tr}, for all the shear panels is given by:

$$M_{tr} = \sum_{i=1}^{n} \rho_i Q_{si} = \sum_{i=1}^{n} \rho_i^2 k_{si} \theta \quad \text{(Eq 11-32)}$$

Equation 11-32 shows that the torsional resistance from each panel is proportional to $\rho_i^2 k_{si}$. The total torsional moment resistance, M_{tr}, is set equal to the M_t, and the additional shear force due to torsion, Q_{si}, is calculated using Equations 11-30 and 11-32. Note that the torsional rotation, θ, in these equations does not need to be solved for and can be treated as a constant. Also the panel shear stiffness, k_{si}, is not needed if all the panels can be assumed to be equal or if their relative stiffness can be determined.

11.9.5 Structural overturning resistance

The structure shall be designed to resist overturning effects caused by the seismic forces determined from Equation 11-27. At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical-force-resisting elements in the same proportion as the distribution of the horizontal shears to those elements (ASCE/SEI 7-10, section 12.8.5).

The overturning moments at Level x, M_x (kN-m or kip-ft), shall be determined from the following equation:

$$M_x = \sum_{i=x}^{n} F_i (h_i = h_x) \quad \text{(Eq 11-33)}$$

where

F_i = the portion of the seismic base shear, V, induced at Level i
h_i and h_x = the height (ft or m) from the base to Level i or x.

Foundations designed for the foundation overturning design moment, M_f (kip-ft or kN-m), at the soil/foundation interface, determined using Equation 11-33 at the foundation level, may be reduced by 25% for foundations of structures that satisfy both of the following conditions (see ASCE/SEI 7-10, section 12.13.4):

1. The structure is designed in accordance with the Equivalent Lateral Force Analysis as set forth in ASCE/SEI 7-10, section 12.8.
2. The structure is not an inverted pendulum or cantilevered column type structure.

Overturning effects at the soil/foundation interface may be reduced by 10% for foundations of structures designed in accordance with the modal analysis requirements of ASCE/SEI 7-10 sections 12.9 and 12.13.4).

11.9.6 Story drifts

Story drifts shall be calculated based on the application of design seismic forces to a mathematical model of the structure. The model shall include the stiffness and strength of all elements that are significant to the distribution of forces and deformations in the structure, and it shall represent the spatial distribution of the mass and stiffness of the structure. The design story drift, Δ, shall be computed as the difference in the deflections at the center of mass at the top and bottom of the story under consideration. The deflections of Level x, δ_x (in. or mm), shall be determined according to the following equation (ASCE/SEI 7-10, Eq 12.8-15):

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \qquad \text{(Eq 11-34)}$$

where

δ_{xe} = the deflections determined by an elastic analysis (in. or mm) based on the forces defined in Equation 11-27.

For determining compliance with the story-drift limitations in Table 11-7, the deflections of Level x, δ_x (in. or mm), shall be calculated as expressed in Equation 11-34. For the purposes of this drift analysis only, the computed fundamental period of the structure, T, in seconds, may be used without the upper-bound limitations specified in Table 11-8 (ASCE/SEI 7-10, section 12.8.6.2), when determining drift-level seismic-design forces.

11.9.7 P-delta effects

The story drifts and member forces and moments due to P-delta effects shall be determined in accordance with ASCE/SEI 7-10, section 12.8.7. The design story drift, Δ (in. or mm), shall be increased by the incremental factor relating to the P-delta effects if required by the following recommendations. The P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects do not need to be considered when the stability coefficient, θ, as determined by the following equation, is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \quad \text{(Eq 11-35)}$$

where

P_x = the total vertical design load at and above Level x (kip or kN), where no individual load factors needs to exceed 1.0 in computing P_x
Δ = the design story drift occurring simultaneously with V_x (in. or mm)
V_x = the seismic shear force acting between Level x and x-1 (kip or kN).

The stability coefficient, θ, shall not exceed θ_{max}, which is determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \quad \text{(Eq 11-36)}$$

where

β = the ratio of shear demand to shear capacity for the story between Level x and x - 1. This ratio may conservatively be taken as 1.0.

When the stability coefficient, θ, is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to P-delta effects, a_d, can be determined by rational analysis (NEHRP 2009, Part 2 "Commentary," section C12.8.7). Alternatively, the P-delta effects can be accounted for by multiplying the drifts and member forces by 1.0/(1 - θ). When θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

11.10 Cold-formed steel seismic requirements

All boundary members, chords, and collectors shall be designed to transfer seismic forces originating in other portions of the structure to the shear panels, in accordance with ASCE/SEI 7-10, section 12.10.2. Connections for diagonal strap-to-column and column-to-anchor and shear panel anchorage, and collectors shall have adequate strength to account for the effects of material overstrength as indicated in these recommendations.

The pullout resistance of screws shall not be used to resist seismic forces.

Shear panels shall be anchored such that the bottom and top tracks are not required to resist uplift forces by bending of the track or track web. Both flanges of studs shall be braced to prevent lateral torsional buckling (IBC 2003, section 2211.4.3).

Provision shall be made for pretensioning or other methods of installing tension-only diagonal straps in order to guard against loose straps (AISI S213-07-C/SI-09-C, section C4).

The recommendations presented here require all-steel design, and does not permit the use of plywood sheathing or oriented strand board in cold-formed steel shear panels.

Shear panel design shall be based on the cold-formed steel shear design recommendations presented here. This design requires that shear panels be adequately anchored at their top and bottom to a floor diaphragm. Shear panels in the two orthogonal directions must be anchored to the same diaphragm at each floor level to tie the two orthogonal lateral load-resisting systems together. Shear panels above the ground floor must have shear panels in the same direction at every floor level below them.

Using the following recommendations, the diagonal straps are sized to resist the total horizontal loads at each floor level as defined in Equations 11-

12 and 11-13, based on trial shear panel locations and aspect ratios. Then the greater loads defined in Equations 11-18 and 11-19 shall be used to size the shear panel columns. Finally, the panel connections and anchors are designed according to the recommendations that follow.

11.11 Diagonal strap design

The diagonal straps are designed to resist the seismic story shears, V_x, given in Equation 11-29 that have been increased by the additional shear force due to torsion (Q_{si} in Equation 11-30). The shear panels shall be configured and diagonal straps designed so that the lateral shear panel design strength, $\phi_t Q_{sy}$ satisfies the following equation:

$$\emptyset_t Q_{sy} = \emptyset_t \sum_{i=1}^{n} \left[n_{si} b_{is} t_{is} F_{sy} \left(\frac{W_i}{\sqrt{H_i^2 + W_i^2}} \right) \right] \geq V_x + Q_{si} \quad \text{(Eq 11-37)}$$

where

ϕ_t = the resistance factor for tensile members, 0.90 (ASI 2007a, Appendix A, section C2)
n = the number of shear panels in the building frame for which the shear forces V_x and Q_{si} are applied
n_{si} = the number of diagonal straps (panel faces with straps) for shear panel i
b_{si} = the width of the diagonal straps in shear panel i
t_{si} = the thickness of the diagonal straps in shear panel i
F_{sy} = the design yield strength of the diagonal straps
W = the width of shear panel i
H = the height of shear panel i

This equation assumes the diagonal straps are the sole lateral-load-resisting element. It defines the lateral design capacity of the diagonal straps assuming they are tension only members and their design strength is defined by the AISI Specification S100-2007 (AISI 2007a, Appendix A, Equation C2-1). The number of shear panels, panel width, height, and strap size and strength shall be designed to meet the requirements of Equation 11-37. All diagonal strap material must be ASTM A 1003/A 1003M, Type H steel (ASTM 2013b). Diagonal straps may not use rerolled steel, because the rerolling strain hardens the material, increasing material strength variability and reducing elongation (see Chapter 4 for a discussion of this concern).

11.12 Column design

The columns may be hollow structural sections (HSS) or can be built up from studs. The columns of the Panel A and C configurations are built up with cold-formed steel studs. These studs must be oriented to form a closed cross-section as shown on the Test Panels A3 and C1 drawings in Appendix A. Individual studs must be welded to each other with a weld thickness equal to the thickness of the studs. The welds are intermittent, with a length and spacing that will ensure composite behavior of the column.

Structural tubing column design (Panel D configuration, Drawing D2 in Appendix A of this report) follows the same procedure, but consists of a single member that is a closed section by itself. The equations in these recommendations are used such that the number of studs making up this column is one.

11.12.1 Column applied loads

Loads applied to the columns are defined based on Equation 11-18, where the effects of gravity load and seismic forces are additive and diagonal strap overstrength is accounted for. Only that portion of gravity loads applied to the tributary area of the shear panel columns is included in the design of these columns. However, the full horizontal seismic force, $\Omega_o Q_E$, applied to the shear panel and resisted by the diagonal straps, will add a vertical component to the columns, increasing axial load. This horizontal load is based on the actual designed area of the diagonal straps as defined in Equation 11-17. The total column axial load at the maximum ultimate stress in the diagonal straps, P_{vumax} is:

$$P_{vumax} = \frac{GL_{max}}{2} + F_{sumax} n_s b_s t_s \left(\frac{H}{\sqrt{H^2+W^2}}\right) \qquad \text{(Eq 11-38)}$$

where

GL_{max} = the maximum gravity load per shear panel, i.e., from $(1.2 + 0.2S_{DS})D + \Omega_o Q_E + L + 0.2S$ in Equation 11-18

F_{sumax} = the maximum estimated ultimate stress in the diagonal straps, which equals to 1.5 F_{su} for ASTM A1003/A1003M, Type H, Grade 33 steel (F_{su} = 45 ksi and 310 MPa), and 1.25 F_{su} of Grade 50 steel (F_{su} = 65 ksi and 448 MPa) (ASTM 2013b; Larsen 1998).

11.12.2 Column axial capacity

The columns shall be designed such that their design strength, P_c, exceeds the total axial applied load, P_{vumax}. (Equation 11-38). Column capacity is determined based on AISI provisions. The design axial strength, P_c, shall be determined based on the AISI Standard S100-07 (AISI 2007a, section C4 “Concentrically Loaded Compression Members”). This guidance is applied to columns built-up with cold-formed steel studs or individual structural tubing members. The design strength equals the resistance factor times the nominal axial strength, P_n, determined as follows for columns built-up with cold-formed steel studs or individual structural tubing members (ibid.):

$$P_c = \phi_c P_{cn} = \phi_c A_e F_{cn} \qquad \text{(Eq 11-39)}$$

where

ϕ_c = the resistance factor for compression, which equals 0.85.
A_e = the effective area at the stress F_{cn}
F_{cn} = the nominal strength of the column, determined as follows:

for $\lambda_c \leq 1.5$,

$$F_{cn} = \left(0.658^{\lambda_c{}^2}\right) F_{cy} \qquad \text{(Eq 11-40)}$$

for $\lambda_c > 1.5$,

$$F_{cn} = \left(\frac{0.877}{\lambda_c{}^2}\right) F_{cy} \qquad \text{(Eq 11-41)}$$

where

$$\lambda_c = \sqrt{\frac{F_{cy}}{F_e}} \qquad \text{(Eq 11-42)}$$

and

F_e = the elastic flexural buckling stress for closed cross-sections as defined in the following equation:

$$F_e = \frac{\pi^2 E}{\left(\frac{KH}{r}\right)^2} \quad \text{(Eq 11-43)}$$

where

E = the modulus of elasticity, equal to 29,000 ksi
K = the effective length factor
H = the laterally unbraced height of the column
F_{cy} = the design yield strength of the column
r = the radius of gyration of the full, unreduced column cross section, calculated as follows:

$$r = \sqrt{I/_{Ac}} \quad \text{(Eq 11-44)}$$

The effective area, A_e, is calculated as follows for columns built up from cold-formed steel studs such that they form a closed section, or structural tube columns (AISI 2007a, C4.1):

$$A_e = A_c - nt_c(w-b) \quad \text{(Eq 11-45)}$$

where

A_c = the nominal column area
n = the number of studs making up the column, or equal to 2 when using structural tube columns
t_c = the thickness of the stud material used in the built-up columns or thickness of the structural tube column
w = the flat width of the stud web making up the built-up columns, or the width of the structural tubing face perpendicular to the plane of the panel.

Assuming the outside radius of the stud or tube corners is twice the thickness, t, then w may be calculated as follows:

$$w = b_c - 4t_c \quad \text{(Eq 11-46)}$$

where

b_c = the depth of the studs making up the built-up columns, or the structural tubing width perpendicular to the plane of the panel

b = the effective width and shall be determined as follows (AISI 2007, B2.2):

For $0.50 \geq \frac{d_h}{w} \geq 0, and \frac{w}{t_c} \leq 70$ and the distance between centers of holes ≥ 0.5w and ≥ $3d_h$,

$$b = w - d_h \text{ when } \lambda \leq 0.673 \quad \text{(Eq 11-47)}$$

$$b = \frac{w\left[1 - \frac{0.22}{\lambda} - \frac{0.8d_h}{w} + \frac{0.085d_h}{w\lambda}\right]}{\lambda} \text{ when } \lambda > 0.673 \quad \text{(Eq 11-48)}$$

In all cases, $b \leq w = d_h$

where

d_h = the diameter of holes

λ = a slenderness factor defined as follows (AISI 2007, B2.1):

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad \text{(Eq 11-49)}$$

where

f = the stress in compression element, which for compression members is taken equal to F_{cn} defined in Equations 11-40 and 11-41 (AISI 2007, C4).

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t_c}{w}\right)^2 \quad \text{(Eq 11-50)}$$

where

k = the plate buckling coefficient, equal to 4 for the studs making up the built-up columns or structural tube columns
μ = Poisson's ratio for steel, equal to 0.30.

11.12.3 Column bending and composite behavior

The column anchor design provisions presented later in these recommendations will create a moment connection. The primary purpose of the anchor design is to resist shear and uplift forces. However, this anchor design will also allow the columns to act as a moment frame, providing limited structural redundancy and widening of the hysteretic load-deflection envelopes of the shear panel. This will allow the panels to absorb more energy under cyclic loading conditions, which reduces building accelerations in an earthquake. Columns built up from studs must be designed to act as a composite cross section in order to provide this moment capacity. This requires welding between the studs that provide the shear transfer needed to develop the maximum moment in the columns. When one diagonal strap is in tension, the full gravity load on the shear panel may be carried in a single column, with the other column having no axial load. The maximum moment in a column will occur when it has no axial load. Therefore the welds shall be designed for the full moment capacity of the columns. This design requirement will allow the shear panel columns to continue providing bending resistance beyond the lateral yield deflection of the columns. These welds shall resist the maximum shear between the studs, which will be between the studs closest to the column neutral axis. This shear, q, is defined as follows:

$$q = \frac{V_c Q}{I_c} \qquad \text{(Eq 11-51)}$$

where

V_c = the maximum column shear due to column moment only
Q = the moment of the column cross-sectional area on one side of the critical weld about the critical weld plane
I_c = the moment of inertia of the column due to bending in the plane of the shear panel.

The maximum column shear, V_c, due to the maximum column moment, M_c, only is determined as follows:

$$V_c = \frac{2M_c}{H} = \frac{2F_{cy}I_c}{Hc} \qquad \text{(Eq 11-52)}$$

where

F_{cy} = the yield strength of the column. This strength is not increased for column material overstrength because weld failure is controlled by the column material strength, so that any material overstrength would result in proportionately greater weld strength.

c = the distance to the column neutral axis to the extreme fiber in the plane of the shear panel.

The moment of the column cross-sectional area on one side of the critical weld about the critical weld plane, Q, is defined as follows:

$$Q = \int_A y dA = A\bar{y} \qquad \text{(Eq 11-53)}$$

where

A = the area of column cross-section on one side of the critical weld plane closest to the column neutral axis

$\bar{y}$ = the distance from the neutral axis of the column cross-sectional area on one side of the critical weld plane to the critical weld failure plane.

Built-up columns are fabricated by welding individual studs together to form a closed cross-section using flare V-groove welds. The same weld size and spacing shall be used between all studs in the built-up column. These welds are design according to AISI (AISI 2007a, section E2.5, “Flare Groove Welds”), assuming double shear. The maximum spacing between centers of intermittent welds, s_{max}, is determined as follows:

$$s_{max} = 1.5\emptyset_G t_c F_{cu} \frac{L}{q} \qquad \text{(Eq 11-54)}$$

where

ϕ_G = the resistance factor for flare groove welds, equal to 0.55
t_c = the stud thickness of the built-up columns
F_{cu} = the ultimate strength of the column steel
L = the length of intermittent groove welds
q = the maximum shear determined in Equation 11-51.

Intermittent welds shall be made at both the top and bottom ends of the columns, regardless of the maximum center-to-center spacing, s_{max}.

11.12.4 Column combined axial and moment capacity

The combination of axial load and bending shall be evaluated using Equations C5.2.2-1, C5.2.2-2, and C5.2.2-3 of the AISI Specification (AISI 2007a, section C5.2.2, "Combined Compressive Axial Load and Bending, LRFD and LSD Methods"). Moment is only considered in the strong direction of the panels (M_x in these equations) because the column anchors are much more flexible in their weak direction, so that the panels will not resist loads in this direction. The combination of axial load and moment on the columns shall be evaluated based on the following interaction equations (modifications of AISI 2007a, Eq C5.2.2-1, C5.2.2-2, and C5.2.2-3):

$$I = \frac{\bar{P}}{\varphi_c P_n} + \frac{C_{mx}\bar{M}_x}{\varphi_b M_{nx} \alpha_x} \le 1.0 \qquad \text{(Eq 11-55)}$$

$$I = \frac{\bar{P}}{\varphi_c P_{no}} + \frac{\bar{M}_x}{\varphi_b M_{nx}} \le 1.0 \qquad \text{(Eq 11-56)}$$

When $\frac{\bar{P}}{\varphi_c P_n} \le 0.15$, then Equation 11-57 may be used in lieu of Equations 11-55 and 11-56.

$$I = \frac{\bar{P}}{\varphi_c P_n} + \frac{\bar{M}_x}{\varphi_b M_{nx}} \le 1.0 \qquad \text{(Eq 11-57)}$$

where

$\bar{P}$ = the required compressive axial strength, which is similar to Equation 11-38 but based on the maximum yield strength of the strap rather than the slightly more conservative maximum ultimate strength.

Since the straps must be fabricated from ASTM A1003/A1003M, Type H steel (ASTM 2013b), the straps will never be rerolled, so that the strap stress will never reach maximum ultimate stress, making the maximum yield strength a reasonable upper limit. The required compressive strength is calculated as follows:

$$\bar{P} = \frac{GL_{max}}{2} + F_{symax} n_s b_s t_s \left(\frac{H}{\sqrt{H^2+W^2}}\right) \qquad \text{(Eq 11-58)}$$

where

F_{symax} = maximum estimated yield stress in the diagonal straps, which equals 2 F_{sy} for ASTM A1003/A1003M, Type H, Grade 33 steel (F_{sy} = 33 ksi and 228 MPa) and 1.5 F_{sy} for Grade 50 steel (F_{sy} = 50 ksi and 345 MPa) (Larsen 1998)

$\Phi_c P_n$ = P_c, which equals the column axial design strength defined in Equation 11-39

C_{mx} = a coefficient set equal to 0.85 for compression members in frames subject to joint translation (AISI 2007, section C5.2.2)

$\bar{M}_x$ = the required flexural strength, set equal to the applied moment at maximum estimated strap yield strength, δ_{symax}, defined in Equation 11-59.

This equation assumes the columns are 50% fixed at their top and bottom by the panel anchors. Tension in the diagonal straps is responsible for the large axial load component of Equations 11-55 and 11-56, and this tension force will tend to counteract the moments applied to these columns, so that the 50% fixity of the columns is a reasonable limitation. This moment is also conservatively based on the maximum panel lateral deflection at which the diagonal strap will yield. The moment includes the column bending and P-delta effect of axial load. For the P-delta calculation in Eq 11-56, the vertical load, P_{vumax}, is a summation of the vertical loads from all columns whose lateral load resistance is provided by this column. The applied moment, The applied moment, $\bar{M}_x$, is defined as follows:

$$\bar{M}_x = \frac{(50\%)6EI_c\delta_{symax}}{H^2} + \bar{P}\delta_{symax} \qquad \text{(Eq 11-59)}$$

where

δ_{symax} = the maximum estimated lateral panel deflection at the maximum estimated yield strength of the diagonal straps (F_{symax}), and is defined as follows:

$$\delta_{symax} = \frac{F_{symax}}{E}\left(\frac{H^2+W^2}{W}\right) \qquad \text{(Eq 11-60)}$$

Φb = the resistance factor for bending, equal to 0.95 (AISI 2007a, section C5.2.2)

where

M_{nx} = the column nominal flexural strength, and this is defined as follows (modification of AISI 2007a, Equation C3.1.1-1):

$$M_{nx} = F_{cy}\frac{I_c}{c} \qquad \text{(Eq 11-61)}$$

$$\alpha_x = 1 - \frac{\bar{P}}{P_{Ex}} > 0 \qquad \text{(Eq 11-62)}$$

$$P_{Ex} = \frac{\pi^2 E I_c}{(K_x H)^2} \qquad \text{(Eq 11-63)}$$

$$P_{no} = A_e F_{cy} \qquad \text{(Eq 11-64)}$$

11.12.5 Column shear capacity

The trial column design must be checked for shear capacity. The diagonal straps fasten to the columns near their connection to the tracks and column anchor. Therefore the column must either have adequate shear capacity for the maximum horizontal seismic force, $\Omega_0 Q_E$, applied to the shear panel, or the column shear capacity must be augmented with other components. The column shear design strength, V_c, shall be determined as follows for columns built up with cold-formed steel studs or individual structural tubing members (AISI 2007a, section C3.2.1, “Shear Strength of Webs without Holes”):

$$\phi_v V_n = \phi_v A_w F_v \qquad \text{(Eq 11-65)}$$

For $\frac{h_c}{t_c} \leq \sqrt{\frac{E k_v}{F_{cy}}}$:

$$F_v = 0.60F_{cy} \quad \text{(Eq 11-66)}$$

For $\sqrt{\frac{Ek_v}{F_{cy}}} < \frac{h}{t} \leq 1.51\sqrt{\frac{Ek_v}{F_{cy}}}$:

$$F_v = \frac{0.60\sqrt{Ek_vF_{cy}}}{h_c/t_c} \quad \text{(Eq 11-67)}$$

For $\frac{h}{t} > 1.51\sqrt{\frac{Ek_v}{F_{cy}}}$:

$$F_v = \frac{0.904Ek_v}{h_c/t_c} \quad \text{(Eq 11-68)}$$

where

ϕ_v = the resistance factor for shear, equal to 0.95
V_n = the nominal column shear strength
A_w = the area of the column in shear

Only that portion of the column web that has a diagonal strap attached will be loaded in shear and available to resists these loads, expressed as follows:

$$A_w = n_sh_ct_c \quad \text{(Eq 11-69)}$$

where

h_c = the column depth, equal to the number of studs, n times the column stud flange width, b_f, for columns built up from studs or depth of the tube in the direction of the shear panel for structural tube columns
F_v = the column nominal shear stress
k_v = the column shear buckling coefficient, calculated in accordance with AISI S100-07 (AISI 2007a, section C3.2.1)

However, in the case of the shear panels, the diagonal straps apply shear load directly to the column faces, which are loaded in shear. This direct load transfer would be similar to a very large value of k_v. Therefore, Equation 11-66 is always used to define the shear capacity of the panel columns.

The example design in the following chapter shows that the columns normally will have insufficient shear capacity by themselves, and require additional shear capacity from their anchorage detail (see "Anchor Load Assumptions," section 11.14.1 of this report, for anchorage shear design details).

11.13 Connection design

Diagonal strap-to-column connections can be constructed using either screws or welds. As the following sections indicate, practical screwed connections are more difficult than welds for meeting connection design requirements.

11.13.1 Connection design assumptions and applied loads

This paragraph provides design assumptions that define loading and load-path issues for cold-formed steel shear panel diagonal strap-to-column connections. These loads are based only on the maximum lateral force, $\Omega_o Q_E$. This force results from the right-hand term in Equation 11-17, $\Omega_o Q_E$, which accounts for diagonal strap material overstrength. The maximum estimated ultimate force in the diagonal straps (in the axis of the straps), P_{sumax}, is:

$$P_{sumax} = F_{sumax} n_s b_s t_s \qquad \text{(Eq 11-70)}$$

The diagonal strap-to-column connections must be designed to resist the forces defined by Equation 11-70. Panel design will require the use of angle section anchors as described under "Panel Anchors" (section 11.14) because of the shear-transfer requirements. These anchors will also transfer loads between the column and beams above and below, or floor slab (i.e., diaphragm above and below), thereby eliminating the need for load transfer with a column-to-track connection. In low seismic zones it may be possible to transfer the shear forces with a column-to-track connection only, without anchors. However, it is considered more reasonable to use fewer shear panels rather than many with low lateral-load capacity. Therefore all shear panel design recommendations presented here require the use of anchors.

11.13.2 Screwed fastener connection design

Self-tapping screwed connection capacity definition shall follow the AISI Standard S100-07 (AISI 2007a, section E4 "Screw Connections"). Screws shall be installed and tightened in accordance with the manufacturer's recommendations. Screw connections loaded in shear can fail in one mode or in combination of several modes. These modes are screw shear, edge tearing, tilting and subsequent pullout of the screw, and bearing of the joined materials. The commentary for AISI S100 (2007b, section E4.3) gave further explanation and illustration of these modes of failure.[12] The minimum distance between the centers of screws shall not be less than three times their nominal diameter (AISI 2007a, section E4.1). The minimum distance from the center of a screw to the edge of a connected part perpendicular to the direction of loading (edge distance) shall not be less than 1.5 times the nominal screw diameter (AISI 2007a, section E4.2). The AISI provisions focus on the tilting and bearing modes of failure. Two cases are given depending on the ratio of the connected member thicknesses. The screw head will be in contact with the diagonal strap, and the strap will normally be thinner than the column. However, when the strap is the same thickness or thicker than the column, tilting becomes a more critical mode of failure. The AISI section E4 guidance on design shear strength per screw, P_s, applied to diagonal strap-to-column screw connections is summarized here. The design shear (AISI 2007a, section E4.3.1) and pullover per screw (AISI 2007a, section E4.4.2), P_s shall be calculated as follows:

$$P_s = \emptyset_s min(P_{ns} and P_{nov}) \qquad \text{(Eq 11-71)}$$

where the nominal shear strength per screw, P_{ns}, shall be determined as follows:

For $t_c/t_s \le 1.0$, P_{ns} shall be taken as the smallest of

$$P_{ns} = 4.2\sqrt{t_c^3 d}\, F_{cu} \qquad \text{column tilting mode of failure} \qquad \text{(Eq 11-72)}$$

$$P_{ns} = 2.7 t_s d F_{sumax} \qquad \text{diagonal strap bearing mode of failure} \qquad \text{(Eq 11-73)}$$

[12] The Commentary section of the 2007 revision of AISI S100 (AISI 2007b) does not include these illustrations.

$P_{ns} = 2.7t_c dF_{cu}$ column bearing mode of failure (Eq 11-74)

For $t_c/t_s \geq 2.5$, P_{ns} shall be taken as the smaller of

$P_{ns} = 2.7t_s dF_{sumax}$ diagonal strap bearing mode of failure (Eq 11-75)

$P_{ns} = 2.7t_c dF_{cu}$ column bearing mode of failure (Eq 11-76)

For $1.0 < t_c/t_s < 2.5$, P_{ns} shall be determined by linear interpolation between the two cases above

where

- ϕ_s = the screw resistance factor for shear, equal to 0.5
- d = the nominal screw diameter
- t_s = the thickness of the diagonal strap or member in contact with the screw head or washer
- t_c = the thickness of the column or member not in contact with the screw head or washer
- F_{sumax} = the maximum estimated ultimate strength of the diagonal straps, which are the members in contact with the screw heads or washers, which equals 1.5 F_{su} for ASTM A 1003/A 1003M, Type H, Grade 33 steel (F_{su} = 45 ksi and 310 MPa), and 1.25 F_{su} for Grade 50 steel (F_{su} = 65 ksi or 448 MPa)
- F_{cu} = the tensile strength of the columns, which are the members not in contact with the screw head or washer, and equals 45 ksi, 65 ksi and 58 ksi, for Grade 33, Grade 50, and ASTM A500, Grade B HSS structural tube steel, respectively.

The nominal shear strength per screw, P_{ns}, may also be determined by AISI Tables IV-9a and IV-9b (AISI 2008) for connections to various sheet thicknesses for sheets with ultimate strengths of 45 ksi (310 MPa) and 65 ksi (448 MPa), ASTM A 1003/A 1003M, Type H, Grade 33 and 50, respectively (ASTM 2013b). These tables may only be used if the grades of the materials being connected are the same.

The nominal shear strength per screw, P_{ns}, may also be limited by end distance of a connected part as defined by AISI (2007b, Appendix A, section E4.3.2), where the distance to the end is parallel to the direction of the applied force (e.g., near the end of a strap). This strength is calculated as follows:

$$P_{ns} = t_s e F_{sumax} \qquad \text{(Eq 11-77)}$$

where

e = the distance in the line of the applied force, from the center of a screw to the end of the connected part (diagonal strap for the shear panels).

The nominal shear strength of the screws, P_{ss}, shall be determined based on manufacturer's data (AISI 2007b, section E4.3.3), which must be based on tests according to AISI section F1.1.

The nominal pull-over strength, P_{nov}, shall be calculated as follows (AISI 2007b, section E4.4.2, "Pull-Over"):

$$P_{nov} = 1.5 t_s d'_w F_{sumax} \qquad \text{(Eq 11-78)}$$

where

d'_w = effective pull-over diameter determined in accordance with

a. For a round head, hex head, or hex washer head screw with an independent and solid steel washer beneath the screw head:

$$d'_w = d_h + 2t_w + t_s \leq d_w \qquad \text{(Eq 11-79)}$$

where

d_h = screw head diameter or head washer head integral washer diameter
t_w = steel washer thickness
d_w = steel washer diameter

b. For a round head, a hex head, or hex washer head screw without an independent washer beneath the screw head:

$d'_w = d_w$ but not larger than 0.5 in. (12.7 mm)

c. For a domed (non-solid and independent) washer beneath the screw head, it is permissible to use d'_w as calculated in Equation 11-79, with d_h, t_w, and t_s (t_1) as defined in AISI S100-07, Figure E4.4.2(3). In this equation, d'_w cannot exceed ⅝ in. (16 mm).

The modes of failure expressed in Equations 11-72 through 11-78 are defined alongside or before the equations. The connection-applied loads are based on the maximum estimated ultimate strength of the strap (F_{sumax} in Equation 11-70), which recognizes that as the applied load increases with increasing strap strength, the connection capacity also increases similarly for modes of failure based on this strength. However, for modes of failure based on the column strength (Equations 11-72, 11-74, and 11-76) or screw shear capacity based on manufacturer's data (P_{ss}), no increase is permitted.

Finally, the minimum number of screws required at each diagonal strap-to-column connection, n_{screws}, is calculated as follows:

$$n_{screws} \geq \frac{P_{sumax}}{n_s P_s} \quad \text{(Eq 11-80)}$$

11.13.3 Block shear rupture strength

The design shear strength along a potential shear rupture plane between fasteners of connected members, R_n, shall be determined in accordance with AISI S100-07 (AISI 2007a, Appendix A, section E5.3 "Block Shear Rupture") as follows:

$$R = \emptyset_R R_n \quad \text{(Eq 11-81)}$$

where

$\emptyset_R$ = the shear rupture resistance factor, equal to 0.65

R_n = the nominal block shear rupture strength, determined as the lesser of Equations 11-82 and 11-83:

$$R_n = 0.6 F_{sy} A_{gv} + F_{su} A_{nt} \quad \text{(Eq 11-82)}$$

$$R_n = 0.6 F_{su} A_{nv} + F_{su} A_{nt} \quad \text{(Eq 11-83)}$$

where

A_{gv} = the gross area subject to shear
A_{nv} = the net area subject to shear
A_{nt} = the net area subject to tension.

AISI S100-07 (AISI 2007a, Appendix A, section E3.2 “Rupture in Net Section (Shear Lag)”), defines the provisions for this mode of failure. However, for the diagonal strap-to-column screw connections used in these panel configurations, shear rupture rather than this mode will limit the connection capacity.

The shear and tensile rupture strength are based on the diagonal strap ultimate strength of the member in the joint being evaluated. The maximum applied load on this joint is based on the yield strength of the same member, P_{sy}. This will be much less than the maximum estimated strap axial force, P_{su}. The maximum force in the members is not critical, but rather is the minimum ratio of strap ultimate strength over yield strength (F_u/F_y) because the rupture strength capacity is dependent on F_u and the maximum applied force is dependent on F_y. This guidance requires that ASTM A1003/A1003M Type H steels be used for the straps (ASTM 2013b). Type H steels are defined as high-ductility materials in ASTM A1003/A1003M. These are designated as ST50H, ST40H, ST37H, and ST33H for structural-grade, high-ductility steel with design yield strengths of 50, 40, 37, and 33 ksi, respectively. Note B in Table 2 of the ASTM A 1003/A 1003M specification requires a minimum F_u/F_y ratio of 1.08 for Type H steels. Therefore the strap yield strength, P_{sy}, may be defined simply based on the ultimate strength of these materials. This requirement is expressed as follows:

$$R_{ns} \geq P_{sy} \qquad \text{(Eq 11-84)}$$

where

$$P_{sy} = \frac{F_{su}}{1.08} n_s b_s t_s \qquad \text{(Eq 11-85)}$$

When the strap-to-column rupture strength is evaluated based on Equation 11-84, the resistance factor in Equation 11-81 may be increased to 1.0, because of the ASTM minimum material requirement of F_u/F_y stated above. However, the 1.08 ratio for F_u/F_y will often be too small to make screwed connections practical, and the designer may be forced to use welded connections. The example design presented in Chapter 12 illustrates this difficulty.

11.13.4 Welded connection design

Welded design follows AISI guidance (AISI 2007a, section E2 "Welded Connections"), which covers connections of members in which the thickness of the thinnest member is 3/16 in. (4.76 mm) or less. Arc welds shall be made in accordance with AWS D1.3 and its commentary (AWS 2008). Resistance welds shall be made in accordance with the procedures in AWS C1.1. (AWS 2012)

Welded diagonal strap-to-column connections require fillet welds (AISI 2007b, section E2.4 "Fillet Welds"). The welds at the sides of the straps will be loaded in the longitudinal direction, and welds at the ends of the straps will be loaded in the transverse direction. The weld thickness should be equal to the thickness of the strap material. Ultimate failure of fillet-welded joints has usually been found to occur by the tearing of the sheet steel adjacent to the weld. In most cases, the higher strength of the weld material prevents weld shear failure, so this recommendation is based on sheet tearing (AISI 2007b, section E2.4 "Fillet Welds"). The AISI commentary further explains that research demonstrates that weld throat failure does not occur for materials thinner than 0.10 in. (ibid.), and the AISI Specification S100-07 (AISI 2007a) requires the welded connection capacity be determined based on the strength of the weld material only for welds thicker than 0.10 in. The shear strength of welded diagonal strap-to-column connections shall be determined based on the AISI Specification S100-07 (AISI 2007a) and these are summarized below. The design shear strength for loading in the longitudinal direction, P_L, shall be determined as follows:

For $L/t < 25$
$$P_L = \phi_{L1} P_n = \phi_{L1}\left(1 - \frac{0.01L}{t}\right) L t F_u \qquad \text{(Eq 11-86)}$$

For $L/t \geq 25$
$$P_L = \phi_{L2} P_n = \phi_{L2} 0.75 t L F_u \qquad \text{(Eq 11-87)}$$

where

ϕ_{L1} = 0.60
L = the length of longitudinal fillet weld

t = the least value of the thicknesses of the diagonal straps (t_s) or columns (t_c) being welded
F_u = the design ultimate strength of the thinner material being welded (F_{su} or F_{cu})
ϕ_{L2} = 0.50.

The design shear strength for loading in the transverse direction, P_T, shall be determined as follows:

$$P_T = \phi_T P_n = \phi_T t L F_u \qquad \text{(Eq 11-88)}$$

where

ϕ_T = 0.65.

For fillet welds to both strap and column material thicker than 0.10 in. (2.54 mm), the design shear strength for both longitudinal and transverse loading due to weld failure, P_W, shall not exceed the following (AISI 2007b, section E2.4)[13]:

$$P_w = 0.75 \emptyset_w t_w L F_{xx} \qquad \text{(Eq 11-89)}$$

where

ϕ_w = 0.60
t_w = the effective throat, equal to 0.707 times the least of (1) the thickness of the strap, t_s, (2) the thickness of the weld against the strap in the axis of the strap thickness, which should normally equal the thickness of the strap, or (3) the thickness of the weld along the column in the axis of the strap, which should normally equal or exceed the strap thickness. (A larger effective throat shall be permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w. Figure E2.4-1 (AISI 2007b) illustrates these weld thicknesses and limitations.)

[13] AISI commentary (AISI 2007b) indicated that this equation is needed to cover the possibility of weld failure through the throat of the weld material, because research showed that for high strength welded members, weld throat failure could occur for welded materials thicker than 0.10 in.

F_{xx} = the tensile strength of electrode classification (AISI 2007b, section E2.2.1.2).

The fillet weld longitudinal and transverse shear strengths are based on the ultimate strength of the thinner member (normally the diagonal strap) of the joint. The maximum applied load on this joint is based on the yield strength, P_{sy}, of the same member. Similar to the rupture strength, the maximum force in the members is not critical, but rather is the minimum ratio of F_u/F_y because the weld strength capacity is dependent on F_u and the maximum applied force is dependent on F_y. Weld connections shall be evaluated based on the following equation, where the applied strap yield strength, P_{sy}, is defined according to Equation 11-70:

$$[min(P_L + P_T, P_w)]n_s \geq P_{sy} \qquad \text{(Eq 11-90)}$$

11.14 Panel anchors

Panel anchors must be installed on both sides of the shear panel columns because the columns by themselves have inadequate shear capacity. Furthermore, if the columns were simply fastened to the track, the track would be loaded in bending, due to uplift. The track is very weak in bending and should not be relied on to restrain the columns. Anchors consisting of angle iron sections with a stiffening plate shall be welded to both sides of the columns at both their tops and bottoms to provide the required panel anchorage. A stiffening plate shall be welded to each angle and the angles shall be drilled with through holes and anchored to the supporting diaphragm above and below the shear panel using embedded anchor bolts or bolts through intermediate floor diaphragms. See Chapter 12 (Figure 12-4 through Figure 12-15) for examples of this anchor configuration.

The columns (see section 11.12.5) and anchors together must have adequate capacity to resist the horizontal forced defined by Equation 11-91. The vertical leg of the angle iron anchors must extend beyond the critical shear plane. For screwed fastener connections, the critical shear plane is along the horizontal row of screws closest to the track in the diagonal strap-to-column connection. For welded connections, the critical shear plane is along the strap-to-column weld near the track. The anchor angle iron legs are welded to the columns along the top of the vertical leg and along the vertical edge of these legs to the corners of the columns. These welds force this face of the column to act in a composite fashion with the angle vertical legs.

11.14.1 Anchor load assumptions

The most critical load condition for anchors is when the effects of gravity load and seismic forces counteract each other. This condition is expressed in Equation 11-19. The selected angle and stiffener plate anchors shall resist the applied shear and uplift forces. These anchors also provide limited moment resistance. The configuration of these anchors (see Figure 12-4 through Figure 12-15 for examples) gives them strength and stiffness to resist shear and uplift forces where they anchor the columns to the diaphragms. The anchors should remain elastic against these forces. However, moments will be applied to the column anchors by two means. At small deformations, the greatest moment in the column anchors will come from eccentric loading of the anchor through the diagonal strap tension. At larger panel deformation, the columns will displace in-plane and this will apply moments to the anchors as they resist rotation. The legs of the angles in the anchors should be designed for the horizontal force when the diagonal strap it at its maximum ultimate stress, P_{humax} (see Equation 11-91). However, these legs may yield and deform in-elastically under the maximum moment applied through eccentric loading from the strap and bending of the columns, when the straps and columns are at their maximum estimated stress. As the legs deform, their load path will more effectively carry the loads that result from the maximum estimated strap force and column bending. These recommendations are written to permit inelastic ductile response in the column anchors, which will actually improve ductile system behavior and improve design economy under these extreme load conditions.

$$P_{humax} = \Omega_0 Q_E = F_{sumax} n_s b_s t_s \left(\frac{W}{\sqrt{H^2+W^2}}\right) \qquad \text{(Eq 11-91)}$$

11.14.2 Anchor bending capacity

The legs of the anchor angles will be loaded in bending by the horizontal loads applied by the panel diagonal straps. The faces of the columns carry a portion of this load in shear down to the base of angles and transfer the load to horizontal legs of the angles (see Equation 11-65). The vertical legs of the anchor angles together with the out-of-plane faces of the column will together carry a portion of this load in bending to the bottom of the angle and stiffener plate. The critical bending plane of this angle and column section will be along a diagonal line from the top center of the angle next to the fillet weld to the stiffener down to where the vertical leg of the angle begins to thicken at the radii intersection with the horizontal leg of the angle. The vertical legs of the angles and the face of the columns will carry this load together in bending as a composite section because they are welded to each other along both the top and sides of the vertical leg of the angles. Figure 12-4 through Figure 12-5 show example anchor configurations that aid the visualization of this load path from the straps to the anchor bolts.

The total design shear strength, V_T, from the column loaded in shear and the vertical legs of the angles loaded in bending must exceed the maximum shear panel horizontal seismic force, P_{humax}. All anchors are made using two angles, on either side of the column, so that V_T may be expressed as:

$$V_T = V_C + P_{Ah} \geq P_{humax} \qquad \text{(Eq 11-92)}$$

where

V_c = the column shear capacity determined according to Equation 11-65

P_{Ah} = the horizontal load capacity of the vertical legs of the angles on both sides of the columns and the faces of the columns to which the angles are welded based on the composite bending capacity of these two components, defined as follows:

$$P_{Ah} = 2\frac{M_A}{L_A}n_s \qquad \text{(Eq 11-93)}$$

M_A = the design moment capacity of a single leg of angle acting in composite with the face of the column to which it is welded, based on AISI/AISC 360-10 section F1 and F7, for a single angle leg and portion of the column defined as follows (AISC 2010a, section F7):

$$M_A = \emptyset_b F_{Ay} Z_A \qquad \text{(Eq 11-94)}$$

Φ_b = the bending resistance factor equal to 0.90 (AISC 2010a, section F1)

F_{Ay} = angle yield strength, normally ASTM A36, 36 ksi (AISC 2011, Part 2: "General Design Considerations," Table 2-4)

Z_A = the plastic section modulus of the critical section vertical leg of the angle and attached column, at a diagonal angle from the top of the stiffener to the point angle along the outside corner of the column just above where the angle begins to thicken at the fillet where the two legs of the angle join, defined as follows:

$$Z_A = \frac{W_A(t_A+t_c)^2}{4} \quad \text{(Eq 11-95)}$$

W_{Ab} = the width of the angle along the critical bending plane defined as follows:

$$W_{Ab} = \sqrt{{d_{Ah}}^2 + {d_{Av}}^2} \quad \text{(Eq 11-96)}$$

d_{Ah} = the horizontal moment arm for angle leg loaded in bending, defined as follows:

$$d_{Ah} = \frac{b_c}{2} - t_{wA} - \frac{t_s}{2} \quad \text{(Eq 11-97)}$$

t_{wA} = the thickness of the weld connecting the anchor stiffener plate to the anchor angle

t_s = the thickness of the anchor stiffener plate

d_{Av} = the vertical height of the angle that is vulnerable to bending failure, which is defined as follows:

$$d_{Av} = H_A - k \quad \text{(Eq 11-98)}$$

H_A = the width of the angle leg that is oriented vertically along the side of the column

k = the anchor angle thickness, t_A plus the leg-to-leg fillet radii of the angle shown in AISC (2011), Part 1, "Dimensions and Properties," Table 1-7

t_A = the angle thickness

L_A = the angle moment arm, perpendicular to the critical bending plane defined by W_{Ab}, defined as follows:

$$L_A = \frac{d_{Av}}{W_{Ab}} L_{Av} \quad \text{(Eq 11-99)}$$

L_{Av} = the vertical moment arm, which equals the vertical distance from the critical shear plane, at the last row of screws or horizontal weld that connects the diagonal strap to the columns, to point on the angle vertical leg where it begins to thicken at the radii intersection with the horizontal leg (k defined above).

11.14.3 Column-to-anchor angle weld design

The angles can yield significantly through many cycles with no loss of shear and uplift resistance (but some loss of moment resistance). The welds along the top edge of the angles to the columns will be fillet welds, while the welds along the sides of the angles along the corners of the columns will be flare bevel groove welds (see Figure E2.5-2 of AISI 2007b). The maximum fillet weld thickness between anchors and columns are limited to the values shown in Table 11-9, when the minimum thickness of the component welded (the column) is greater or equal to 3/16 in. For columns with thickness less than 3/16 in., no maximum weld thickness is identified in AISI provisions (AISI 2007b, section E2.5 and Figure E2.5-4)[14]. The maximum angle thickness shall be based on the column-to-anchor weld thickness as indicated in Table 11-10.

Table 11-9. Maximum column-to-anchor fillet weld thickness (AISC 2010a, section J2b).

Column Material Thickness, t_c	Maximum Weld Thickness, t_w
3/16 in. (4.8 mm) ≤ t < ¼ in. (6 mm)	$t_w = t_c$
t ≥ ¼ in. (6 mm)	t_w = tc – 1/16 in. (2 mm)

Table 11-10. Maximum angle thickness based on column-to-anchor fillet weld thickness (AISC 2010a, Table J2.4).

Weld Thickness, t_w	Maximum Angle Thickness, t_A
1/8 in. (3 mm)	1/4 in. (6 mm)
3/16 in. (5 mm)	1/2 in. (13 mm)
1/4 in. (6 mm)	3/4 in. (19 mm)
5/16 in. (8 mm)	1-1/8 in. (29 mm)[15]

[14] The AISI Specification in fact provides guidance for Flare Groove Welds (E2.5), where the weld thickness is greater than twice the thickness of the thinner connected member—double shear design in Figure E2.5-4.

[15] Maximum thickness of standard angles shown in the *Steel Construction Manual* (AISC 2011).

The column-to-angle weld design strength, P_A, shall exceed the total uplift force applied to one angle at one side of the column due to uplift and bending. This is expressed as follows:

$$\frac{P_{vymax}}{2} + P_M \leq P_A = P_T + P_G \qquad \text{(Eq 11-100)}$$

where

P_A = the total vertical design capacity of the column-to-angle weld
P_T = the design strength of the transverse loaded fillet weld at the horizontal column-to-angle weld (Equation 11-88)
P_G = the design strength of the longitudinal loaded flare bevel groove weld at the vertical column-to-angle welds at the corner of the columns (Equation 11-101 or 11-102). The design strength for this column-to-angle weld shall be determined based on AISI guidance (AISI 2007a, section E2.5, "Flare Groove Welds"). The application of this guidance to the design of column-to-angle welds is determined as follows:

For $t_c \leq t_w < 2t_c$ (single shear)

$$P_G = 0.75\phi_G t_c L F_{cu} \qquad \text{(Eq 11-101)}$$

For $t_w \geq 2t_c$ (double shear)

$$P_G = 1.5\phi_G t_c L F_{cu} \qquad \text{(Eq 11-102)}$$

where

ϕ_G = the resistance factor for flare groove welds, equal to 0.55
t_c = the thickness of the column material
L = the length of the flare bevel groove weld
F_{cu} = the ultimate strength of the column steel
P_{vymax} = the net anchor vertical load at the maximum yield stress in the diagonal straps, expressed by:

$$P_{vymax} = F_{symax} n_s b_s t_s \left(\frac{H}{\sqrt{H^2+W^2}}\right) - \frac{GL_{min}}{2} \qquad \text{(Eq 11-103)}$$

where

GL_{min} = the minimum gravity load per shear panel, i.e., $(0.9 - 0.2S_{DS})D$, in Equation 11-19

P_M = the tensile force per anchor, beyond half the net anchor vertical load, P_{vymax}, that must be available to develop the design yield stress in the columns through combined tension and bending, which is determined by Equation 11-104. This assumes the anchor bolts are sufficiently tightened to provide a moment restraint:

$$P_M = \frac{M_{Rem}}{h_c} \qquad \text{(Eq 11-104)}$$

where M_{Rem} is the moment capacity remaining in the column at which the maximum column fiber stress reaches its design yield value when subjected to both this moment and the maximum diagonal strap yield stress, P_{symax}. This capacity is defined as follows:[16] [17]

$$M_{Rem} = \left(F_{cy} - \frac{P_{vymax}}{A_c}\right)\left(\frac{I_c}{h_c - c}\right) \qquad \text{(Eq 11-105)}$$

11.14.4 Anchor bolt design

The anchor bolts that fasten the column anchors to the reinforced concrete beam or slab are designed next. The same detail used in the anchors at the base of the columns shall be used in the anchors at the top of the columns. The anchor bolts shall be sized based on the bolt shear strength, P_v, tensile strength, P_t, and cone failure design strength, P_c. The anchor bolt shear design strength, P_v, (AISC 2010a, section J3.7, "Combined Tension and Shear in Bearing-Type Connections") shall exceed the applied shear load per bolt, P_{hAB}. This is expressed as follows:

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[16] Note that the actual remaining moment capacity of the column can be greater than expressed by this equation because the actual column yield stress can be greater than the design yield stress. However, the anchor welds that are designed based on this expression are limited by the ultimate strength of the column material, so overstrength in the column yield will also result in greater strength in the anchor connection. This equation may also underestimate the loads applied to the anchor itself and anchor bolts. Still, this expression is sufficiently conservative, due to the significant overstrength accounted for in the diagonal strap (Equation 11-103) and the overstrength that will be present in both the anchor material and anchor bolts.

[17] The moment of inertia, I_c, in Equation 11-105 is divided by $h_c - c$, rather than c, because $h_c - c$ is the distance from the neutral axis to the outside extreme fiber of the column, which will be most critically stressed when the diagonal strap is in tension.

$$P_v \geq P_{hAB} = \frac{P_{humax}}{n_{AB}} \quad \text{(Eq 11-106)}$$

where

$$P_v = \emptyset_{tv} F_{nv} \frac{\pi}{4} d_{AB}^2 \quad \text{(Eq 11-107)}$$

n_{AB} = the total number of anchor bolts in the anchors on both sides of the column

ϕ_{tv} = the tensile and shear resistance factor, equal to 0.75 (AISC 2010a, section J3.7)

F_{nv} = the nominal shear strength of the anchor bolts (AISC 2010a, Table J3.2 "Nominal Strength of Fasteners and Threaded Parts,") ksi (MPa)

d_{AB} = the diameter of the anchor bolts

P_{humax} = the maximum shear panel horizontal force defined by Equation 11-91.

The anchor bolt tensile design strength, P_t, shall exceed the applied tensile force per bolt, P_{tAB}.[18] This is expressed as follows:

$$P_t \geq P_{tAB} \quad \text{(Eq 11-108)}$$

where

P_t = the anchor bolt tensile strength (ϕR_n in ANSI/AISC 360-10, section J3.7), determined as follows:

$$P_t = \emptyset R_n = \emptyset_{tv} F'_{nt} \frac{\pi}{4} d_{AB}^2 \quad \text{(Eq 11-109)}$$

[18] Consideration has been given to increasing the applied tensile force per anchor bolt, P_{tAB} to account for the effects of out-of-plane prying action at these joints, between stiffener plates and the anchor bolts in accordance with AISC *Steel Construction Manual* (AISC 2011, Part 9, pp 9-10 to 9-13). However, the anchor bolts are placed as close as possible to the stiffener plates to minimize this effect; prying in this out-of-plane direction is a minor secondary effect. Furthermore, the vertical leg of the anchor angles will be much more vulnerable to yielding between corners of the columns and the stiffener plates. The vertical leg yielding and deformation will act as a fuse to limit the deformation of the horizontal leg of the angle and thereby minimize prying action.

where

F'_{nt} = the nominal tensile stress modified to include the effects of shear stress, ksi (MPa), in accordance with ANSI/AISC 360-10 (AISC 2010a, section J3.7 and Table J3.2), as follows:

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\varphi_{nt}F_{nv}} f_{rv} \leq F_{nt} \qquad \text{(Eq 11-110)}$$

where

F_{nt} = the nominal tensile stress for the bolts, ksi (MPa), ANSI/AISC 360-10 (AISC 2010a, Table J3.2)
F_{nv} = the nominal shear stress for the bolts, ksi (MPa), ANSI/AISC 360-10 (AISC 2010a, Table J3.2)
f_{rv} = the required shear stress, defined as follows:

$$f_{rv} = \frac{P_{hAB}}{\pi/4 d_{AB}^2} \qquad \text{(Eq 11-111)}$$

The applied tensile force per anchor bolt, P_{tAB}, is calculated as follows:

$$P_{tAB} = \frac{P_{symax}L_S - \left(\frac{GL_{min}}{2}\right)\left(\frac{h_c}{2} + W_A\right) + M_{Rem}}{(d_c + h_c + W_A)\left(\frac{n_{AB}}{2}\right)} \qquad \text{(Eq 11-112)}$$

where P_{symax} is the maximum yield strength of the diagonal strap(s) in the shear panel, in the axis of the strap. This is determined as follows:

$$P_{symax} = F_{symax} n_s b_s t_s \qquad \text{(Eq 11-113)}$$

where

L_s = the distance between the where the centerline of the diagonal strap-to-column connection crosses the outside vertical plane of the column to the inside edge of the interior anchor of the same column, perpendicular to the axis of the strap. This is the lever arm that is multiplied by P_{symax} to apply a moment about the inside corner of the interior anchor about which the anchor would rotate if the anchor bolts failed, defined as follows:

$$L_S = \frac{WS_v}{\sqrt{W^2+(H-2S_v)^2}} + (h_c + W_A)\left(\frac{H-2S_v}{\sqrt{W^2+(H-2S_v)^2}}\right) \qquad \text{(Eq 11-114)}$$

where

S_v = the vertical distance between where the centerline of the diagonal strap-to-column connection crosses the outside vertical plane of the column, to the top of the column top connections or bottom of the column bottom connections
W_A = the width of the anchor (angle section) in the in-plane direction of the shear panel
d_c = the horizontal distance between the face of the column to the anchor bolt(s) at the exterior of the column.
n_{AB} = the number of anchor bolts per column (either 2 or 4), symmetrically placed to both the interior and exterior of the columns (see Figure 12-4 through Figure 12-15 for examples).

The anchor bolts should be placed as close to each other as possible in the out-of-plane direction (i.e., on either side of the anchor stiffer plate) so as to minimize bending stress on the anchor angle, deformation of the angle, or prying action. Equation 11-115 provides the minimum distance between these anchor bolts. For anchors with two anchor bolts (n_{AB} = 2), only one anchor bolt is placed on each side of the column and d_{c-c} is the twice the distance between the center of the anchor bolt and center of the stiffener plate in the out-of-plane direction.

$$d_{c-c\,\min} = OD + 2t_{wA} + t_S \qquad \text{(Eq 11-115)}$$

where

OD = the outside diameter of the anchor bolt washer, for the anchor bolt of diameter, d_{AB} (AISC 2011, Part 7, “Design Considerations for Bolts,” Table 7-14)
t_{wA} = the thickness of the angle to stiffener weld
t_S = the thickness of the stiffener plate.

11.14.5 Anchor angle thickness and anchor angle to stiffener weld

Trial anchor angle, selected based on Equation 11-92, must be checked for capacity between the stiffener plate and anchor bolt. Four modes of failure are possible: (1) weld failure between the stiffener plate and angle; (2)

shear rupture failure of the stiffener base metal along the weld to the angle; (3) shear yielding of the angle between the stiffener weld and bolt; and (4) shear rupture of the angle between the stiffener weld and bolt. The capacity of the stiffener plate to angle weld is defined as follows (AISC 2010a, Table J2-5):

$$P_{tAB} \leq P_{Aw} = 0.60\Phi_w F_{xx} t_{Aw} L_{Aw} \qquad \text{(Eq 11-116)}$$

where

Φ_w = the weld shear loaded resistance factor, equal to 0.75
t_{Aw} = the fillet weld thickness between the stiffener plate and angle. The minimum effective thickness used to calculate the fillet weld strength in Equation 11-116 is 0.707 times the nominal weld thickness, t_{Aw} (see the definition for fillet weld effective throat, t_w in AISI 2007b (section E2.4, Fillet Welds, p 91)
L_{Aw} = the effective length of the weld between the stiffener plate and angle. The effective length of this weld is taken as less than the width of the angle minus the angle thickness and radii of the angle fillet (W_A – k), recognizing that the load will be concentrated near the anchor bolt. The edge distance between the anchor bolt and the end of the anchor and the outside diameter of the bolt washers (OD), will all influence the effective length.

The shear rupture capacity of the stiffener base metal along the weld to the angle is defined as follows (AISC 2010a, section J4.2, Eq. J4-4):

$$P_{tAB} \leq P_{Su} = 0.60\Phi_{su} F_{Su} t_{Aw} L_{Aw} \qquad \text{(Eq 11-117)}$$

where

Φ_{Su} = the shear rupture resistance factor, equal to 0.75
F_{Su} = the ultimate strength of the stiffener steel.

The shear yielding capacity of the angle between the stiffener weld and anchor bolts is defined as follows (AISC 2010a, section J4.2, Eq J4-3):

$$P_{tAB} \leq P_{Ay} = 0.60\Phi_{sy} F_{Ay} t_A L_{Ay} \qquad \text{(Eq 11-118)}$$

where

Φ_{sy} = the shear yielding resistance factor, equal to 1.00
L_{Ay} = the effective length of critical yield and rupture surface of the angle between the stiffener plate and anchor bolt. This length is taken as the smaller of the twice the anchor bolt washer outside diameter and the washer outside diameter plus the edge distance from the bolt to the edge of the anchor, defined as follows:

$$L_{Ay} = min(2OD, OD + W_A - d_c) \qquad \text{(Eq 11-119)}$$

The shear rupture capacity of the angle between the stiffener weld and anchor bolts is defined as follows (AISC 2010a, section J4.2, Eq J4-4):

$$P_{tAB} \leq P_{Au} = 0.60\Phi_{su}F_{Au}t_A L_{Ay} \qquad \text{(Eq 11-120)}$$

where

F_{Au} = the ultimate strength of the angle steel.

11.14.6 Cast-in anchor bolt breakout strength in tension

Panel anchors must be attached to the foundation or floor and roof diaphragms using cast-in anchors (or through-bolts for intermediate floor diaphragms and roof diaphragms as defined in the next section). The nominal concrete breakout strength in tension, N_{cb}, of a single anchor or N_{cbg} of a group of anchors shall not exceed the following specifications (ACI 2011a, Appendix D: "Anchoring to Concrete," section D.5.2.1).

For one anchor bolt on each side of the column (i.e., n_{AB} = 2):

$$P_{tAB} \leq \Phi_{ct}N_{cb} = \Phi_{ct}\frac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b \qquad \text{(Eq 11-121)}$$

For two anchor bolts on each side of the column (i.e., n_{AB} = 4):

$$P_{tAB} \leq \frac{2\Phi_{ct}N_{cbg}}{n_{AB}} = \frac{2\left(\Phi_{ct}\frac{A_{Nc}}{A_{Nco}}\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b\right)}{n_{AB}} \qquad \text{(Eq 11-122)}$$

where

Φ_{ct} = the concrete anchor strength reduction factor for concrete breakout, side-face blowout, pullout, or pryout under tension loads, equal to 0.75, when supplementary reinforcement is present (Condition A) (ACI 2011a, Appendix D, section D.4.3.c)

A_{Nc} = the projected concrete failure area of a single anchor or group of anchors that shall be approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward 1.5 h_{ef} (effective embedment depth of the anchor bolts) from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors. A_{Nc} shall not exceed nA_{Nco} where n is the number of anchors in the group that resist tension, and equals $N_{AB}/2$. The following expression is used to calculate A_{Nc} (ibid.):

$$A_{Nc} = \left(c_{a1} + 1.5h_{ef}\right)\left(c_{a2} + d_{c-c}+1.5h_{ef}\right) \qquad \text{(Eq 11-123)}$$

where

c_{a1} = the edge distance from the anchor bolts outside the columns to the edge of the concrete diaphragm in the plane of the shear panel. If c_{a1} is greater than or equal to 1.5 h_{ef}, then c_{a1} is set equal to 1.5 h_{ef}.

h_{ef} = the effective embedment depth of the anchor bolts.

c_{a2} = the edge distance from the anchor bolt closest to the edge of the concrete diaphragm in the out-of-plane direction of the shear panel. If c_{a2} is greater than or equal to 1.5 h_{ef}, then it is set equal to 1.5 h_{ef}.

d_{c-c} = the actual center-to-center distance between two anchor bolts at the outside of the column when the number of anchor bolts on both sides of the column, n_{AB}, equals four. If there is a single anchor bolt on each side of the column, where n_{AB} equals 2, d_{c-c} becomes 0 (zero) in Equation 11-126. Note that the expression in Equation 11-115 $d_{c-c\,min}$ applies to the case where one anchor bolt is located on both sides of the column and then it is meant to define the out-of-plane distance from the center of the stiffener to the single anchor bolt.

A_{Nco} = the projected concrete failure area of a single anchor with edge distance equal to or greater than 1.5 h_{ef}, where

$$A_{Nco} = 9h_{ef}^2 \qquad \text{(Eq 11-124)}$$

$\Psi_{ed,N}$ = a modification factor for edge effects for single anchors or anchor groups loaded in tension. This term equals 1.0 if the minimum of the edge distances, c_{a1} and c_{a2}, defined above, is equal to or greater than 1.5 h_{ef}. If either of these terms is less than 1.5 h_{ef}, then $\Psi_{ed,N}$ is calculated based on the minimum of these terms as follows:

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \qquad \text{(Eq 11-125)}$$

$\Psi_{c,N}$ = a modification factor permitted for regions of where analysis indicates no cracking at service load levels. This factor may be 1.25 for cast-in anchors and 1.4 for post-installed anchors where the value for k_c used in Equation 11-126 is 17. Normally a value of 1.0 should be used, assuming some cracking of the concrete occurs.

$\Psi_{cp,N}$ = a modification factor post-installed anchors designed for untracked concrete without supplementary reinforcement to prevent splitting. This factor shall be computed in accordance with ACI 318-11, section D.5.2.7, using one of many critical edge distances defined in ACI 318-11, section D.8.6, for various types of post-installed anchors (ACI 2011a).

$\Psi_{ec,N}$ = a modification factor for anchor bolt groups loaded eccentrically in tension. This applies when two anchor bolts are at the outside of the column, when those bolts are not loaded equally. For these shear panels, the two anchor bolts would always be located the same distance from the face of the column, d_c, and therefore should be loaded equally, so that this term equals 1.0.

N_b = the basic concrete breakout strength of a single anchor in tension in cracked concrete, calculated as

$$N_b = k_c \lambda_a \sqrt{f_c'} h_{ef}^{1.5} \qquad \text{(Eq 11-126)}$$

where

k_c = 24 for cast-in anchors and 17 for post-installed anchors.
λa = a modification factor for lightweight concrete; which shall be 1.0λ for cast-in or undercut anchors; 0.8λ for expansion or adhesive anchors; and 0.6λ for adhesive anchors where bond failure applies (see ACI 2011a, section D.5.5.2).
λ = a modification factor to account for lightweight concrete, equal to 0.85 for sand lightweight concrete and 0.75 for all-lightweight concrete and 1.0 for normal-weight concrete (ACI 2011a, section 8.6.1).
f_c' = the specified concrete compressive strength, in pounds per square inch, psi.

The anchor bolts shall not be installed too close to the edge of a concrete beam or slab, as defined by c_{a2} as this reduces the projected concrete failure area defined by Equation 11-123. If a shear panel is designed with only 2 anchor bolts per column (n_{AB} = 2) the diagonal straps and anchor bolts could be placed to the inside of the anchor stiffener plates. This would allow the shear panel to be placed close to the edge of the beam or slab to which it is anchored.

The anchor recommendations presented here are sufficiently conservative so that the lack of symmetry resulting from only two anchor bolts will not compromise ductile performance. The relatively large distance between the anchor bolts and column faces, d_c, should further reduce asymmetric loading of the anchor. However, if edge distance is not an issue, the use of four anchor bolts per column (n_{AB} = 4) is recommended.

11.14.7 Cast-in anchor bolt breakout strength in shear

The nominal concrete breakout strength in shear, V_{cb}, of a single anchor or V_{cbg} of a group of anchors, shall not exceed the following specifications (ACI 2011a, Appendix D, section E6.2.1).

For one anchor bolt on each side of the column, i.e., n_{AB} = 2:

$$P_{hAB} \leq \varphi_{cv} V_{cb} = \varphi_{cv} \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b \qquad \text{(Eq 11-127)}$$

For two anchor bolts on each side of the column (i.e., n_{AB} = 4):

$$P_{hAB} \leq \frac{2\varphi_{cv}V_{cbg}}{n_{AB}} = \frac{2\left(\varphi_{cv}\frac{A_{Vc}}{A_{Vco}}\psi_{ec,V}\psi_{ed,V}\psi_{c,V}\psi_{h,V}V_b\right)}{n_{AB}} \qquad \text{(Eq 11-128)}$$

where

φ_{cv} = the concrete anchor strength reduction factor for concrete breakout in shear, equal to 0.75, when supplementary reinforcement is present (Condition A) (ACI 2011a, section D.4.3.c).

A_{Vc} = the projected failure area of a single anchor or group of anchors as defined in ACI 318-11, section D.6.2.1 and Figure RD.6.2.1(b) (ACI 2011a). This figure defines this area for various combinations of edge distance, depth of anchor, depth of concrete, and single or groups of anchors. A_{Vc} shall not exceed nA_{Vco}, where n is the number of anchors in the group and equals $N_{AB}/2$. Equation 11-128 assumes the group failure area includes only the bolts on one side of a column (i.e., two bolts) because the large distance between them and the bolts on the other side of the column relative to the bolt effective embedment depth, h_{ef}.

A_{Vco} = the projected concrete failure area of a single anchor in a deep member with edge distance equal to or greater than $1.5c_{a1}$, where

$$A_{Vco} = 4.5c_{a1}^2 \qquad \text{(Eq 11-129)}$$

$\Psi_{ed,V}$ = a modification factor for edge effects for single anchors or anchor groups loaded in shear. This term equals 1.0 if the minimum of the out-of-plane edge distance, c_{a2}, is greater or equal to the 1.5 times the in-plane edge distance, c_{a1}. If c_{a2} is less than $1.5c_{a1}$, then $\Psi_{ed,V}$ is calculated as follows:

$$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} \qquad \text{(Eq 11-130)}$$

$\Psi_{c,V}$ = a modification factor permitted for regions of where analysis indicates no cracking. This factor may be 1.2 for anchors in cracked concrete with reinforcement of a No. 4 bar or greater between the anchor and the edge. It may be 1.4 for anchors in concrete with reinforcement of a No. 4 bar or greater between the anchor and the edge, and with the reinforcement enclosed within stirrups spaced at not more than 4 in. Normally a value of 1.0 should be used, assuming some cracking of the concrete occurs.

$\Psi_{h,V}$ = a modification factor for anchors located in concrete members where the depth of the member, h_a, is less than $1.5c_{a1}$, which shall be computed as follows (but shall not be less than 1.0) (ACI 2011a, Appendix D ("Anchoring to Concrete," section D.6.2.8):

$$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \qquad \text{(Eq 11-131)}$$

$\Psi_{ec,V}$ = a modification factor for anchor bolt groups loaded eccentrically in shear. This applies when two anchor bolts are at the outside of the column and are not loaded equally. If diagonal straps were installed on only one face of the shear panels, two anchor bolts were installed outside each column (i.e., n_{AB} = 4), the eccentric loading of the bolts would equal the depth of the panel divided by 2 (b_c/2).

V_b = the basic concrete breakout strength of a single anchor in shear in cracked concrete, which shall equal the smaller of Equation 11-132 and 11-133:

$$V_b = \left[7\left(\frac{l_e}{d_{AB}}\right)^{0.2}\sqrt{d_{AB}}\right]\lambda_a\sqrt{f_c'}c_{a1}^{1.5} \qquad \text{(Eq 11-132)}$$

l_e = the load-bearing length of the anchor for shear; and equals the length of the anchor bolt, h_{ef}, for anchors with constant stiffness over their full length; equals twice the bolt diameter (d_{AB}) for torque-controlled expansion anchors with a distance sleeve separated from expansion sleeve; and shall not exceed eight times the diameter of the anchor bolt, d_{AB}.

$$V_b = 9\lambda_a\sqrt{f_c'}c_{a1}^{1.5} \qquad \text{(Eq 11-133)}$$

11.14.8 Through-bolt floor diaphragm evaluation

Panel anchors may be attached to intermediate floor diaphragms or roof diaphragms using bolts installed through holes cast or drilled through the floor or roof diaphragms. Shear reinforcing shall be installed through the area between the anchors above one column and below the column of the story above. Positive and negative longitudinal reinforcing steel shall be installed through the joint area between the column anchors of the panels above and below. The shear and moment capacity of the beam or slab must be evaluated to ensure they can resist effects of the applied tensile force per bolt, $P_{tAB,}$ and applied shear force per bolt, $P_{hAB.}$ The shear design capacity, ϕV_n, in the region of the panel anchors shall exceed the applied shear forces expressed as follows:

$$\emptyset V_n \geq P_{tAB} \frac{n_{AB}}{2} + V_u \quad \text{(Eq 11-134)}$$

where

ϕ = the shear strength reduction factor, equal to 0.75 (ACI 2011a, section 9.3.2.3)
V_n = the nominal shear strength determined based on guidance in ACI 318-11 (ACI 2011a, section 11.1.1, Eq 11-2).
V_u = the factored shear force applied to the beam or slab section between the column anchors from the floor or roof gravity load and vertical seismic forces.

The flexural design capacity, ϕM_n, in the region of the panel anchors shall exceed the applied moment, M_u, expressed as follows:

$$\emptyset M_n \geq M_u = P_{tAB} \frac{n_{AB}}{2} (d_c + h_c + W_A) + M_{u1} \quad \text{(Eq 11-135)}$$

where

ϕ = the flexure strength reduction factor, equal to 0.90 (ACI 2011a, section 9.3.2.1)
M_n = the nominal flexure strength determined based on guidance in ACI 318-11 (ACI 2011a, Chapter 10).
M_{u1} = the factored moment applied to the beam or slab section between the column anchors from the floor or roof gravity load and vertical seismic forces.

Through-bolt anchors must also meet the requirements for concrete breakout strength in shear expressed by Equations 11-127 and 11-128.

12 Seismic Design Example

An example problem is presented here to demonstrate the design process presented in Chapter 11. Shear panels will be designed in the short direction of the building only to illustrate the design process. In an actual building the lateral-load-resisting system must be designed in both directions. This example is a U.S. Army barracks building of a type that will be designed for construction at Fort Lewis, located between Tacoma and Olympia, WA. This building is similar to a Corps of Engineers Prototype 3 Story Steel Stud Framed Barracks Building for Seismic Zones 0 – 2 (Matsen Ford Design 1997). A three-dimensional view of one end of this building is shown in Figure A-1 in Appendix A of this report. The reader is referred to tabular data and equations presented in Chapter 11 of this report. When needed, ASCE/SEI 7-10 (ASCE 2010) and other guidance is referenced.

12.1 Risk category

The barracks building is a Risk Category II structure (ASCE/SEI 7-10, Table 1.5-1).

12.2 Importance factors

This risk category gives the barracks building a Seismic Importance Factor, I_e, of 1.0 (see Table 12-1).

12.3 Ground motion definition

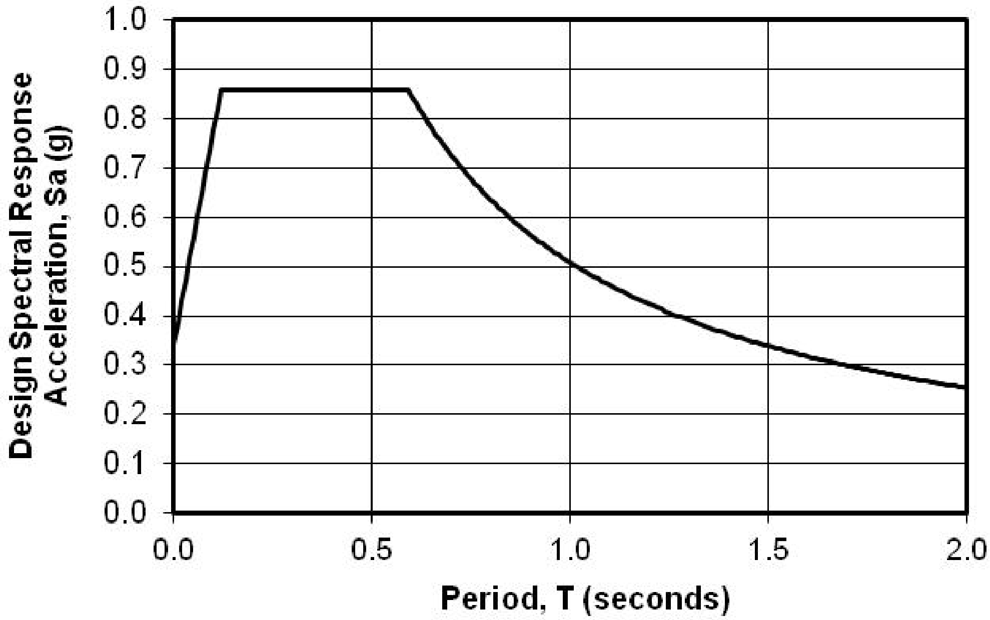
The maximum considered earthquake ground motions were determined by searching the U.S. Geological Survey website (http://geohazards.usgs.gov/designmaps/us/application.php) for Fort Lewis, WA, using ASCE/SEI 7-10 as the building code reference document, Site Soil Classification D, and Risk Category I/II/II. Based on this input, the site provided a spectral response acceleration for short periods, S_s, of 1.287 g and the spectral response acceleration for 1 second, S_1, of 0.508 g. Table 12-1 summarizes these values. The soil conditions were unknown, so a reasonable worst-case site classification of D was used. Values of site coefficients, F_a and F_v, were obtained from Table 11-2 and Table 11-3, and are shown in Table 12-1. Values for the maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at 1 second, S_{M1}, adjusted for site class effects, were calculated using Equations 11-1 and 11-2, and are shown in Table 12-1. Design earthquake spectral response acceleration at short periods, S_{DS}, and 1 second period, S_{D1}, were calculated using Equations 11-3 and 11-4, and are shown in Table 12-1. The long-period transition period, T_L, was determined using the USGS website noted above, and has a value of 6 seconds.

Table 12-1. Earthquake ground motion definition summary for Fort Lewis.

Risk Category	II
Seismic importance factor, I_e	1.0
Short period spectral response acceleration, S_S	1.287 g
1 second spectral response acceleration, S_1	0.508 g
Site classification	D
Site coefficient, F_a	1.0
Site coefficient, F_v	1.5
Adjusted short period spectral response acceleration, S_{MS}	1.287 g
Adjusted 1 second spectral response acceleration, S_{M1}	0.762 g
Design short period spectral response acceleration, S_{DS}	0.858 g
Design short period spectral response acceleration, S_{D1}	0.508 g
T_0	0.118 seconds
T_S	0.592 seconds
Long-period transition period, T_L	6 seconds
Assumed design spectral response acceleration, S_a	0.858 g
Seismic design category	D
Response modification coefficient, R	4
Deflection amplification factor, C_d	3.5

A design response spectrum was developed from these terms, as described in Chapter 11, using Equations 11-5 through 11-7, and plotted in Figure 12-1. For the natural period of the structure, T, this spectrum defines values of effective acceleration. The natural period of the barracks building, T, will almost certainly fall between T_0 and T_S, defined in Chapter 11, so that the design spectral acceleration, S_a, will equal S_{DS}. Values for T_0 and T_S are shown in Table 12-1. After the building frame is designed, the building natural period will be calculated to ensure that it falls between T_0 and T_S, and corrections will be made if needed.

Figure 12-1. Design response spectrum for Fort Lewis, WA, barracks building.



12.4 Seismic design category

The seismic design category for the barracks building was determined from Table 11-4 or Table 11-5, based on the risk category (II) and values of S_{DS} and S_{D1}. If the tables give different categories, the larger letter is chosen. For the barracks building, the seismic design category is D (see Table 12-1).

12.5 Structural design criteria

The lateral-load-resisting system of the barracks building will be designed with cold-formed steel shear panels with diagonal straps acting as the sole lateral-load-resisting elements. Values of the response modification coefficient, R, and deflection amplification factor, C_d, are taken from Table 11-6 and shown in Table 12-1.

12.6 Structural configuration and redundancy

The diaphragms of the barracks building are reinforced concrete and considered rigid. This building is in Seismic Design Category D, and the first and second floors of this three-story building will resist more than 35% of the base shear. Therefore, the braced frame requirements of ASCE/SEI 7-10, Table 12.3-3, must apply to this building if the redundancy factor, ρ, is to be kept at 1.0 (not increased to 1.3). These requirements will be kept because the removal of an individual brace or connection would not result in more than a 33% reduction in story strength, nor would it cause an extreme torsional irregularity (Type 1b defined in ASCE/SEI 7-10, Table 12.3-1). These requirements will be satisfied because the design concept for the barracks building will permit the distribution of multiple shear panels throughout the plan of the building (see Figure 12-2). Therefore, the redundancy factor was set to 1.0.

12.7 Barracks building load combinations and calculations

The effects of gravity load (dead, live, and snow) and seismic forces shall be combined as defined by Equations 11-14 and 11-15. As explained in Chapter 11, the total lateral force that must be resisted by the shear panel diagonal straps is simply defined by ρQ_E in these equations. In the case of the barracks building, this becomes Q_E because ρ equals 1.0., and the shear panel dimensions and diagonal strap design are based on the forces defined by this term.

The barracks building will be designed to act independently in the two orthogonal directions. Figure 12-2 and Figure 12-3 show schematic drawings of the barracks building. Figure 12-2 shows the plan view and long-direction elevation. Figure 12-3 shows the short-direction elevation of the building.

Figure 12-2. Barracks long-direction elevation and plan views.

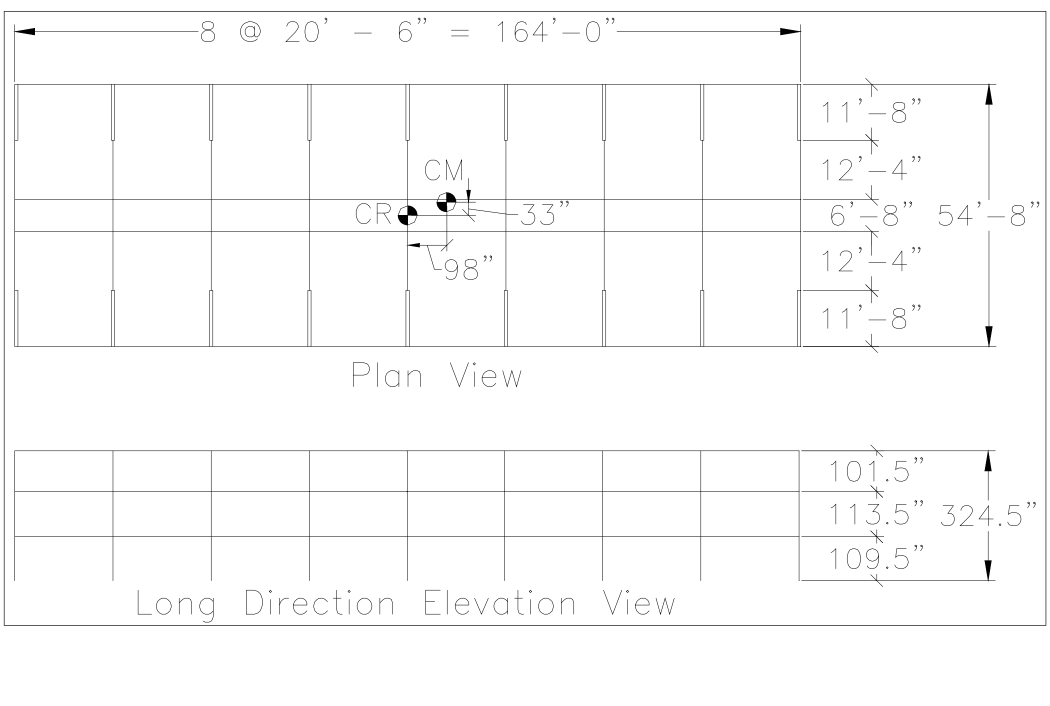


Figure 12-3. Barracks building short-direction elevation and plan views.

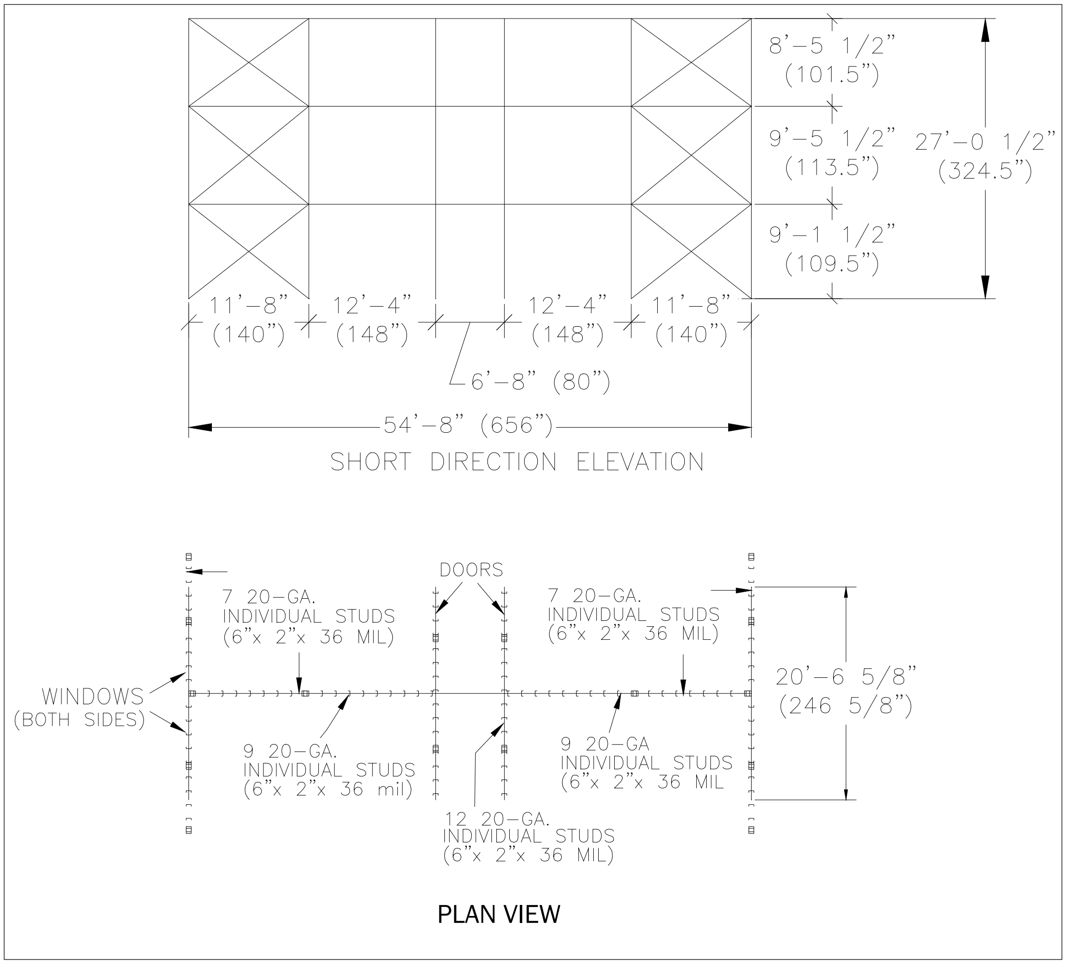


Table 12-2 and Table 12-3 summarize the weight calculations made for the entire building using a spreadsheet. These weights include roof and floor dead load (20 and 45 psf, respectively); exterior wall weight (10 psf); interior wall weight (10 psf); brick veneer weight (40 psf); and room and corridor live load (40 and 80 psf, respectively) (Matsen Ford Designs 1997, prototype drawings-Sheet C-1). The brick veneer is self-supporting for gravity loads, and for vertical and in-plane lateral seismic forces. The building lateral-load-resisting system (shear panels) resists out-of-plane lateral seismic forces from the brick veneer weight. Therefore, the short-direction shear panels resist the out-of-plane long-direction brick veneer lateral seismic forces.

Table 12-2. Barracks building, weight calculations.

Floor Level	Roof and Floor Dead Load (psf)	Floor Length (ft)	Floor Width (ft)	Floor Area (ft^2)	Total Floor Dead Load D (kips)	Story Height (ft)	Exterior Walls (psf)	Total Exterior Walls EW (kips)	Interior Walls (psf)	Total Interior Walls IW (kips)	Total Dead Load D_T= D+EW+IW (kips)	Self Supporting for gravity Brick Veneer (psf)	Long Direction Brick Veneer B_L (kips)	Short Direction Brick Veneer B_S (kips)	Short Direction Seismic Weight W_S (k-mass)
Roof 3rd	20	164	54.7	8988	180	4.2	10	19	10	30	228	40	56	18	284
2nd	45	164	54.7	8988	404	9.0	10	39	10	64	507	40	118	39	625
1st	45	164	54.7	8988	404	9.3	10	41	10	66	511	40	122	41	634
															1543

Table 12-3. Barracks building, additional load calculations.

Floor Level	Room Live Load (psf)	Room Area (ft^2)	Corridor Live Load (psf)	Corridor Area (ft^2)	Total Floor Live Load L (kips)	Sloped Roof Snow Load p_s (psf)	Roof Area (ft^2)	Total Roof Snow Load S (kips)
Roof 3rd	0	7892	0	1096	0	10.1	8988	91
2nd	40	7892	80	1096	403	0	0	0
1st	40	7892	80	1096	403	0	0	0

The short-direction shear panels will be placed at every bay (20 ft, 6-5/8 in. spacing) of the building, as shown in Figure 12-2, for a total of nine short-direction frames. A trial shear panel configuration will be assumed in which two shear panels are placed at every frame, as shown in Figure 12-3. That figure shows two shear panels placed against the perpendicular outside walls of the building. Shear panels will be located at each floor level, with decreasing capacity at the higher floor levels.

The ground snow load, p_g, for Fort Lewis is 20 psf (ASCE/SEI 7-10, Chapter 7 and Chapter 7 commentary). Based on ASCE/SEI 7-10, the flat-roof snow load, p_f, is calculated as follows (ASCE/SEI 7-10, Eq 7.3-1):

$$p_f = 0.7C_eC_tI_sp_g = 0.7(0.9)(1.0)(1.0)(20psf) = 12.6psf \quad \text{(Eq 12-1)}$$

where

C_e = the exposure factor (ASCE/SEI 7-10, Table 7-2), which for a terrain category C, fully exposed roof is 0.9
C_t = the thermal factor (ASCE/SEI 7-10, Table 7-3), which is taken as 1.0
I_s = the snow importance factor (ASCE/SEI 7-10, Table 1.5-2), which for Risk Category II of the barracks building is 1.0.

The sloped-roof snow load, p_s, is calculated as follows (ASCE/SEI 7-10, Eq 7.4-1):

$$p_s = C_s p_f = (0.80)(12.6psf) = 10.1psf \qquad \text{(Eq 12-2)}$$

where

C_s = the roof slope factor (ASCE/SEI 7-10, Figure 7.2b), which is 0.8 for the barracks building with a cold unobstructed slippery surface and a 5/12 roof slope.

The calculation of the snow load, S, is summarized in Table 12-3.

12.8 Earthquake force definition

Seismic forces are now defined based on the equivalent lateral force procedure (see section 11.9). The seismic base shear, V, in the direction of the shear walls is given by Equation 12-3 (previously Equation 11-20):

$$V = C_s W \qquad \text{(Eq 12-3)}$$

The seismic response coefficient, C_s (Equation 11-21), is calculated with the variables given in Table 12-1, which becomes:

$$C_s = \frac{S_{DS}}{R/I_e} = \frac{0.858\,g}{4/1.0} = 0.215g \qquad \text{(Eq 12-4)}$$

The value of C_s need not exceed the following (using Equation 11-22 and 11-23):

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.508g}{0.24(4/1.0)} = 0.536g \text{ for } T \le T_L \qquad \text{(Eq 12-5)}$$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} = \frac{(0.508g)(6)}{(0.24)^2(\frac{4}{1.0})} = 13.6\ g \text{ for } T > T_L \qquad \text{(Eq 12-6)}$$

but shall not be less than (using Equation 11-24):

$$C_s = 0.044S_{DS}I_e = 0.044(0.858g)(1.0) = 0.038g \qquad \text{(Eq 12-7)}$$

The approximate fundamental period, T_a, in seconds is calculated using the following equation (based on Equation 11-26):

$$T_a = C_t h_n^x = (0.020)(27)^{0.75} = 0.24\, seconds \qquad \text{(Eq 12-8)}$$

where

C_t = 0.020 for cold-formed steel shear panels with diagonal straps
h_n = the height, which is 27 ft to the top of the shear walls for the barracks building.

This approximate period, T_a, was used for the fundamental period, T, in Equations 12-5 and 12-6 without correction.

12.9 Short-direction earthquake force definition

The effective seismic weight, W, including roof and floor dead load, exterior walls, and brick veneer perpendicular to the direction under consideration were calculated from the loads presented in Table 12-2 as follows:[19]

$$W = D_T + B = D + EW + IW + B \qquad \text{(Eq 12-9)}$$

For the short direction of the building this weight, W_S, becomes:

$$W_s = D_T + B_L = D + EW + IW + B_L \qquad \text{(Eq 12-10)}$$

where

D_T = the total dead load
B = the brick veneer weight
D = the floor and roof dead load
EW = the exterior wall weight
IW = the interior wall weight

[19] Based on ASCE/SEI 7-10, Section 12.7.2, the barracks building is not used for storage, so no live load is included in the effective seismic weight; permanent equipment is not used; and the flat snow load does not exceed 30 psf, so snow loads are not included.

B_L = the brick veneer weight in the long direction of the building carried by the shear panels in the short direction of the building.

The cumulative total weight in the short direction of the building, W_S, at the first floor is equal to 1,543 kips, as shown in Table 12-2.

The base shear in the short direction of the building, V_S, is now calculated from Equation 11-20:

$$V_S = C_S W_S = (0.215g)(1{,}543kips) = 331kips \qquad \text{(Eq 12-11)}$$

The vertical distribution of lateral seismic forces in the short direction, F_{xS}, induced at any level shall be determined using Equation 11-27. These values are determined based on the vertical distribution factor in the short direction, C_{vxS}, calculated in Equation 11-28. Values for W_{xS}, h_x, w_i, and h_i used in Equation 11-28 are given in Table 12-4.

The short-direction lateral seismic forces, F_{xS}, shown in Table 12-4 are the lateral force per frame in the short direction. There are nine frames in the short direction, n_S[20], so that lateral force per frame is calculated as follows:

$$F_{xS} = \frac{C_{vxS} V_S}{n_S} \qquad \text{(Eq 12-12)}$$

Table 12-4. Short-direction lateral seismic force calculations for barracks building.

Panel Level	Short Direction Total Weight W_S (k-mass)	Seismic Response Coefficient C_s (g)	Short Dir Base Shear V_S (kips)	Height at Floor Level h_{xS} or h_{xL} (ft)	Short Dir Vertical Distribution Factor C_{vxS}	Number Frames in Short Dir n_S	Short Dir Lateral Seismic Force/frame F_{xS} (kips)	Accidental Torsion M_{tax} (kip-ft)	Max. Add Shear due to Acc Torsion Q_{sic} (kips)	Short Dir Seismic Story Shear V_{xS} (kips)
Roof										
3rd	284			27.0	0.306	9	11.3	1074	2.6	13.9
Cumulative	284									
2nd	625			18.6	0.463	9	17.0	1627	4.0	34.9
Cumulative	909									
1st	634			9.1	0.230	9	8.5	809	2.0	45.3
Cumulative	1543	0.215	331							

[20] The symbol for the number of frames in the short direction, n_S, must not be confused with the number of faces with diagonal straps on a given shear panel, n_s.

The barracks building floor is a metal deck filled with concrete so that the diaphragm is rigid (ASCE/SEI 7-10, section 12.3.1.2). The building is very regular in plan, so the center of rigidity, C_R, in both directions should be at the center of the building. The accidental torsion is accounted for by off-setting the center of mass, C_M, 5% in both directions in plan at each floor level (see Figure 12-2). The total mass at each floor level in each direction (long and short) is multiplied by the 5% of the building dimension in that direction to calculate the accidental torsional moment, M_{ta}, at each floor level. As were the lateral seismic forces, the accidental torsional moments, M_{tax}, were distributed along the floors of the building according to the vertical distribution factor given in Equation 11-28, which is expressed as follows:

$$M_{tax} = 0.05[V_S C_{vxS}(FloorLength) + V_L C_{vxL}(FloorWidth)] \qquad \text{(Eq 12-13)}$$

where

C_{vxL} = vertical distribution factor in the long direction
V_L = the base shear in the long direction.

Table 12-4 gives values for accidental torsional moments, M_{tax}, at each floor level.

The torsional resistance, M_{tr} (see Equation 11-32), is proportional to the square of the distance from the center of resistance to the plane of each panel. The torsional resistance is also proportional to the lateral stiffness of each panel. Therefore, because the barracks building is very long in one direction, the shear panels in the short direction near the ends of the building will dominate the torsional resistance. For this example, it will be assumed that all torsional resistance comes from the shear panels in the short direction. The torsional resistance from all shear panels, M_{tr}, in the short direction can be expressed as follows (based on Equation 11-32):

$$M_{tr} = \sum_{i=1}^{n} \rho_i^2 k_{si}\theta = 4[(20.5')^2 + (2x20.5')^2 + (3x20.5')^2 + (4x20.5')^2]k_{si}\theta = 4(20.5')^2(30)k_{si}\theta \qquad \text{(Eq 12-14)}$$

The shear panel farthest from the center of rigidity provides the greatest torsional resistance. However, the end panels in the short direction against the exterior walls will not be loaded as heavily as the panels one bay in from the end because the end panels have only half the tributary area as the panel one bay in. Therefore, the panels which are one bay in from the end will be the most critically loaded because of lateral loads in the short axis and the full width of that bay, and because of its large contribution to torsional resistance. The torsional resistance of the two shear panels that make up the critical short-direction frame, M_{trc}, may be expressed as follows:

$$M_{trc} = \sum_{i=1}^{n} \rho_i^2 k_{si}\theta = 2[(3x20.5)^2]k_{si}\theta = 2(20.5')^2(9)k_{si}\theta \qquad \text{(Eq 12-15)}$$

Equation 12-15 shows that the critical short-direction frame provides 3/20 of the total building torsional resistance (Equation 12-15 divided by Equation 12-14). This ratio can be used to calculate the applied torsion in the critical frame by equating the total accidental torsion, M_{ta}, and torsional resistance from all shear panels, M_{tr}, as follows:

$$M_{trc} = \frac{M_{trc}}{M_{tr}} M_{ta} = \frac{3}{20} M_{ta} \qquad \text{(Eq 12-16)}$$

The additional shear force due to accidental torsion for the critical frame is now calculated by solving Equation 11-30 for Q_{sic}, as follows:

$$Q_{sic} = \frac{M_{trc}}{\rho_c} \qquad \text{(Eq 12-17)}$$

Values of this additional shear force are given in Table 12-4 for each floor level.

Values of seismic story shear in the short direction, V_{xS}, are calculated by modifying Equation 11-29 to include the effects of accidental torsion as follows:

$$V_{xS} = \sum_{i=x}^{n} (F_i + Q_{sic}) \qquad \text{(Eq 12-18)}$$

12.10 Long-direction earthquake force definition

The same process is repeated for the definition of earthquake forces in the long direction of the building. These results are summarized in Table 12-5. The effects from accidental torsion are not added to the frames in the long direction of the building. The frames in the short direction near the ends of the building are much more effective in resisting torsion than those in the long direction, due to their greater distance from the center of rotation (see Equation 11-32).

Table 12-5. Long-direction lateral seismic force calculations for the barracks building.

Panel Level	Long Direction Total Weight W_L (k-mass)	Seismic Response Coefficient C_s (g)	Long Dir Base Shear V_L (kips)	Height at Floor Level h_{xS} or h_{xL} (ft)	Vertical Distribution Factor C_{vxL}	Number Frames in Long Dir n_L	Long Dir Lateral Seismic Force/frame F_{xL} (kips)	Long Dir Seismic Story Shear V_{xL} (kips)
Roof								
3rd	247			27.0	0.305	2	44.0	44.0
Cumulative	247							
2nd	547			18.6	0.464	2	67.0	111.1
Cumulative	794							
1st	552			9.1	0.230	2	33.2	144.3
Cumulative	1345	0.215	289					

12.11 Diagonal strap design

From the values of seismic story shear, V_x ($V_x + Q_{si}$ in Equation 11-37), the shear panels are configured and the diagonal straps designed according to Equation 11-37. Values of the shear panel design strength, $\phi_t Q_{sy}$, are given in Table 12-6. Two identical shear panels are designed to resist the applied story shear in the short direction, V_{xS}, per shear panel ($V_x + Q_{si}$ in Equation 11-37), and these are defined in Table 12-6. Trial shear panel dimensions and diagonal strap sizes for each floor level are defined so that the design strength, $\phi_t Q_{sy}$, exceeds the applied story shear ($V_x + Q_{si}$) per shear panel using the spreadsheet program that models Equation 11-37. Table 12-6 shows trial shear panel configurations that meet this requirement for each floor of the critical frame in the barracks building example. All diagonal straps require ASTM A1003/A1003M, Type H, Grade 33 or Grade 50 steel (ST33H or ST50H) (ASTM 2013b). Panel dimensions are based on the dimensions given for shear wall Type SW-3 (Interior Load-Bearing Walls) of the barracks building drawings (Matsen Ford Design 1997, Sheet S-6). Table 12-6, and all similar panel design tables that follow, was generated by the *Excel Cold-Formed Steel Seismic Design Tool* (Wilcoski 2014). Users of this design tool should design shear panels by changing the cells with white background; cells with gray background are calculated based on the Chapter 11 design recommendations and should not be changed by the user.

Table 12-6. Diagonal strap design in the short direction.

Column Type / Figure No.	Panel Width W (in.)	Panel Height H (in.)	Strap Faces n_s (#)	Strap Width b_s (in.)	(ga)	Strap Design Thickness t_s (in.)	Strap Initial Lat Stiffness k_s (k/in)	Yield Stress of Strap F_{sy} (ksi)	Strap Lat Yield Capacity Q_{sy} (kips)	Design Shear Strength $\phi_t Q_{sy}$ (kips)	Lat Defl at Strap Yielding δ_{sy} (in.)	Applied Story Shear V_{xS} (kips)	Elastic Lateral Defl δ_{xe} (in.)	Defl Amp Factor C_d	Seismic Import Factor I_e	Design Story Drifts $\Delta=\delta_x$ (in.)	Stability Coeff θ	Allow Story Drifts Δ_a (in.)
3rd Floor/12-4	132	101.5	1	4	14	0.0716	31	33	7.5	6.7	0.239	6.94	0.221	3.5	1.0	0.77	0.0009	2.03
3rd Floor/12-7	132	101.5	1	4	16	0.0568	25	50	9.0	8.1	0.362	6.94	0.279	3.5	1.0	0.98	0.0011	2.03
3rd Floor/12-8	132	101.5	2	4	18	0.0453	40	33	9.5	8.5	0.239	6.94	0.175	3.5	1.0	0.61	0.0007	2.03
3rd Floor/12-9*	132	101.5	1	4	16	0.0568	25	50	9.0	8.1	0.362	6.94	0.279	3.5	1.0	0.98	0.0011	2.03
2nd Floor/12-10	140	113.5	2	6	18	0.0453	53	50	21.1	19.0	0.400	17.43	0.331	3.5	1.0	1.16	0.0029	2.27
2nd Floor/12-11*	140	113.5	2	6	18	0.0453	53	50	21.1	19.0	0.400	17.43	0.331	3.5	1.0	1.16	0.0029	2.27
2nd Floor/12-12	140	113.5	2	6	18	0.0453	53	50	21.1	19.0	0.400	17.43	0.331	3.5	1.0	1.16	0.0029	2.27
1st Floor/12-13*	140	109.5	2	6	16	0.0568	69	50	26.9	24.2	0.389	22.65	0.328	3.5	1.0	1.15	0.0045	2.19
1st Floor/12-14	140	109.5	2	6	16	0.0568	69	50	26.9	24.2	0.389	22.65	0.328	3.5	1.0	1.15	0.0045	2.19
1st Floor/12-15	140	109.5	2	4	12	0.1021	83	50	32.2	29.0	0.389	22.65	0.274	3.5	1.0	0.96	0.0038	2.19

*Asterisk designates selected panels.

The diagonal straps are the sole lateral-load-resisting element, and as such they determine the story drifts. The elastic deflections, δ_{xe}, at each floor level are calculated as follows:

$$\delta_{xe} = \frac{\delta_{sy}}{Q_{sy}}\frac{V_{xS}}{n_S} \quad \text{(Eq 12-19)}$$

where δ_{sy} is the lateral deflection at diagonal strap yielding given by

$$\delta_{sy} = \frac{F_{sy}}{E}\left(\frac{H^2+W^2}{W}\right) \quad \text{(Eq 12-20)}$$

Values of δ_{xe} are given in Table 12-6 for the trial diagonal straps at each floor level in the short direction of the building. The design story drifts, Δ, are the differences in deflection at the center of mass at the top and bottom of the story under consideration. These deflections are calculated from the elastic deflection, δ_{xe}, as follows (from Equation 11-34):

$$\Delta = \delta_x = \frac{C_d \delta_{xe}}{I_e} \quad \text{(Eq 12-21)}$$

where

C_d = the deflection amplification factor given in Table 12-1 (3.5 for diagonal-strap panels)

I_e = the seismic importance factor given in Table 12-1 (1.0 for the barracks building).

Values for the design story drifts are given in Table 12-6.

Increases in design story drift, Δ related to P-delta effects are now evaluated. P-delta effects do not need to be considered if the stability coefficient, θ, is equal to or less than 0.10. The stability coefficient, θ, is defined in Equation 11-35 and values are given in Table 12-6. These values are well below 0.10, so design story drifts do not need to be increased. Values of design story drifts, Δ, must be less than the allowable story drifts, Δ_a, given in Table 11-7. For the barracks building this may be expressed as follows (from Table 11-7):

$$\Delta_a = 0.020H \qquad \text{(Eq 12-22)}$$

Values of design story drift, Δ, and allowable story drift, Δ_a, are given in Table 12-6 for each floor level for the trial panels in the short direction of the barracks building. The values in Table 12-6 show that design story drifts fall below allowable drifts. Therefore, these trial sizes meet the drift requirements.

12.12 Column design

Columns are either built up from studs (Panel A or C configuration) or are structural tubes (Panel D). The columns built up with cold-formed steel studs must have the studs oriented to form a closed cross-section as shown on the drawings for test panels A3 and C1 in Appendix A. Individual studs must be welded to each other with a weld thickness equal to the thickness of the studs. The welds are intermittent, with a length and spacing that will ensure composite behavior of the columns.

Structural tubing columns consist of a single tube that is a closed section by itself. This column will provide greater moment resistance because of the heavier anchorage detail, and will provide a greater degree of structural redundancy and widening of the shear panel hysteretic performance.

12.12.1 Column applied loads

Equation 11-18 expresses total load applied to the entire building in the short direction, where the effects of gravity load and seismic forces are additive and diagonal strap overstrength is accounted for. This equation can be expressed in terms of the total dead load, D_T; live load, L; and snow load, S; (given in Table 12-7) as follows:

$$(1.2 + 0.2S_{DS})D_T + L + 0.2S + \Omega_0 Q_E \qquad \text{(Eq 12-23)}$$

The loads expressed in Equation 12-23 are now divided between the number of frames that make up the short-direction lateral-load-resisting system. The building has a total of nine such frames. The loads are distributed based on the tributary area of each frame. Because the end bays have only half the tributary area, the loads are divided by the number of frames minus one, or also stated as the number of bays shown in Table 12-7. The vertical-load-resisting members are the shear panel columns and individual studs, and these are distributed fairly uniformly in plan throughout the building. It is assumed that vertical loads are distributed to these studs in proportion to their cross-sectional area because of the uniform distribution of columns and individual studs in throughout the building in plan. (Normally, gravity loads would be distributed based on tributary area.)

Table 12-7. Gravity load calculations.

Panel Level	S_{DS} (g)	Total Dead Load D_T=D+EW+IW (kips)	Total Floor Live Load, L (kips)	20% of Roof Snow Load, 0.2S (kips)	Short Dir # of bays n_{S-1}	# Studs in Short Dir Col	# Studs in Long Dir Col	Area/ Column Stud A_s (in^2)	Area of Short Dir Col A_{cS} (in^2)	# Ind Stud in Short & Long Dir	Area/ Indiv Stud A_s (in^2)	Area of Indiv & Long Dir Col Studs $A_{I\&cL}$ (in^2)	% Gravity Carried by Short Dir Columns (%)	Gravity /Frame Short Dir GL_{max} (kips)	Gravity /Frame Short Dir GL_{min} (kips)
3rd	0.86	228	0	18	8	8	8	0.430	3.44	68	0.270	21.80	14%	5.6	2.8
Cumulative		228	0	18										5.6	2.8
2nd	0.86	507	403	0	8	12	12	0.680	8.16	68	0.330	30.60	21%	28.9	9.7
Cumulative		736	403	18										34.6	12.6
1st	0.86	511	403	0	8	16	16	0.680	10.88	68	0.330	33.32	25%	34.0	11.5
Cumulative		1247	807	18										68.6	24.0

Table 12-8 shows trial column stud sizes. Each frame has two shear panels in the short direction of the building, and each shear panel has two columns so that the first, second, and third floor columns have four, three, and two studs, respectively. This table also summarizes the size of individual studs for the purpose of determining the area of the column studs relative to all other studs. The individual studs include the interior studs in the shear panels plus all additional individual studs making up the bearing walls in this short-direction frame of the building.

Table 12-8. Trial stud sizes and quantities for one short-direction frame.

Level	Size of Column Studs	Number of Column Studs		Size of Individual Studs	No. of Individual Studs
		Short Direction	Long Direction		
1st Floor	2" x 6" x 43 mil (18 ga)	8	8	2" x 6" x 27 mil (22 ga)	68
2nd Floor	2" x 6" x 68 mil (14 ga)	12	12	2" x 6" x 33 mil (20 ga)	68
first Floor	2" x 6" x 68 mil (14 ga)	16	16	2" x 6" x 33 mil (20 ga)	68

Table 12-7 summarizes the area calculations based on the trial stud sizes. This table shows that 25%, 21%, and 14% of the total gravity load in the tributary area of one short-direction frame is carried by the short-direction shear wall columns. The remaining gravity loads are carried by individual studs and shear panel column studs in the long direction of the building. These gravity loads are summarized in Table 12-7.

The $\Omega_o Q_E$ term in Equation 12-23 accounts for material overstrength in the diagonal straps. The vertical component in the straps will place additional compressive loads in the columns. The total column axial load at the maximum ultimate stress in the diagonal straps, P_{vumax}, is determined from Equation 11-38 and repeated below:

$$P_{vumax} = \frac{GL_{max}}{2} + F_{sumax} n_s b_s t_s \left(\frac{H}{\sqrt{H^2+W^2}}\right) \qquad \text{(Eq 12-24)}$$

Table 12-9 gives values for P_{vumax} for each trial shear wall column at each floor in the short-direction frame of the barracks building.

Table 12-9. Column design for cold-formed steel shear panels— barracks example.

Column Type / Figure No.	Diagonal Strap Ult Stress F_{su} (ksi)	Max Ult Strap Stress F_{sumax} (ksi)	Gravity /Frame Short Dir GL_{max} (kips)	Number Shear Panels /Frame	Max Gravity Load/ Panel GL_{max} (kips)	Column Axial load at Strap Ult P_{vumax} (kips)	Column Yield Stress F_{cy} (ksi)	Column Ultimate Stress F_{cu} (ksi)	Column Design Thickness (ga)	Column Design Thickness t_c (in.)	Number of Studs /Column n	Panel Thickness /Column b_c (in.)	Col Stud Flange Width b_f (in.)	Column Depth h_c (in.)
3rd Floor/12-4	45	68	5.6	2	2.82	13.2	33	45	16	0.0568	2	6.0	2.0	4.0
3rd Floor/12-7	65	81	5.6	2	2.82	12.7	50	65	16	0.0568	2	6.0	2.0	4.0
3rd Floor/12-8	45	68	5.6	2	2.82	16.3	50	65	16	0.0568	2	6.0	2.0	4.0
3rd Floor/12-9*	65	81	5.6	2	2.82	12.7	50	65	16	0.0568	2	6.0	2.0	4.0
2nd Floor/12-10	65	81	34.6	2	17.29	36.4	50	65	14	0.0716	3	6.0	2.0	6.0
2nd Floor/12-11*	65	81	34.6	2	17.29	36.4	50	65	14	0.0716	3	6.0	2.0	6.0
2nd Floor/12-12	65	81	34.6	2	17.29	36.4	50	65	14	0.0716	4	6.0	2.0	8.0
1st Floor/12-13*	65	81	68.6	2	34.28	51.3	50	65	12	0.1021	3	6.0	2.0	6.0
1st Floor/12-14	65	81	68.6	2	34.28	51.3	46	58		0.1875	1	6.0	6.0	6.0
1st Floor/12-15	65	81	68.6	2	34.28	58.0	46	58		0.1875	1	6.0	6.0	6.0

*Asterisk designates selected panels.

12.12.2 Column axial capacity

Table 12-9 also presents trial column configurations defined in terms of their yield stress, F_{cy}; column stud or structural tubing material thickness, t_c; number of studs per column, n; panel thickness, b_c; and column depth, h_c. The panel thickness is the column width in the out-of-plane direction of the panel, and column depth is the column width in the in-plane direction of the panel. Each of the column studs are 6 in. deep with a 2 in. wide flange, b_f. They are welded together to form a closed column section and are oriented so that the stud flanges are parallel to the plane of the shear panels (see test panels A3 and C1 in Appendix A). In this orientation, the column depth, h_c, is the number of studs per column times 2 in. Table 12-10 presents the column capacity calculations. This table gives the column nominal areas, A_c; distance to the extreme fiber, c; in-plane and out-of-plane moments of inertia; and radius of gyration. The column anchors are designed to provide moment connections at their tops and bottom, so that the effective length factor, K, becomes 0.5.

Table 12-10. Column capacity calculations for shear panels — barracks example.

Column Type / Figure No.	Nominal Column Area A_c (in^2)	Distance to Extreme Fiber c (in.)	In-Plane Mom of Inertia I_x (in^4)	In-Plane Radius of Gyration r_y (in.)	Out-of-Plane Mom of Inertia I_y (in^4)	Out-of-Plane Radius of Gyration r_x (in.)	Eff Length Factor K	Elastic Flexural Stress F_e (ksi)	λ_c	Nominal Axial Stress F_{cn} (ksi)	Knockout hole dia d_h (in.)	Flat Width w (in.)	F_{cr} (ksi)	Slenderness factor λ	Eff Width b (in.)	Eff Column Area A_e (in^2)	Column Design Strength P_c (kips)
3rd Floor/12-4	1.14	2.00	3.21	1.68	5.95	2.29	0.5	314	0.32	31.6	1.5	5.77	9.32	1.84	2.15	0.725	19.5
3rd Floor/12-7	1.14	2.00	3.21	1.68	5.95	2.29	0.5	314	0.40	46.8	1.5	5.77	9.32	2.24	1.81	0.687	27.3
3rd Floor/12-8	1.14	2.00	3.21	1.68	5.95	2.29	0.5	314	0.40	46.8	1.5	5.77	9.32	2.24	1.81	0.687	27.3
3rd Floor/12-9*	1.14	2.00	3.21	1.68	5.95	2.29	0.5	314	0.40	46.8	1.5	5.77	9.32	2.24	1.81	0.687	27.3
2nd Floor/12-10	2.15	3.21	10.29	2.19	11.14	2.28	0.5	426	0.34	47.6	1.5	5.71	15.1	1.78	2.18	1.389	56.2
2nd Floor/12-11*	2.15	3.21	10.29	2.19	11.14	2.28	0.5	426	0.34	47.6	1.5	5.71	15.1	1.78	2.18	1.389	56.2
2nd Floor/12-12	2.86	4.00	22.52	2.80	14.86	2.28	0.5	461	0.33	47.8	1.5	5.71	15.1	1.78	2.18	1.851	75.2
1st Floor/12-13*	3.06	3.21	14.46	2.17	15.63	2.26	0.5	451	0.33	47.7	1.5	5.59	32.1	1.22	2.86	2.23	90.3
1st Floor/12-14	4.27	3.00	23.8	2.36	23.8	2.36	0.5	532	0.29	44.4	1.5	5.25	123	0.60	3.75	3.99	150
1st Floor/12-15	4.27	3.00	23.8	2.36	23.8	2.36	0.5	532	0.29	44.4	1.5	5.25	123	0.60	3.75	3.99	150

*Asterisk designates selected panels.

The last two rows in Table 12-9 and Table 12-10 are for panels with columns made up of 6 x 6 x 3/16 in. hollow structural sections (HSS) structural tubing members (panel D2 configuration). The tubing material is ASTM A500/A500M Grade B (ASTM 2013c), with minimum yield stress, F_{cy}, and minimum ultimate stress, F_{cu}, values of 46 ksi and 58 ksi, respectively. Similar to the column studs, it is assumed that 1.5 in. wide holes will be drilled through the faces of the column that are out of plane to the shear panel. These holes are for electrical mechanical tubing (EMT) conduit.

The elastic flexural stress, F_e, shown in Table 12-10 is calculated based on Equation 11-43, and λ_c is calculated based on Equation 11-42. The nominal axial stress, F_{cn}, is then calculated based on either Equation 11-40 or 11-41, depending on the value of λ_c.

The effective areas, A_e, of the columns are calculated according to Equation 11-45. Values of the terms used to define this area are also given in Table 12-10. Finally, the column design strength, P_c, is calculated according to Equation 11-39. Values of P_c are given in Table 12-10 for each trial column. Through an iterative process in the spreadsheet program, trial column configurations were defined where P_c exceeds the column axial load at the maximum ultimate stress in the diagonal straps, P_{vumax}. From these results, the column configurations marked with an asterisk in Table 12-9 and Table 12-10 were selected for the three floor levels.

12.12.3 Column bending and composite behavior

The shear panel anchor recommendations will provide moment resistance at the column ends, especially when no axial load is applied to the columns. The columns built up from studs must be designed to act as a composite cross-section. Table 12-11 gives the intermittent weld length, L (2 in. for each built-up column in Table 12-11), and maximum center-to-center intermittent weld spacing, s_{max}, needed to ensure composite behavior of the columns. This is based on Equation 11-54. Based on the values of s_{max} given in Table 12-11, actual weld spacing is selected that round down to the nearest full inch from the values shown in the table. These welds are made between all studs in the column and begin at both ends of the columns.

Table 12-11. Column intermittent weld design.

Column Type / Figure No.	Max Column Shear V_{cm} (kips)	Area on 1 Side of Crit Weld A (in^2)	Distance to Neutral Axis y (in.)	Mom of Column Area Q (in^3)	Weld Shear/ Length q (k/in)	Intermittent Weld Length L (in.)	Max o.c. Spacing s_{max} (in.)
3rd Floor/12-4	1.0	0.57	1.60	0.9	0.3	2.0	14.3
3rd Floor/12-7	1.6	0.57	1.60	0.9	0.4	2.0	13.6
3rd Floor/12-8	1.6	0.57	1.60	0.9	0.4	2.0	13.6
3rd Floor/12-9*	1.6	0.57	1.60	0.9	0.4	2.0	13.6
2nd Floor/12-10	2.8	1.43	1.61	2.3	0.6	2.0	12.1
2nd Floor/12-11*	2.8	1.43	1.61	2.3	0.6	2.0	12.1
2nd Floor/12-12	5.0	1.43	2.20	3.1	0.7	2.0	11.1
1st Floor/12-13*	4.1	2.04	1.62	3.3	0.9	2.0	11.7
1st Floor/12-14							
1st Floor/12-15							

*Asterisk designates selected panels.

12.12.4 Column combined axial and moment capacity

The combination of axial load and bending was evaluated for each trial shear panel. For each case, interaction values were determined according to Equations 11-55 and 11-56. Table 12-12 shows that the interaction values, I were below 1.0 for all but the columns in the first row, and this was not a selected panel.

12.12.5 Column shear capacity

The column design shear capacity, V_c, was calculated according Equation 11-65 for each trial column. The values are shown in the second column of Table 12-13. These are below the strap maximum estimated ultimate lateral capacity ($P_{humax} = \Omega_o Q_E$), calculated according to Equation 11-17, with values given in the third column of Table 12-13. Therefore, the additional shear capacity from anchors is needed to resist the maximum lateral force, as shown under "Anchor shear capacity," section 12.14.1.

Table 12-12. Column combined axial and moment capacity.

Column Type / Figure No.	Required Compressive Strenth $\bar{P}$ (kips)	Column Fixity (%)	Strap Max Yield Stress F_{symax} (ksi)	Max Est Lat Defl at Strap Yield δ_{symax} (in.)	Applied Moment @δ_{symax} M_a (k-in)	C_{mx}	P_{Ex}	α_x	Effective Section Modulus S_e (in^3)	Column Nominal Moment M_{nx} (k-in)	Column Axial w/F_n=F_y P_{no}	$\bar{P}/\phi_c P_n$	Eq 11-55 Column Interaction I	Eq 11-56 Column Interaction I
3rd Floor/12-4	12.9	50%	66	0.478	19.1	0.85	356	0.96	1.60	52.9	23.9	0.665	1.000	1.017
3rd Floor/12-7	11.8	50%	75	0.543	21.1	0.85	356	0.97	1.60	80.2	34.3	0.432	0.676	0.682
3rd Floor/12-8	16.0	50%	66	0.478	20.6	0.85	356	0.96	1.60	80.2	34.3	0.585	0.826	0.818
3rd Floor/12-9*	11.8	50%	75	0.543	21.1	0.85	356	0.97	1.60	80.2	34.3	0.432	0.676	0.682
2nd Floor/12-10	34.3	50%	75	0.600	62.3	0.85	914	0.96	3.21	160.4	69.5	0.610	0.971	0.990
2nd Floor/12-11*	34.3	50%	75	0.600	62.3	0.85	914	0.96	3.21	160.4	69.5	0.610	0.971	0.990
2nd Floor/12-12	34.3	50%	75	0.600	111.8	0.85	2001	0.98	5.63	281.5	92.6	0.456	0.818	0.854
1st Floor/12-13*	48.7	50%	75	0.584	89.6	0.85	1381	0.96	4.50	225.2	111.3	0.539	0.908	0.933
1st Floor/12-14	48.7	50%	75	0.584	129.2	0.85	2273	0.98	7.93	364.9	183.5	0.323	0.647	0.685
1st Floor/12-15	54.9	50%	75	0.584	132.8	0.85	2273	0.98	7.93	364.9	183.5	0.365	0.699	0.735

*Asterisk designates selected panels.

Table 12-13. Column shear capacity.

Column Type / Figure No.	Column Shear Strength V_c (kips)	Strap Lat Ult Capacity P_{humax} (kips)
3rd Floor/12-4	4.3	15.3
3rd Floor/12-7	6.5	14.6
3rd Floor/12-8	13.0	19.4
3rd Floor/12-9*	6.5	14.6
2nd Floor/12-10	24.5	34.3
2nd Floor/12-11*	24.5	34.3
2nd Floor/12-12	32.6	34.3
1st Floor/12-13*	34.9	43.7
1st Floor/12-14	59.0	43.7
1st Floor/12-15	59.0	52.3

*Asterisk designates selected panels.

12.13 Diagonal strap-to-column connections

This section defines the applied loads and design requirements for diagonal strap connections to the panel columns.

12.13.1 Connection design assumptions and applied loads

Diagonal strap-to-column connections are designed to resist the maximum estimated ultimate force in the strap, P_{sumax}, defined by Equation 11-70. P_{sumax} values are given in the second column of Table 12-14 for each panel.

12.13.2 Screwed fastener connection design

All screws used in this example are #10 self-tapping hex-head screws, which is the largest practical size that will not interfere with drywall installation. The nominal screw diameter, d, for #10 screws is 0.190 in. (AISI 2007b, Table C-E4-1). The following fastener layout recommendations are based on text in section 11.13.2 of this report:

- Minimum distance between centers of fasteners is 3d = 0.57 in.
- For connections subjected to shear forces in only one direction, the minimum distance from centers of fasteners to the edge of a connected part perpendicular to the force is 1.5d = 0.29 in.

The design shear and pull-over per screw, P_s, was calculated for every shear panel screwed connection according to Equations 11-71 through 11-76 and 11-78, based on the thicknesses and strength of the connected members and the dimensions of the screws. Table 12-14 gives the ratio of t_c/t_s and the resulting design shear per screw, as defined by these equations. A screw head diameter, d_w, of 0.402 in.[21] was used for the screws.

[21] This dimension was measured from #10 hex washer head screws (ITW Buildex Part Number 1129000) used in test panels at ERDC-CERL. Measurement was made using a vernier caliper, and the diameter at the base of the washer head was consistently 0.402 in. ± 0.004 in. (10.2 mm ± 0.1 mm).

Table 12-14. Screwed connection design.

Column Type / Figure No.	Max Est Ult Strap Force P_{sumax} (kips)	Nominal Screw Dia d (in.)	Col/Strap Thickness Ratio (t_c/t_s)	Column Tilting Eq 11-72 P_{ns} (kips)	Strap Bearing Eq 11-73 P_{ns} (kips)	Column Bearing Eq 11-74 P_{ns} (kips)	Strap Bearing Eq 11-75 P_{ns} (kips)	Column Bearing Eq 11-76 P_{ns} (kips)	Nominal Shear P_{ns} (kips)	Screw head dia d_w (in.)	Nominal Pull-over P_{nov} (kips)	Manufacturer's Nom Shear P_{ns} (kips)	Min End Dist e (in.)	Design Shear /Screw P_s (kips)	Number Screws /Face n_{screws} (#)
3rd Floor/12-4	19.3	0.19	0.79	1.116	2.479	1.312	2.479	1.312	1.116	0.402	2.913	1.232	0.23	0.558	34.6
3rd Floor/12-7	18.5	0.19	1.00	1.613	2.369	1.895	2.369	1.895	1.613	0.402	2.785	1.232	0.27	0.616	30.0
3rd Floor/12-8	24.4	0.19	1.26	1.613	1.567	1.895	1.567	1.895	1.567	0.402	1.842	1.013	0.33	0.506	24.1
3rd Floor/12-9*	18.5														
2nd Floor/12-10	44.1	0.19	1.58	2.279	1.887	2.387	1.887	2.387	1.887	0.402	2.218	1.013	0.28	0.506	43.6
2nd Floor/12-11*	44.1														
2nd Floor/12-12	44.1	0.19	1.58	2.279	1.887	2.387	1.887	2.387	1.887	0.402	2.218	1.013	0.28	0.506	43.6
1st Floor/12-13*	55.4														
1st Floor/12-14	55.4														
1st Floor/12-15	66.4														

*Asterisk designates selected panels.

Ultimate shear values from the manufacturer's data,[22] P_u, based on the smaller thickness of the members being connected were used to calculate nominal screw shear strength, P_{ns}, according to the following equation:

$$P_{ns} = \frac{P_u}{1.25} \qquad \text{(Eq 12-25)}$$

Equation 12-25 was not included in the Chapter 11 design recommendations because the format of manufacturer's test data was not given and may need to be evaluated on a case-by-case basis, as shown in the present example. Table 12-14 provides values for this nominal shear strength, based on manufacturer's fastener shear strength and Equation 12-25. The design shear per screw, P_s, was calculated according to Equation 11-71 from the minimum of P_{ns} and P_{nov}, including P_{ns} as calculated according to Equation 12-25. Table 12-14 presents P_s based on this overall minimum nominal shear, pull-over strength, or screw shear strength. In this example problem, the manufacturer's fastener shear strength data controls the nominal fastener shear strength for all panel configurations except for the one shown on the first row of Table 12-14.

The minimum end distance for screws in the diagonal straps was defined using Equation 11-77 by setting P_{ns} equal to the minimum shear strength described in the previous paragraph and solving for end distance, e. Values for this minimum end distance are shown in Table 12-14.

22 From ITW Buildex Catalog for #10 fasteners with #3 drill point.

Finally, the number of screws required at each diagonal strap-to-column connection, n_{screws}, was calculated according to Equation 11-80 for each trial panel configuration. These quantities are given in Table 12-14. These values are very large, and the use of larger screws or welded connections should be considered. Still, each of these connections may be constructed within the overlap area of the strap and column, and within the spacing and edge distance requirements given above in this section. The most difficult joints to lay out are those in Figure 12-4 and Figure 12-7, which are based on installing diagonal straps on only one face of the shear panel. These columns are 4 in. wide, and the straps are also 4 in. wide and oriented at an angle based on the width, W, and height, H, of the overall panels given in Table 12-6. Layouts of the fasteners were selected that keep the column critical shear plane as close as possible to the track while maximizing the net area for rupture strength evaluation. The trial layout shown in Figure 12-4 has 6 fasteners at the first row against the track, and 4, 6, 4, 6, 4, and 5 fasteners in the subsequent rows moving up the joint. Figure 12-5 is a close-up drawing of this joint, showing the fastener locations and critical rupture surface (see next section, “Block shear rupture”). These fasteners are spaced at 9/16 in. on center horizontally and 0.375 in. on center vertically, in a staggered pattern. The other diagonal strap-to-column screwed connections in Table 12-14 are laid out in a similar manner, and are shown in Figure 12-7, Figure 12-8, Figure 12-10 and Figure 12-12.

Figure 12-4. Example connection/anchorage detail— first row of Table 12-6 and Table 12-9 through 12-22.

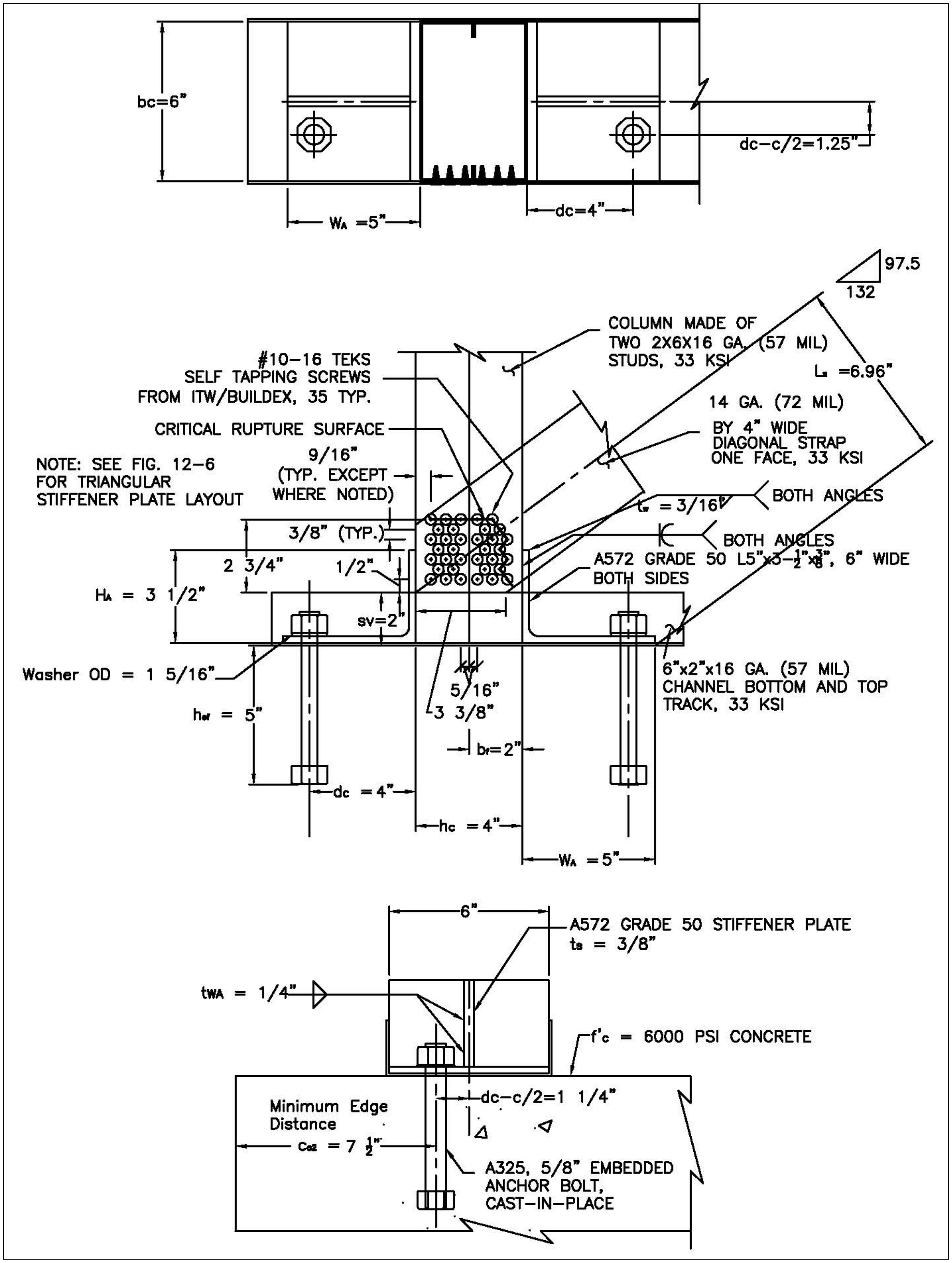


Figure 12-5. Close-up of second-row connection showing fastener locations and critical rupture surface.

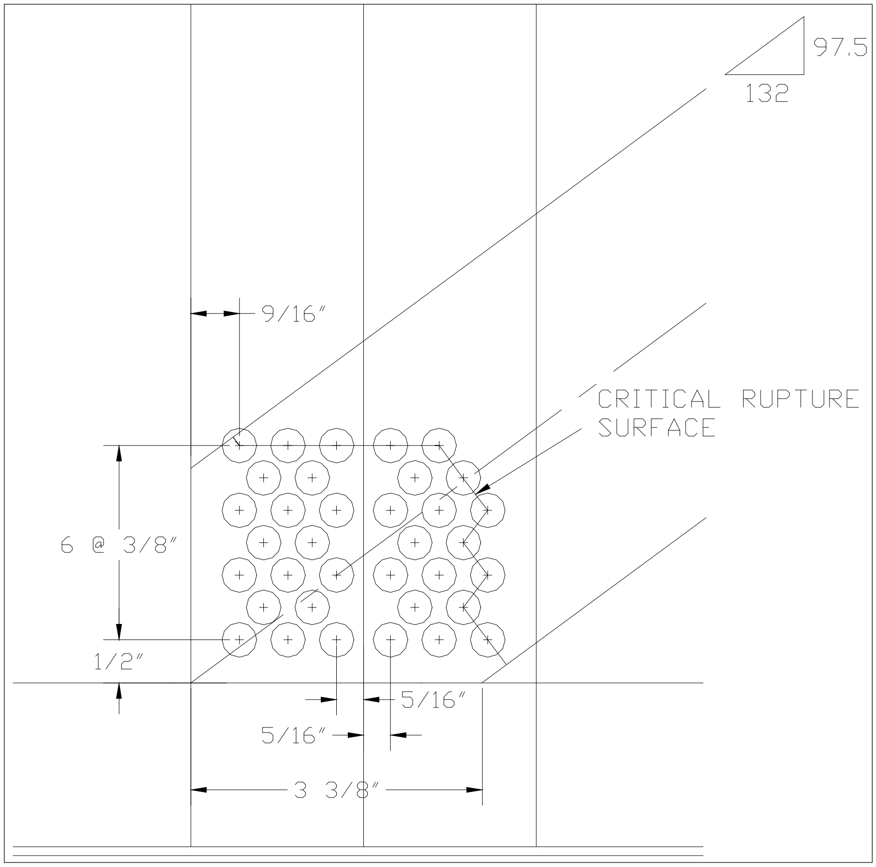


Figure 12-6. Elevation view of the second-row anchor stiffener plates.

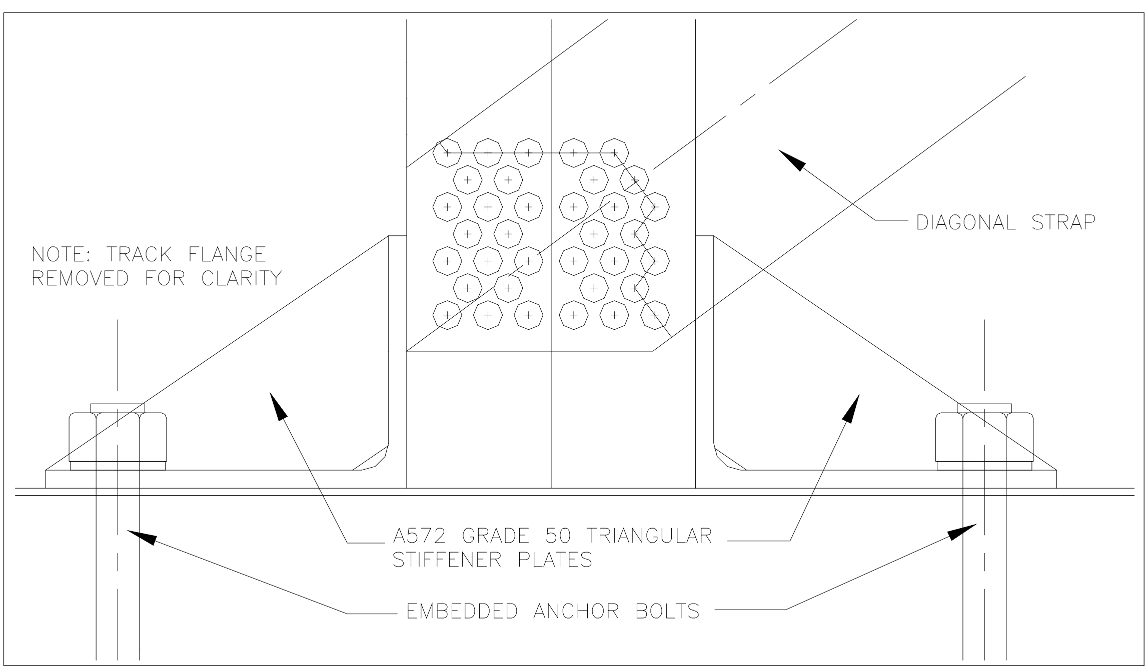


Figure 12-7. Example connection/anchorage detail—second row of Table 12-6 and Table 12-9 through 12-22.

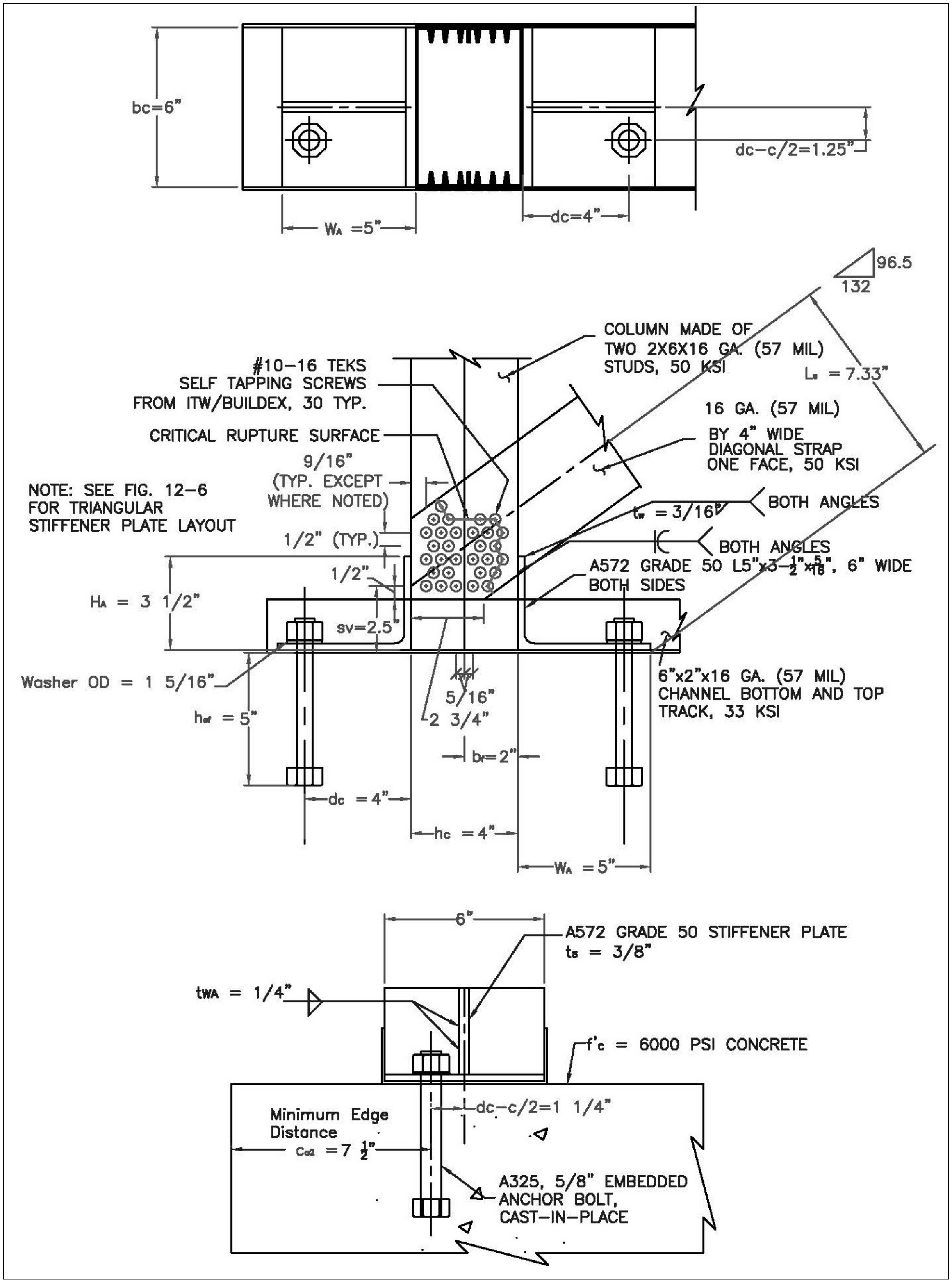
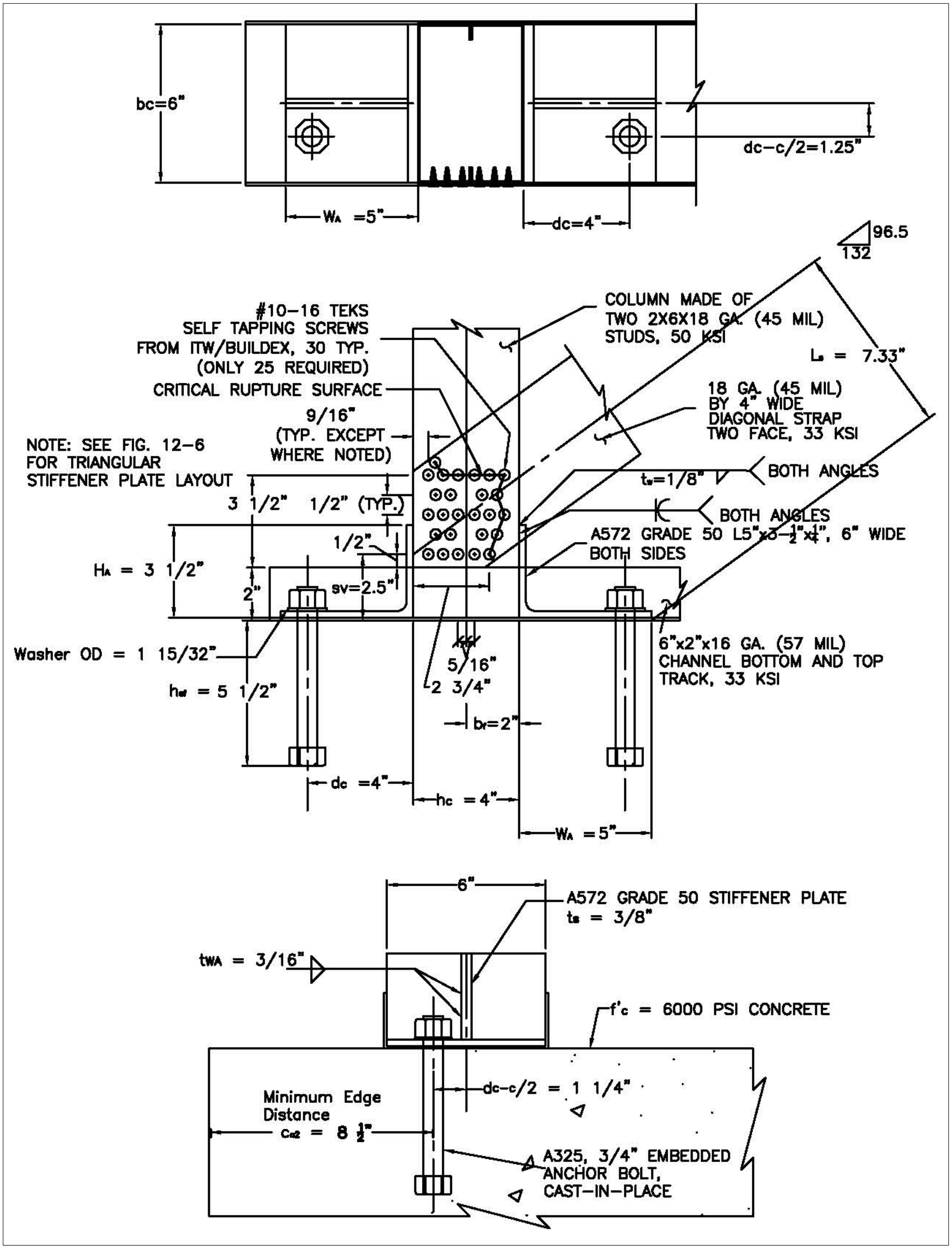


Figure 12-8. Example connection/anchorage detail—3rd row of Table 12-6 and Table 12-9 through 12-22.



12.13.3 Block shear rupture

Figure 12-4, Figure 12-5, Figure 12-7, Figure 12-8, Figure 12-10, and Figure 12-12 show the critical diagonal strap rupture surface for trial screwed connections, for which the rupture strength was calculated. The rupture surface located along the inside edge of the columns and along a horizontal plane will be loaded at approximately a 45-degree angle to the rupture surface. Therefore, the averages of the shear and tension strength (0.8 F_{sy} and 0.8 F_{su}, respectively) are used to modify Equations 11-82 and 11-83 for the shear/tensile strength and tensile strength for the rupture surface loaded at 45 degrees, as shown in Equations 12-26 and 12-27. Values for Equations 12-26 and 12-27 are shown in Table 12-15 for each trial strap-to-column screwed connection.

$$R_n = 0.8F_{sy}A_{gvt} + F_{su}A_{nt} \quad \text{(Eq 12-26)}$$

$$R_n = 0.8F_{su}A_{nvt} + F_{su}A_{nt} \quad \text{(Eq 12-27)}$$

where

A_{gvt} = the gross area subject to shear and tension at 45 degrees
A_{nvt} = the net area subject to shear and tension at 45 degrees
A_{nt} = the net area subject to tension

The design shear rupture is calculated according to Equation 11-81 based on the trial layouts of the fasteners for each diagonal strap-to-column connection. The use of #10 screws resulted in a very large number of screws at each connection, as seen in Table 12-14 and Figure 12-4 through Figure 12-8, Figure 12-10, and Figure 12-12. The screw pattern must stay within the spacing and edge distance limitations presented in section 12.13.2. This design capacity is further modified, as shown in Equation 12-28, for the number of strap faces or straps in each direction used on the shear panels.

$$R = \emptyset_R min(R_n)n_s \quad \text{(Eq 12-28)}$$

When the strap-to-column rupture strength is evaluated based on Equation 12-28, the resistance factor may be increased to 1.0 because of the ASTM minimum material requirement on F_u/F_y (see discussion near Equations 11-84 and 11-85). The strap applied tensile force is defined in Equation 11-85.

None of the trial diagonal strap-to-column screwed connections have a rupture capacity, R, that exceeds the maximum applied load (P_{sy} in Equation 11-85), as shown in Table 12-15. The achieved resistance factor (ϕ_a) may be calculated by combining Equations 11-85 and 12-28, as shown in Equation 12-29:

$$\emptyset_a = \frac{P_{sy}}{min(R_n)n_s} \leq 1.0 \qquad \text{(Eq 12-29)}$$

Table 12-15 shows that all but one of the achieved resistance factors are above 1.0 (Figure 12-12 has the smallest, with a value of 0.926.) The trial screw locations shown in Figure 12-4 through Figure 12-8, Figure 12-10 and Figure 12-12 were defined so as to maximize the achieved resistance factor, ϕ_a, in Equation 12-29.[23] This demonstrates how difficult it is to design ductile screwed connections because the ASTM A 1003/A1003M minimum F_u/F_y ratio is only 1.08 (ASTM 2013b); therefore, all selected connections in this design example are welded.

Table 12-15. Screwed connection rupture strength.

Column Type / Figure No.	Strap Yield Force P_{sy} (kips)	Tension /Shear Gross Area A_{gvt} (in^2)	Tension /Shear Net Area A_{nvt} (in^2)	Tension Net Area A_{nt} (in^2)	Shear Ruputre Eq 12-26 Rn (kips)	Shear Ruputre Eq 12-27 Rn (kips)	Design Rupture Strength R (kips)	Achieved Resistance Factor ϕ_a
3rd Floor/12-4	11.9	0.273	0.191	0.066	10.183	9.852	6.4	1.211
3rd Floor/12-7	13.7	0.213	0.159	0.039	11.073	10.823	7.0	1.264
3rd Floor/12-8	15.1	0.144	0.101	0.051	6.096	5.933	7.7	1.272
3rd Floor/12-9*	13.7							
2nd Floor/12-10	32.7	0.297	0.231	0.046	14.842	14.950	19.3	1.101
2nd Floor/12-11*	32.7							
2nd Floor/12-12	32.7	0.300	0.236	0.087	17.644	17.905	22.9	0.926
1st Floor/12-13*	41.1							
1st Floor/12-14	41.1							
1st Floor/12-15	49.2							

*Asterisk designates selected panels.

[23] Trial screw locations were defined so that they maintain a regular pattern of screws within the overlap area of the diagonal strap and column. The number of screws per connection, n_{screws}, must meet the requirement of Equation 11-80. The overlap area between the diagonal strap and column could have been increased slightly by moving the strap up the column at the connections near the bottom of the columns, but that change would provide limited benefit because the shear rupture surface length would not be increased at all and the only benefit would be to increase the screw spacing by distributing the screws over a larger area. The greater screw spacing may increase the rupture surface area (A_{nt} and A_{nvt} in Equations 12-26 and 12-27) because fewer screws may be located along the rupture surface. However, moving the strap up the column would increase the eccentricity of diagonal strap loading of the anchor (i.e., increase S_v and L_s in Equation 11-114), resulting in larger anchor loads (i.e., increase P_{tAB} in Equation 11-112).

12.13.4 Welded connection design

Figure 12-9, Figure 12-11, and Figure 12-13 through Figure 12-15 show trial layouts of welded diagonal strap-to-column connections. The third-floor weld connection thickness, t, equals the column thickness, t_c (0.0568 in.), while the first- and second-floor welds equal the diagonal strap thickness, t_s. Details on the strap and column sizing are given in Table 12-6 and Table 12-9. The longitudinal welds are along the edges of the diagonal straps. Most of the welds have L/t ratios greater than 25, so Equation 11-87 is used to define the longitudinal weld capacity. Portions of the longitudinal welds that are broken up where they pass over the column stud junctions then have L/t ratios smaller than 25, but Equation 11-87 is still used because it conservatively results in a smaller capacity. The diagonal edges at the end of the diagonal strap are loaded close to 45 degrees so that an average of Equations 11-86 and 11-88 (or an average of Equations 11-87 and 11-88) defines the weld capacity along these diagonal edges. The longitudinal/transverse design shear, P_{LT}, is based on an average of Equations 11-86 and 11-88, and expressed here as Equation 12-30.

$$P_{LT} = \left[\emptyset_{L1}\left(1 - \frac{0.01L}{t}\right) + \emptyset_T\right]\frac{LtF_u}{2} \qquad \text{(Eq 12-30)}$$

The longitudinal/transverse design shear, P_{LT}, based on an average of Equations 11-87 and 11-88, is expressed as follows:

$$P_{LT} = 0.875\emptyset_{LT}tLF_u \qquad \text{(Eq 12-31)}$$

where

ϕ_{LT} = 0.575, which is an average of the resistance factors for longitudinal and transverse loading expressed in Equations 11-87 and 11-88.

The thickness of the welds, based on strap or column thickness, are all less than 0.10 in., except for the one shown in the last row of Table 12-16 (also shown in Figure 12-15), where the strap is 0.102 in. thick. For this case, the strength of the welded connection is calculated based on Equation 11-89. The weld used is a shielded metal arc weld (SMAW) E70XX, with an electrode tensile strength, Fxx, equals 70 ksi. Table 12-16 shows that the capacity of this last welded connection is limited by the weld capacity defined by the conservative assumptions of Equation 11-89, but the AISI

S100-2007 commentary discussion (AISI 2007b, section E2.4) indicates that this limitation is almost certainly overly conservative, and that the capacity should be limited by the those values defined by Equations 11-86 and 12-31. Still, the results reported in Table 12-16 are based on the more conservative results of Equation 11-89.

Table 12-16. Welded connection design strength.

Column Type / Figure No.	Strap Yield Force P_{sy} (kips)	Least Member Thickness t (in)	Longitudinal Weld Length L (in)	Longitudinal Weld Design Strength P_L (kips)	Long/Trans Weld Length L (in)	Long/Trans Weld Design Strength P_{LT} (kips)	Electrode Strength F_{xx} (ksi)	Weld Failure Strength P_w (kips)	Welded Conn Total Capacity $(P_L+P_{LT})n_s$ (kips)
3rd Floor/12-4	11.9								
3rd Floor/12-7	13.7								
3rd Floor/12-8	15.1								
3rd Floor/12-9*	13.7	0.0568	3.24	4.5	5.29	9.8			14.3
2nd Floor/12-10	32.7								
2nd Floor/12-11*	32.7	0.0453	4.20	4.6	8.06	11.9			33.1
2nd Floor/12-12	32.7								
1st Floor/12-13*	41.1	0.0568	4.18	5.8	8.05	15.0			41.5
1st Floor/12-14	41.1	0.0568	3.48	4.8	8.63	16.0			41.7
1st Floor/12-15	49.2	0.1021	4.80	17.3	6.14	18.1	70.0	24.9	49.8

*Asterisk designates selected panels.

Table 12-16 gives the weld thickness, and the length of welds loaded in the longitudinal and longitudinal/transverse directions. Table 12-16 also gives the design capacity of the longitudinal, longitudinal/transverse, and combined capacity ($\min(P_L + P_{LT}, Pw)n_s$) welds, expressed by the following modification of Equation 11-90 based on Equations 12-30 and 12-31 to become Equation 12-32. Comparing the total shear capacity and strap yield strength, P_{sy}, shows that these connections in Table 12-6 meet the requirements of Equation 12-32.

$$[min(P_L + P_{LT}, P_w)]n_s \geq P_{sy} \qquad \text{(Eq 12-32)}$$

Figure 12-9. Example connection/anchorage detail (fourth row of Table 12-6 and Table 12-9 through 12-22).

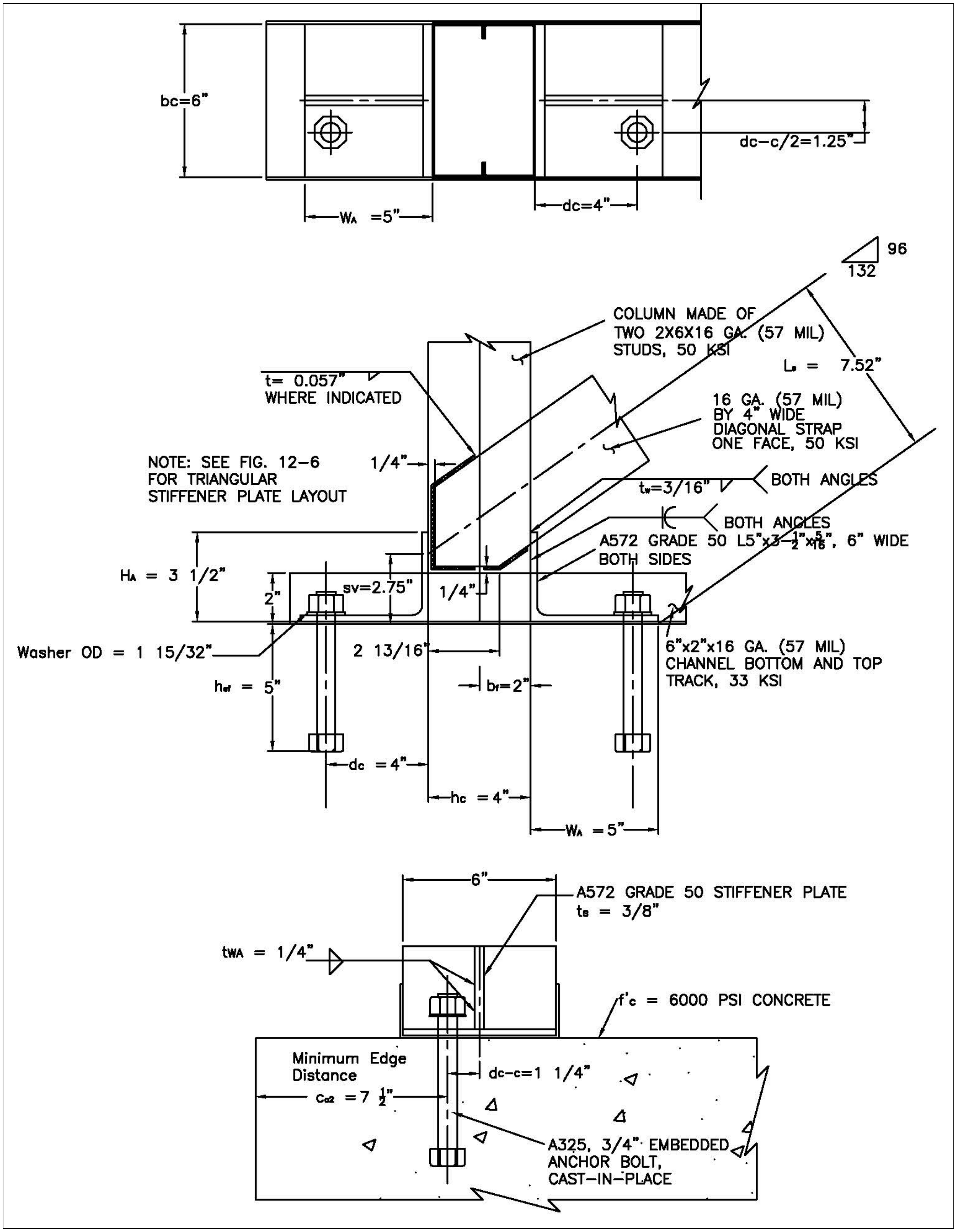


Figure 12-10. Example connection/anchorage detail – 5th row of Table 12-6 and Table 12-9 through 12-22.

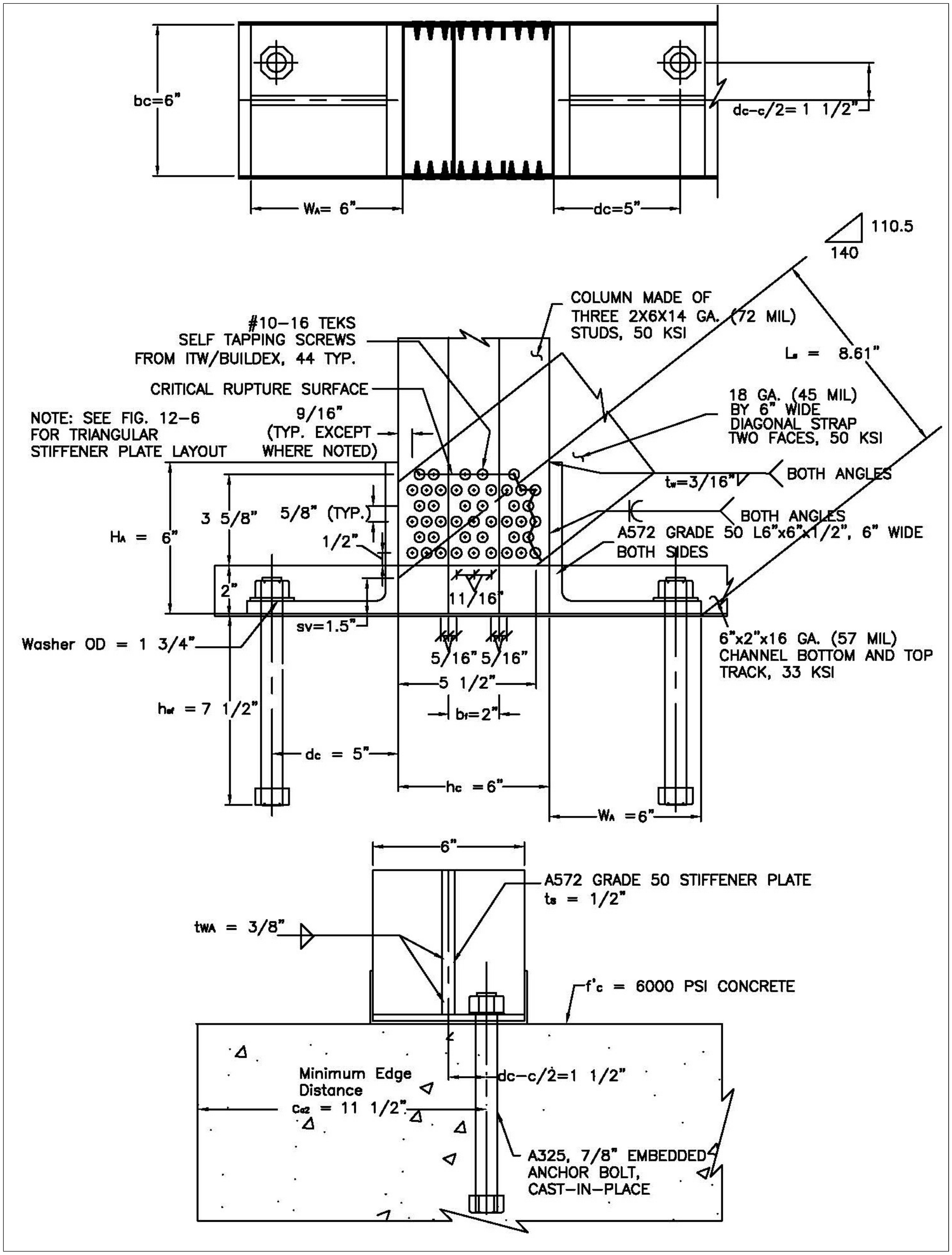


Figure 12-11. Example connection/anchorage detail (sixth row of Table 12-6 and Table 12-9 through 12-22).

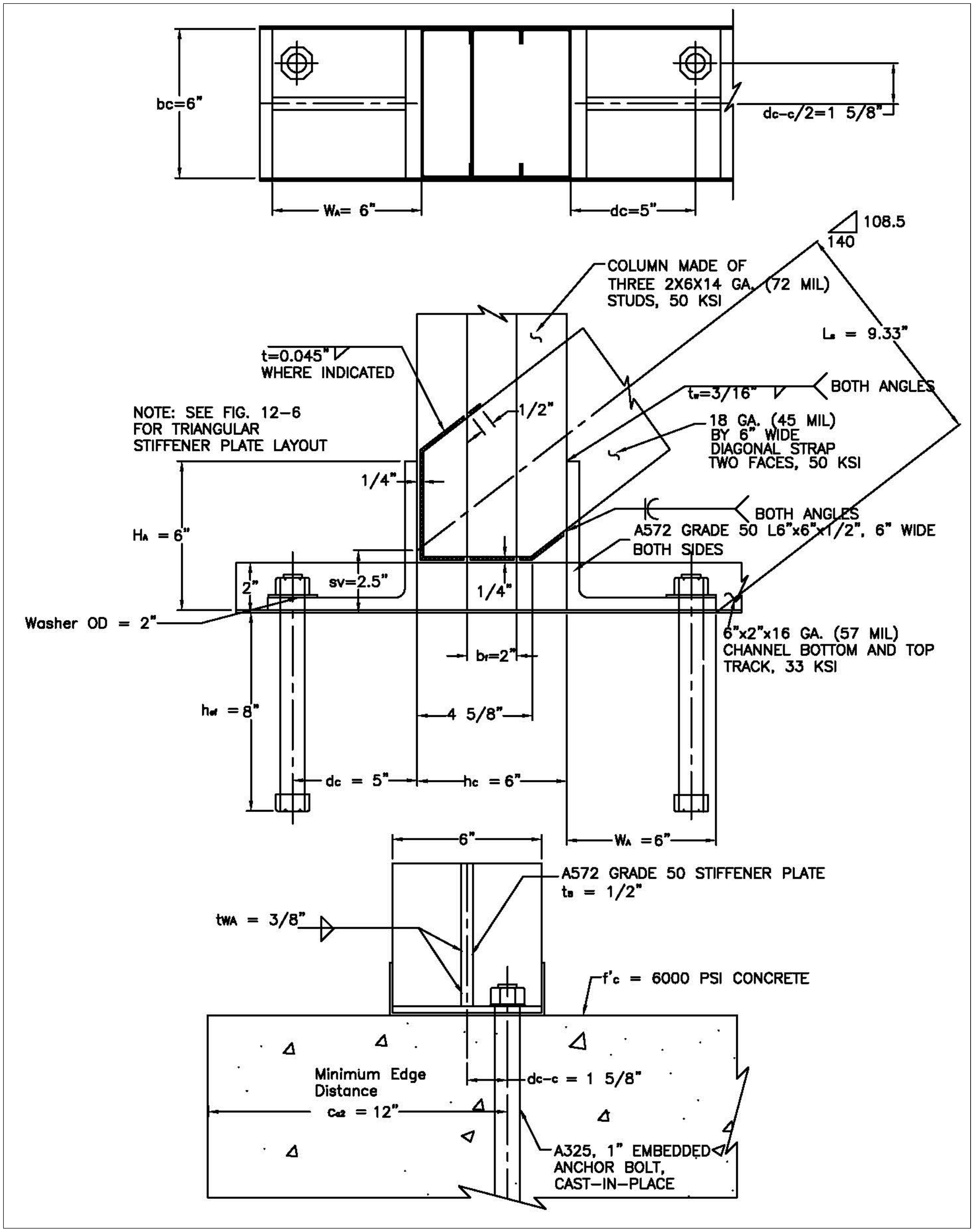


Figure 12-12. Example connection/anchorage detail
(seventh row of Table 12-6 and Table 12-9 through 12-22).

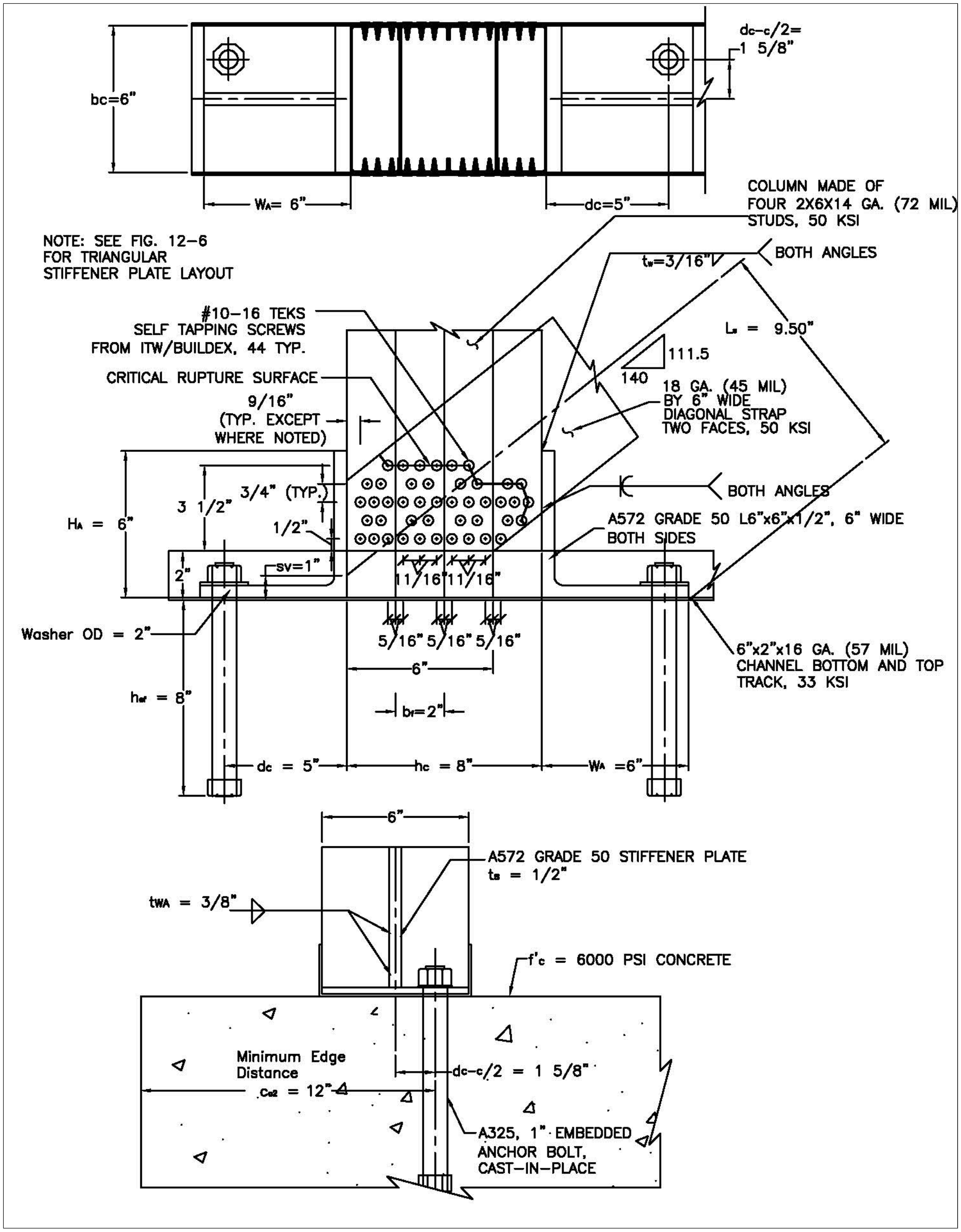


Figure 12-13. Example connection/anchorage detail
(eighth row of Table 12-6 and Table 12-9 through 12-22).

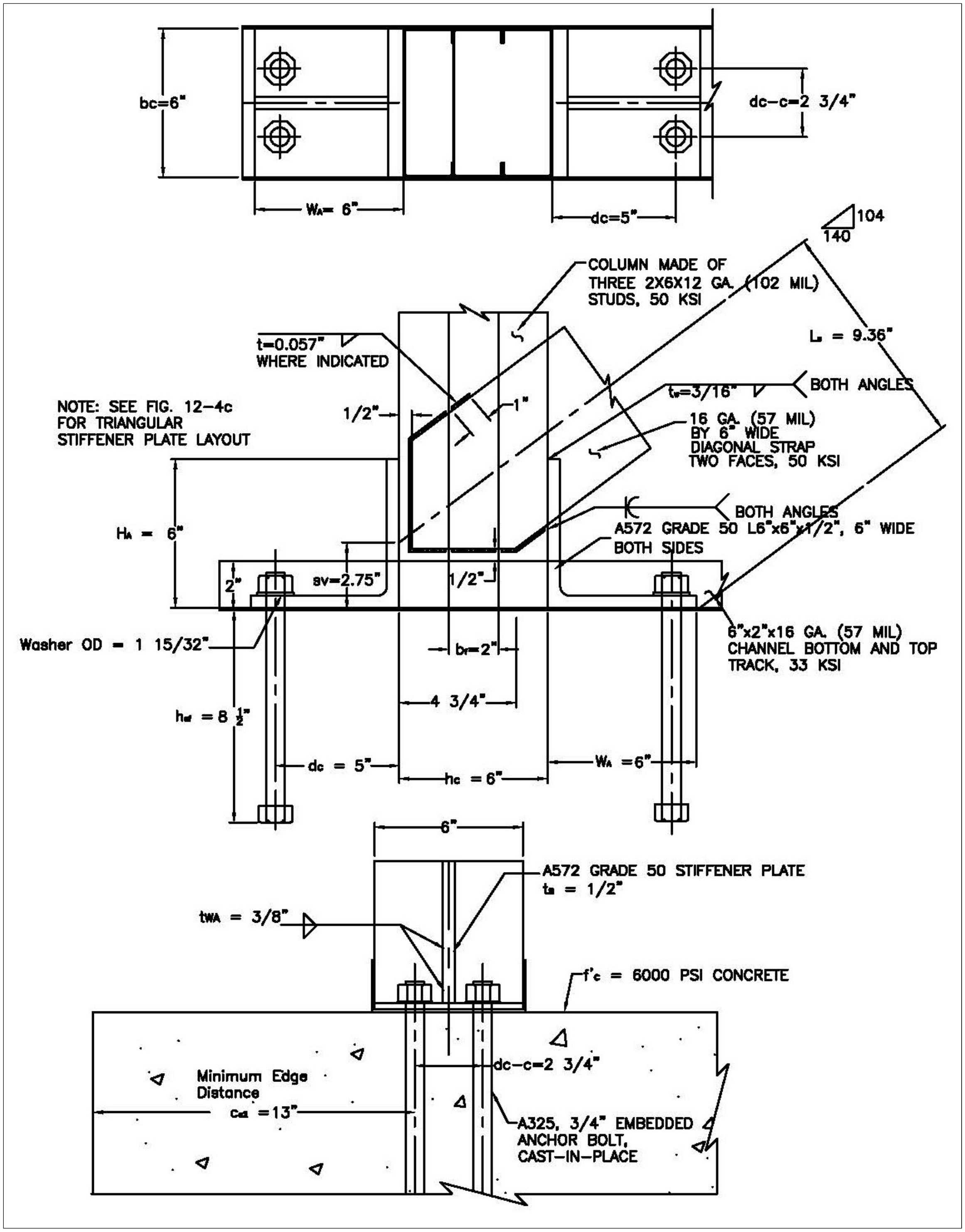


Figure 12-14. Example connection/anchorage detail (ninth row of Table 12-6 and Table 12-9 through 12-22).

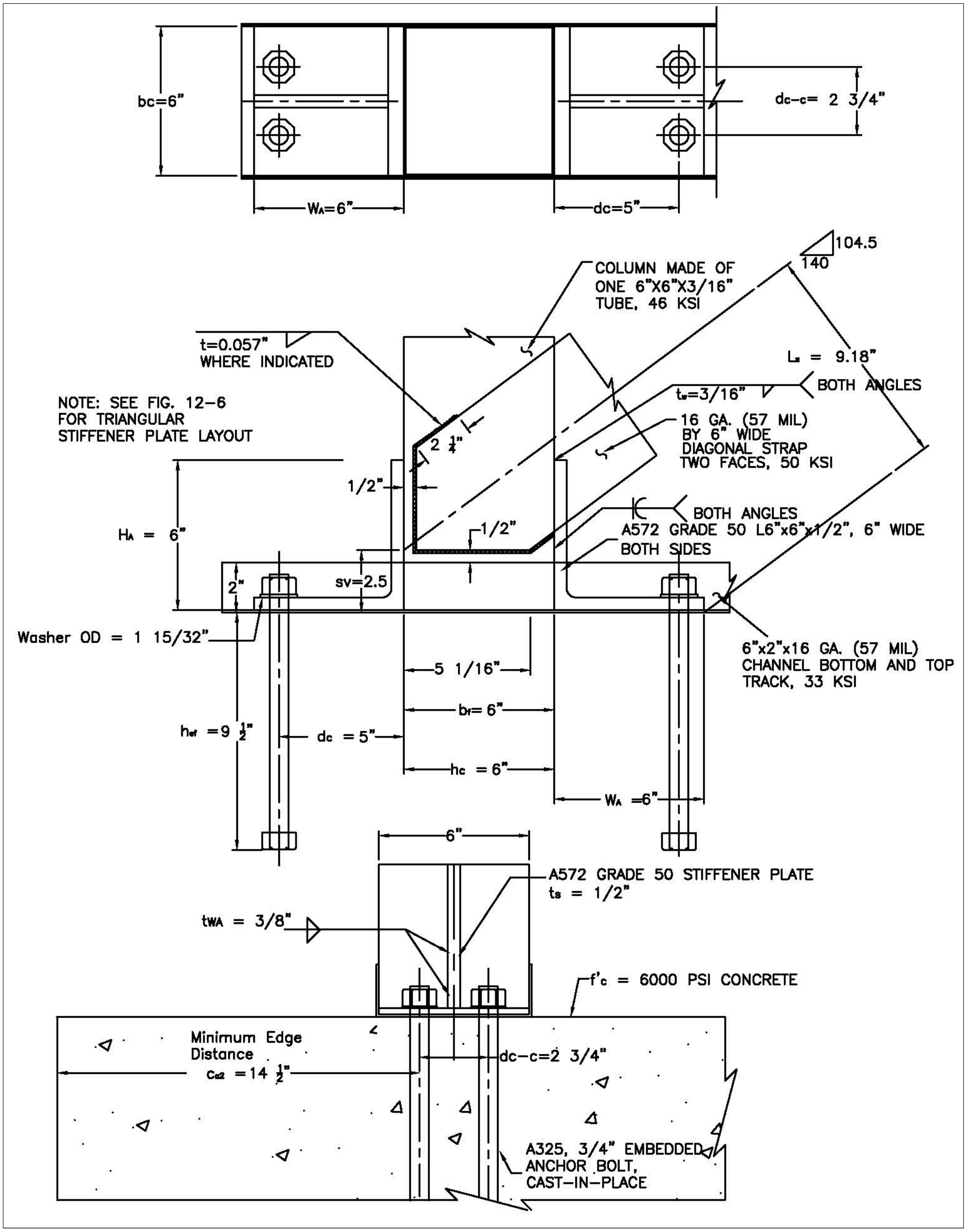
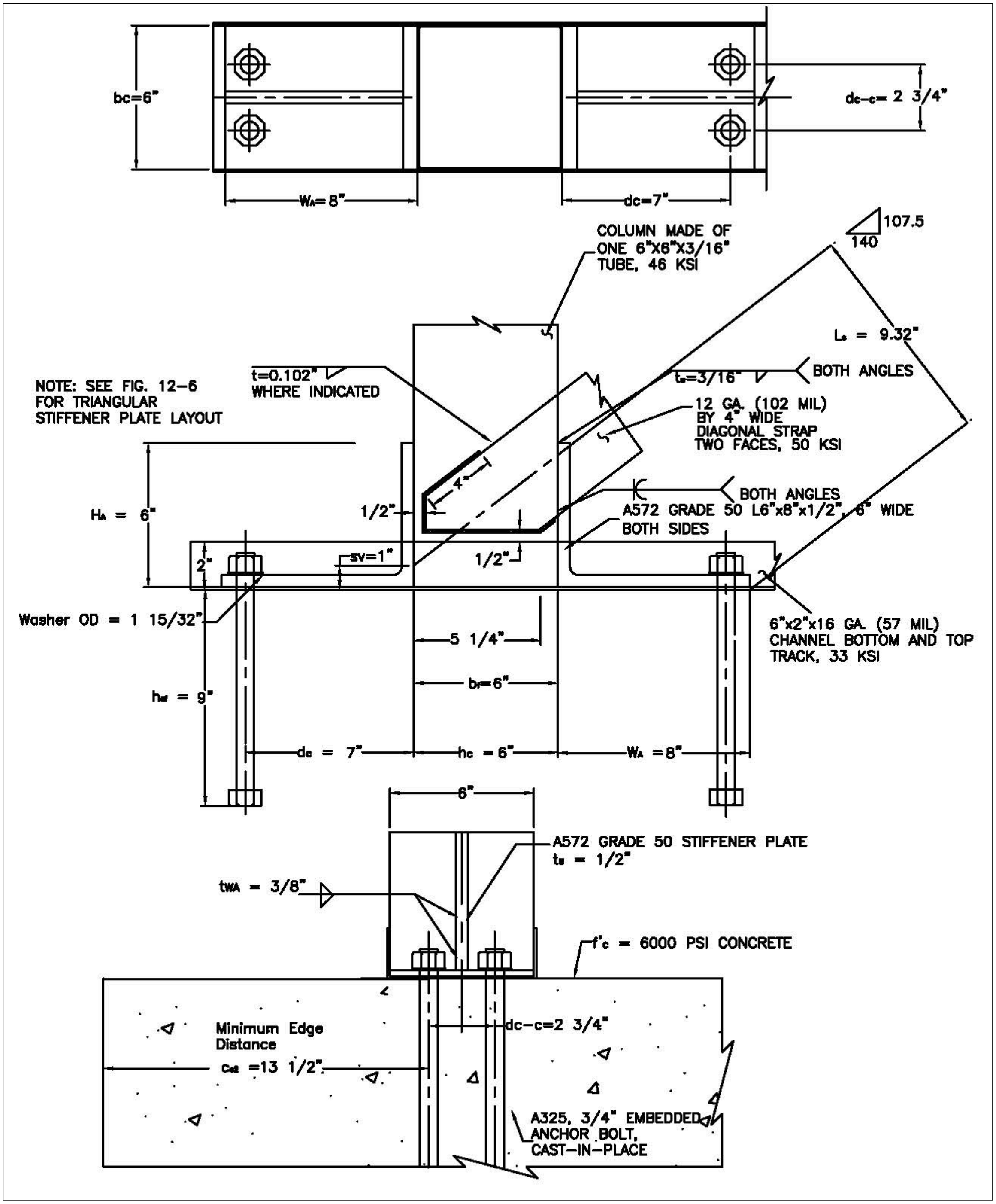


Figure 12-15. Example connection/anchorage detail
(tenth row of Table 12-6 and Table 12-9 through 12-22).



12.14 Shear panel anchors

Panel anchors must be installed on both sides of the shear panel columns. These anchors are installed at both the top and bottom of the columns to anchor the panels to the floor diaphragms both above and below the shear panels. The anchors are needed to provide the required shear, uplift, and moment resistance from the eccentric diagonal strap loading of the anchors. The anchors will also provide limited moment resistance that will allow some moment frame action of the columns, providing system redundancy and a widening of the hysteretic load/deflection envelope. An anchor consists of an angle iron section with a stiffener plate oriented perpendicular to and welded to the legs of the angle. The stiffener plate is oriented in-plane with the panel and is located at the center of the panel in the out-of-plane direction. One leg of the anchors is welded to the columns, while the other rests inside the panel track and is bolted to the diaphragm using embedded anchor bolts (see Figure 12-4 through Figure 12-15).

12.14.1 Anchor shear capacity

All of the trial columns shown in Table 12-9 have insufficient shear capacity by themselves (see Table 12-13) and require additional capacity from their anchorage. The anchor angle vertical leg is loaded in bending, with the critical bending plane defined by Equation 11-96. Each angle leg extends beyond the critical shear plane (i.e., the H_A dimension of the angles in Table 12-17 extends above the shear plane). Figure 12-4 shows such an anchor that consists of an A572 Grade 50, 6 in. long, L 5 x 3.5 x 0.375 in. angle iron section with a 0.375 in. thick stiffener plate (shown in Table 12-20). Anchors are welded to both sides of each column at both the column tops and bottoms. The anchor bending and resulting additional horizontal capacity from the anchors on both sides of the columns are defined in Equation 11-93, and the combined column and anchor capacities are defined according to Equation 11-92. The column shear capacity, V_c, was determined earlier, in section 11.12.5, according to Equation 11-65. Table 12-17 shows the angle yield stress, width (equal to out-of-plane width of column), and thickness of the angles used in these anchors so that their combined strength, V_T, exceeds P_{humax} (Equation 11-92). Table 12-17 includes the column shear capacity presented earlier in Table 12-13 and shows that combined shear strength, V_T, exceeds P_{humax} for all the trial shear panels.

Table 12-17. Column shear and anchor bending design.

Column Type / Figure No.	Column Shear Strength V_c (kips)	Strap Lat Ult Capacity P_{humax} (kips)	Anchor Angle Yield Strength F_{Ay} (ksi)	Size H_A	W_A	t_A (in)	Horiz Moment Arm d_{Ah} (in)	Angle Thickness + fillet radii k (in)	Vert Angle Height d_{Av} (in)	Width of Angle in Bending W_{Ab}	Vertical Moment Arm L_{Av}	Angle Moment Arm L_A (in)	Angle/Col Plastic Section Modulus Z_A (in3)	Angle/Col Moment Capacity M_A (k-in)	Anchor Horiz Strength P_{Ah} (kips)	Total Shear Strength V_T (kips)
3rd Floor/12-4	4.3	15.3	50	L 3.5 x	5.0 x	3/8	2.56	13/16	2.69	3.71	1.69	1.16	0.17	7.79	13.4	17.7
3rd Floor/12-7	6.5	14.6	50	L 3.5 x	5.0 x	5/16	2.56	3/4	2.75	3.76	1.75	1.19	0.13	5.77	9.7	16.2
3rd Floor/12-8	13.0	19.4	50	L 3.5 x	5.0 x	1/4	2.63	11/16	2.81	3.85	1.81	1.24	0.09	4.07	13.2	26.1
3rd Floor/12-9*	6.5	14.6	50	L 3.5 x	5.0 x	5/16	2.56	3/4	2.75	3.76	1.50	1.02	0.13	5.77	11.3	17.8
2nd Floor/12-10	24.5	34.3	50	L 6.0 x	6.0 x	1/2	2.38	1	5.00	5.54	1.50	0.64	0.45	20.34	126	151
2nd Floor/12-11*	24.5	34.3	50	L 6.0 x	6.0 x	1/2	2.38	1	5.00	5.54	1.25	0.54	0.45	20.34	152	176
2nd Floor/12-12	32.6	34.3	50	L 6.0 x	6.0 x	1/2	2.38	1	5.00	5.54	1.50	0.64	0.45	20.34	126	159
1st Floor/12-13*	34.9	43.7	50	L 6.0 x	6.0 x	1/2	2.38	1	5.00	5.54	1.50	0.64	0.50	22.58	140	175
1st Floor/12-14	59.0	43.7	50	L 6.0 x	6.0 x	1/2	2.38	1	5.00	5.54	1.50	0.64	0.65	29.43	183	242
1st Floor/12-15	59.0	52.3	50	L 6.0 x	8.0 x	1/2	2.38	1	5.00	5.54	1.50	0.64	0.65	29.43	183	242

*Asterisk designates selected panels.

12.14.2 Column-to-anchor angle weld design

Column-to-angle welds and angle sizes are selected for each trial configuration based on Table 11-9 and Table 11-10 and the angle thickness needed to provide sufficient bending capacity (see section 12.14.1). The selected weld thickness was 3/16 in. for all but one configuration of the third-floor trial panels so that heavy enough angles could be used in the anchors. This thickness is also greater than twice the thickness of the third-floor columns, so that vertical groove weld strength, P_G, is controlled by double shear at this floor (Equation 11-102)[24]. In this particular case the additional strength of double shear was not needed (see Table 12-18), but the benefit of double shear at this weld may be useful in other cases. For the first- and second-floor panels, the 3/16 in. welds permit the use of 0.5 in. thick angles for their anchors. The vertical weld strength is controlled by single shear (Equation 11-101) in first-floor columns and double shear in second-floor columns. Each of these anchors, defined in Table 12-17, meets the requirements of Equation 11-100, as shown in Table 12-18.

[24] The outside radius of the columns will be about twice the thickness of the column material, so the effective thickness of these welds will conveniently fill the gap created between the column and the angle leg.

Table 12-18. Column-to-anchor weld design.

Column Type / Figure No.	Min Gravity Load/ Panel GL_{min} (kips)	Anchor Uplift @ max Strap Yield P_{vymax} (kips)	Remaining Column Bending Cap M_{Rem} (k-in)	Tensile Force Avail/anchor P_M (kips)	Tensile Force/ Angle $P_{vymax}/2+P_M$ (kips)	Angle Horiz Weld Strength P_T (kips)	Col/Anchor Weld Thickness t_w (in)	Angle Vert Weld Strength P_G (kips)	Angle Tot Weld Strength P_A (kips)
3rd Floor/12-4	1.42	10.8	37.7	9.42	14.82	9.98	3/16	14.77	24.75
3rd Floor/12-7	1.42	9.7	66.5	16.63	21.47	14.41	3/16	21.34	35.75
3rd Floor/12-8	1.42	13.9	60.6	15.16	22.09	14.41	1/8	21.34	35.75
3rd Floor/12-9*	1.42	9.7	66.5	16.63	21.47	14.41	3/16	21.34	35.75
2nd Floor/12-10	6.28	22.5	146	24.26	35.52	18.15	3/16	46.06	64.21
2nd Floor/12-11*	6.28	22.5	146	24.26	35.52	18.15	3/16	46.06	64.21
2nd Floor/12-12	6.28	22.5	237	29.65	40.91	18.15	3/16	46.06	64.21
1st Floor/12-13*	12.0	25.5	216	36.00	48.75	25.88	3/16	32.85	58.74
1st Floor/12-14	12.0	25.5	318	52.92	65.68	42.41	3/16	53.83	96.24
1st Floor/12-15	12.0	31.7	306	50.99	66.86	42.41	3/16	53.83	96.24

*Asterisk designates selected panels.

12.14.3 Anchor bolt design

Embedded anchor bolts are used to anchor the columns to the reinforced concrete floor diaphragms. The same bolt detail is used at both the top and bottom of the columns. The anchor bolts should be positioned with a template before the concrete is cast. Alternatively, for anchors above the first floor, the same bolts that anchor the top of one panel could extend through the concrete to anchor the bottom of the panel above. Holes for these anchors could be drilled through the slab or beam after the concrete is cast. section 11.14.8 provides recommendations on through-bolt anchors. The anchor bolt strength, diameter, and position are defined so that they have adequate shear strength, P_v, and tensile strength, P_t. Then the anchor bolt length will be determined so as to meet the concrete breakout strength, N_{cb} or N_{cbg}, requirements. Table 12-19 shows that all trial anchor bolts easily meet the shear strength requirements of Equation 11-106.

The anchor bolts must provide resistance for the moment from the eccentric loading of the diagonal strap, accounting for the maximum estimated yield overstrength of the strap ($P_{symax}L_s$ in Equation 11-112). Any moment capacity beyond this is not required, but provides beneficial column moment resistance (M_{Rem} in Equation 11-105). The anchor bolt diameter, d_{AB}; strength, P_t; and horizontal distance from the column face, d_c; were determined through an iterative process. All selected anchor bolts were ASTM A325 bolts (ASTM 2014a). The anchor bolt shear and tensile strengths were determined based on Chapter 11 design recommendations, which reference AISI/AISC 360-10, Table 3.2 (AISC 2010a), giving 68 ksi nominal shear strength for the A325 bolts (see Table 12-19). Table 12-19 shows most trial anchors used two anchor bolts per column, n_{AB}. The anchor bolts were positioned a distance from the columns, d_c, which was 1 in. less than the width of the trial anchor angles, W_A. This is the maximum distance away from the column that the anchor bolts can be placed for the selected angle width. Standard angles were selected for the anchors from AISC *Steel Construction Manual*, (AISC 2011, Part 1 "Dimensions and Properties," Table 1-7 "Angles"). Angles for the third-floor anchors were selected with their width, W_A, equal to 5 in. (i.e., L5 x 3-1/2 x 5/16 in. angle sections). Then the d_c was set to 1 in. less, equal to 4 in. for all third-floor column anchors (see Table 12-17).

Table 12-19. Shear panel anchor bolt design.

Column Type / Figure No.	# Anchor Bolts/col n_{AB} (in)	Anchor Bolt Dia d_{AB} (in)	Applied Shear/ Bolt P_{hAB} (kips)	Bolt Nom Shear Strength F_{nv} (ksi)	Bolt Shear Design Strength P_v (kips)	Strap Max Yield Strength P_{symax} (kips)	Conn C/L Vert Dist from Base s_V (in)	Moment Arm of Dia Strap L_s (in)	Anchor Bolts to Column Face Spacing d_c (in)	Tensile Force/ Bolt P_{tAB} (kips)	Bolt Nom Tensile Strength F_{nt} (ksi)	Modified Tensile Stress F'_{nt} (ksi)	Bolt Design Strength $P_t = \phi R_n$ (kips)
3rd Floor/12-4	2	5/8	7.66	68	15.65	18.90	2.00	6.96	4.0	12.63	90	73	16.78
3rd Floor/12-7	2	5/8	7.32	68	15.65	17.05	2.50	7.33	4.0	14.35	90	75	17.23
3rd Floor/12-8	2	3/4	9.69	68	22.53	23.90	2.50	7.33	4.0	17.76	90	78	25.94
3rd Floor/12-9*	2	3/4	7.32	68	22.53	17.05	2.75	7.52	4.0	14.60	90	88	29.08
2nd Floor/12-10	2	7/8	17.14	68	30.67	40.74	1.50	8.61	5.0	27.54	90	67	30.08
2nd Floor/12-11*	2	1	17.14	68	40.06	40.74	2.50	9.33	5.0	29.25	90	78	46.23
2nd Floor/12-12	2	1	17.14	68	40.06	40.74	1.00	9.50	5.0	31.21	90	78	46.23
1st Floor/12-13*	4	3/4	10.91	68	22.53	51.16	2.75	9.36	5.0	18.85	90	73	24.32
1st Floor/12-14	4	3/4	10.91	68	22.53	51.16	2.50	9.18	5.0	21.56	90	73	24.32
1st Floor/12-15	4	3/4	13.07	68	22.53	61.26	1.00	9.32	7.0	19.31	90	65	21.47

*Asterisk designates selected panels.

The vertical distance between where the centerline of the diagonal strap-to-column connection crosses the outside vertical plane of the column, to the top of the column top connections or bottom of the column bottom connections, s_V, is illustrated in Figure 12-4 through Figure 12-15. This distance is used to calculate the moment arm of the diagonal strap, L_s, as shown in Equation 11-114, which is used in determining the anchor bolt applied tensile force according to Equation 11-112. For the third-floor anchors, the angle-to-stiffener weld, t_{Aw}, was the smaller of the maximum permitted by Table 11-9, or 0.25 in., while first- and second-floor anchors used a weld thickness of 0.375 in. This weld thickness should be the minimum that satisfies Equations 11-116 and 11-117 so that the anchor bolts can be placed as close as possible to the stiffener plate (i.e., minimum $d_{c\text{-}c}$, in Equation 11-115 and bending on the anchor angle). The top drawing of

Figure 12-4 and Figure 12-6 show the standard location and orientation of the anchor stiffener plates. Table 12-20 shows stiffener plate thicknesses, t_S, that are configured as shown in Figure 12-4 through Figure 12-15. The selected anchor bolts meet the shear strength requirement of Equation 11-106 and tensile requirement of Equation 11-108. Table 12-19 shows these requirements are met for the anchor bolts of these example panels.

12.14.4 Anchor angle thickness and angle-to-stiffener weld

The anchor angle strength must meet the requirements of Equations 11-118 and 11-120, in addition to the bending requirements of Equation 11-92 presented in Chapter 11. The strengths of the angles are based on an effective length of the critical yield and the rupture surface based on the anchor bolt washer size and distance to the edge of the angle (see Equation 11-119). Table 12-20 shows these requirements were met for all anchor angles.

The weld between the anchor angle and stiffener must have sufficient strength to satisfy Equation 11-116. The base metal capacity of the welded connection of the stiffener must also be checked according to Equation 11-117. The effective weld length, L_{Aw}, was taken as 3 in. for the third-floor anchors and 4 in. for the larger first- and second-floor anchors. Table 12-19 and Table 12-20 shows that the Equation 11-117 requirements are met for all trial anchors except the one shown in Figure 12-8, which was not selected. The asterisk, *, in Table 12-20 indicates that the anchors for the selected panels met these requirements.

Table 12-20. Anchor angle thickness and angle-to-stiffener weld strength.

Column Type / Figure No.	Angle/stiff Weld Thickness t_{Aw} (in)	Electrode Strength F_{xx} (ksi)	Effective Weld Length L_{Aw} (in)	Angle/stiff Weld Strength P_{Aw} (kips)	Yield/Rup Surface Length L_{Ay} (in)	Stiffener Base Metal Rupture P_{Su} (kips)	Angle Shear Yielding P_{Ay} (kips)	Angle/Stiff Ultimate Strength F_{Au} (ksi)	Angle Shear Rupture P_{Au} (kips)	Stiffener Plate Thickness t_S (in)	Washer Outside Diameter OD (in)	Out-of-plane Space btw Bolts d_{c-c} (in)
3rd Floor/12-4	1/4	70	3.0	16.7	2.3	21.9	26.0	65.0	25.4	3/8	1 5/16	2.5
3rd Floor/12-7	1/4	70	3.0	16.7	2.3	21.9	21.7	65.0	21.1	3/8	1 5/16	2.5
3rd Floor/12-8	3/16	70	3.0	12.5	2.5	16.5	18.5	65.0	18.1	3/8	1 15/32	2.5
3rd Floor/12-9*	1/4	70	3.0	16.7	2.5	21.9	23.1	65.0	22.6	3/8	1 15/32	2.5
2nd Floor/12-10	3/8	70	4.0	33.4	2.8	43.9	41.3	65.0	40.2	1/2	1 3/4	3.0
2nd Floor/12-11*	3/8	70	4.0	33.4	3.0	43.9	45.0	65.0	43.9	1/2	2	3.25
2nd Floor/12-12	3/8	70	4.0	33.4	3.0	43.9	45.0	65.0	43.9	1/2	2	3.25
1st Floor/12-13*	3/8	70	4.0	33.4	2.5	43.9	37.0	65.0	36.1	1/2	1 15/32	2.75
1st Floor/12-14	3/8	70	4.0	33.4	2.5	43.9	37.0	65.0	36.1	1/2	1 15/32	2.75
1st Floor/12-15	3/8	70	4.0	33.4	2.5	43.9	37.0	65.0	36.1	1/2	1 15/32	2.75

*Asterisk designates selected panels.

12.14.5 Cast-in anchor concrete breakout strength in tension

Cast-in anchors are used to anchor all the shear panels presented in this example. Post-installed anchors could also be used in accordance with the recommendations in Chapter 11 and ACI 318-11 (ACI 2011a, Appendix D). However, for many applications through-bolts may be used more economically to anchor the panels to intermediate floors and the roof diaphragm. The last section in Chapter 11 (section 11.14.8) provides through-bolt anchorage recommendations. For the cast-in anchor examples presented here, the concrete breakout strength in tension, defined by Equations 11-121 or 11-122, must exceed the applied tensile force per anchor bolt, P_{tAB} (Equation 11-112). Values for the design breakout strength for a single anchor bolt (when $n_{AB} = 2$) were calculated using Equation 11-121 for the first seven rows of Table 12-21. Similarly, design breakout strength for the two anchor bolts in tension (when $n_{AB} = 4$) were calculated using Equation 11-122 for the last three rows of Table 12-21. The effective embedment depth, h_{ef}, was adjusted so that these breakout strengths exceed the applied tensile force per bolt, P_{tAB}.

Table 12-21. Cast-in anchor concrete breakout strength in tension.

Column Type / Figure No.	Tensile Force/ Bolt P_{tAB} (kips)	Effective Embedment Depth h_{ef} (in)	In-plane Edge Distance C_{a1} (in)	Out-of-plane Edge Distance C_{a2} (in)	Actual Out-of-plane Bolt Space d_{c-c} (in)	Concrete Failure Area A_{Nc} (in^2)	Concrete Failure Area A_{Nco} (in2)	Edge Effect Modifcation Factor $\Psi_{ed,N}$	No Cracking Modifcation Factor $\Psi_{c,N}$	Post-Install Modifcation Factor $\Psi_{cp,N}$	Eccentric Modifcation Factor $\Psi_{ec,N}$	Anchor Type Factor k_c	Light Conc Modifcation Factor λ_a	Concrete Compressive Strength f'_c (psi)	Concrete Breakout Strength N_b (kips)	Design Breakout Strength $\phi_{ct}N_{cb}$ (kips)
3rd Floor/12-4	12.63	5.0	7.5	7.5	0	225	225	1.00	1.0	1.0	1.0	24	1.0	6,000	20.8	15.59
3rd Floor/12-7	14.35	5.0	7.5	7.5	0	225	225	1.00	1.0	1.0	1.0	24	1.0	6,000	20.8	15.59
3rd Floor/12-8	17.76	5.5	8.3	8.3	0	272	272	1.00	1.0	1.0	1.0	24	1.0	6,000	24.0	17.98
3rd Floor/12-9*	14.60	5.0	7.5	7.5	0	225	225	1.00	1.0	1.0	1.0	24	1.0	6,000	20.8	15.59
2nd Floor/12-10	27.54	7.5	11.3	11.3	0	506	506	1.00	1.0	1.0	1.0	24	1.0	6,000	38.2	28.64
2nd Floor/12-11*	29.25	8.0	12.0	12.0	0	576	576	1.00	1.0	1.0	1.0	24	1.0	6,000	42.1	31.55
2nd Floor/12-12	31.21	8.0	12.0	12.0	0	576	576	1.00	1.0	1.0	1.0	24	1.0	6,000	42.1	31.55
1st Floor/12-13*	18.85	8.5	12.8	12.8	2.75	720	650	1.00	1.0	1.0	1.0	24	1.0	6,000	46.1	19.14
1st Floor/12-14	21.56	9.5	14.3	14.3	2.75	891	812	1.00	1.0	1.0	1.0	24	1.0	6,000	54.4	22.38
1st Floor/12-15	19.31	9.0	13.5	13.5	2.75	803	729	1.00	1.0	1.0	1.0	24	1.0	6,000	50.2	20.74

*Asterisk designates selected panels.

In these examples, the in-plane edge distance, c_{a1}, and out-of-plane edge distance, c_{a2}, were set equal to 1.5 h_{ef}. These are the minimum edge distances that do not cause a reduction in the concrete failure area, A_{Nc}. The shear panels in these examples are oriented in the short direction of the building, whereas most panels are at the interior of the building where large edge distances should not be a problem in a heavy slab or wide beam. If large edge distances are a concern, those concerns can be reduced by increasing the embedment depth, increasing concrete strength, or using through-bolts. Figure 12-4 through Figure 12-15 show the trial anchor design for each row in Table 12-17 through Table 12-21. Figure 12-9, Figure

12-11, and Figure 12-12 show the selected anchors, which are indicated by an asterisk, *, in the first column of these tables for the third, second, and first floors respectively.

12.14.6 Cast-in anchor concrete breakout strength in shear

The concrete breakout strength in shear, defined by Equations 11-127 or 11-128, must exceed the applied shear force per anchor bolt, P_{hAB} (Equation 11-106). Values for the design breakout strength for a single anchor bolt (when n_{AB} = 2) were calculated using Equation 11-127 for the first seven rows of Table 12-22. Similarly, design breakout strength for the two anchor bolts in shear (when n_{AB} = 4) were calculated using Equation 11-128 for the last three rows of Table 12-22. The effective embedment depth, h_{ef}, was increased for only the first row of Table 12-22, because this was the only case where shear breakout strength required a longer bolt than tension.

Table 12-22. Cast-in anchor concrete breakout strength in shear.

Column Type / Figure No.	Applied Shear/ Bolt P_{hAB} (kips)	Load-bearing length l_e (in)	Concrete Failure Area A_{Vc} (in^2)	Concrete Failure Area A_{Vco} (in2)	Edge Effect Modifcation Factor $\Psi_{ed,V}$	No Cracking Modifcation Factor $\Psi_{c,V}$	h_a<2.5C_{a1} Modifcation Factor $\Psi_{h,V}$	Eccentric Modifcation Factor $\Psi_{ec,V}$	Eq 11-132 Concrete Breakout Strength in Shear V_b (kips)	Eq 11-133 V_b (kips)	Design Breakout Strength $\phi_{cv}V_{cb}$ (kips)	Anchor Avail Mom Resitance $M_{colAvail}$ (k-in)	Column Moment Capacity M_{colCap} (k-in)
3rd Floor/12-4	7.66	5.0	253	253	0.90	1.0	1.0	1.0	13.3	14.3	9.01	38	53
3rd Floor/12-7	7.32	5.0	253	253	0.90	1.0	1.0	1.0	13.3	14.3	9.01	67	80
3rd Floor/12-8	9.69	5.5	306	306	0.90	1.0	1.0	1.0	16.6	16.5	11.15	61	80
3rd Floor/12-9*	7.32	5.0	253	253	0.90	1.0	1.0	1.0	14.1	14.3	9.51	67	80
2nd Floor/12-10	17.14	7.0	570	570	0.90	1.0	1.0	1.0	29.0	26.3	17.76	146	184
2nd Floor/12-11*	17.14	8.0	648	648	0.90	1.0	1.0	1.0	34.2	29.0	19.56	146	184
2nd Floor/12-12	17.14	8.0	648	648	0.90	1.0	1.0	1.0	34.2	29.0	19.56	237	281
1st Floor/12-13*	10.91	6.0	784	732	0.90	1.0	1.0	1.0	32.4	31.7	11.48	216	259
1st Floor/12-14	10.91	6.0	973	914	0.90	1.0	1.0	1.0	38.3	37.5	13.47	318	365
1st Floor/12-15	13.07	6.0	876	820	0.90	1.0	1.0	1.0	35.3	34.6	12.46	306	365

*Asterisk designates selected panels.

Figure 12-4 through Figure 12-15 show the trial anchor design for each row in Table 12-17 through Table 12-22. Figure 12-9, Figure 12-11, and Figure 12-13 show the selected anchors, which are indicated by an asterisk, *, in the first column of these tables for the third, second, and first floors, respectively.

12.15 Summary of example design problem results

Figure 12-9, Figure 12-11, and Figure 12-13 illustrate the details for the selected panels. Details for all panels are given in Table 12-6 and Table 12-9 through Table 12-22. The details of all trial panels are given in those tables, and the connection and anchor details are given in Figure 12-4 through Figure 12-15, to illustrate the variety of panel configurations that may be considered. None of the selected panels used structural tube columns (last two rows in the tables, with the details shown in Figures 12-14 and 12-15) because it was decided to use columns that are all built up from studs. However, the shear panels with structural tube columns meet all the requirements of these design recommendations.

Recommendations in Chapter 11, section 11.11 ("Diagonal strap design," p 216) require that shear panels above the ground floor have shear panels in the same direction below them, as illustrated in Figure 12-3. Through-bolt anchors at intermediate floor levels and at the roof diaphragm may often be more economical than the cast-in anchor bolts shown in the example panels (Figure 12-3). For through-bolt anchors, the anchor of the panel above should be modified to accommodate the through-bolt size and position of the heavier panels below. However, the example panels used anchor bolts to illustrate more generic shear panels that are presented for prescriptive design in Appendix C.

The spreadsheet design program used in Table 12-6 and Table 12-9 through Table 12-21 should be very useful in practical cold-formed steel seismic design (Wilcoski 2014).

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APPENDICES

Appendix A: Prototype Barracks Building and Cold-Formed Steel Test Panel Drawings

This appendix shows a typical three-story barracks framing layout and the six panels tested by ERDC-CERL. The elevation views are a good representation of the typical shear wall panel layout. However, the connection details have been modified since testing the earlier panels and only the details shown in test panels C1 and D2 are recommended. Designers should use the new diagonal strap-to-column connection and column anchorage details shown in the design example in Chapter 12.

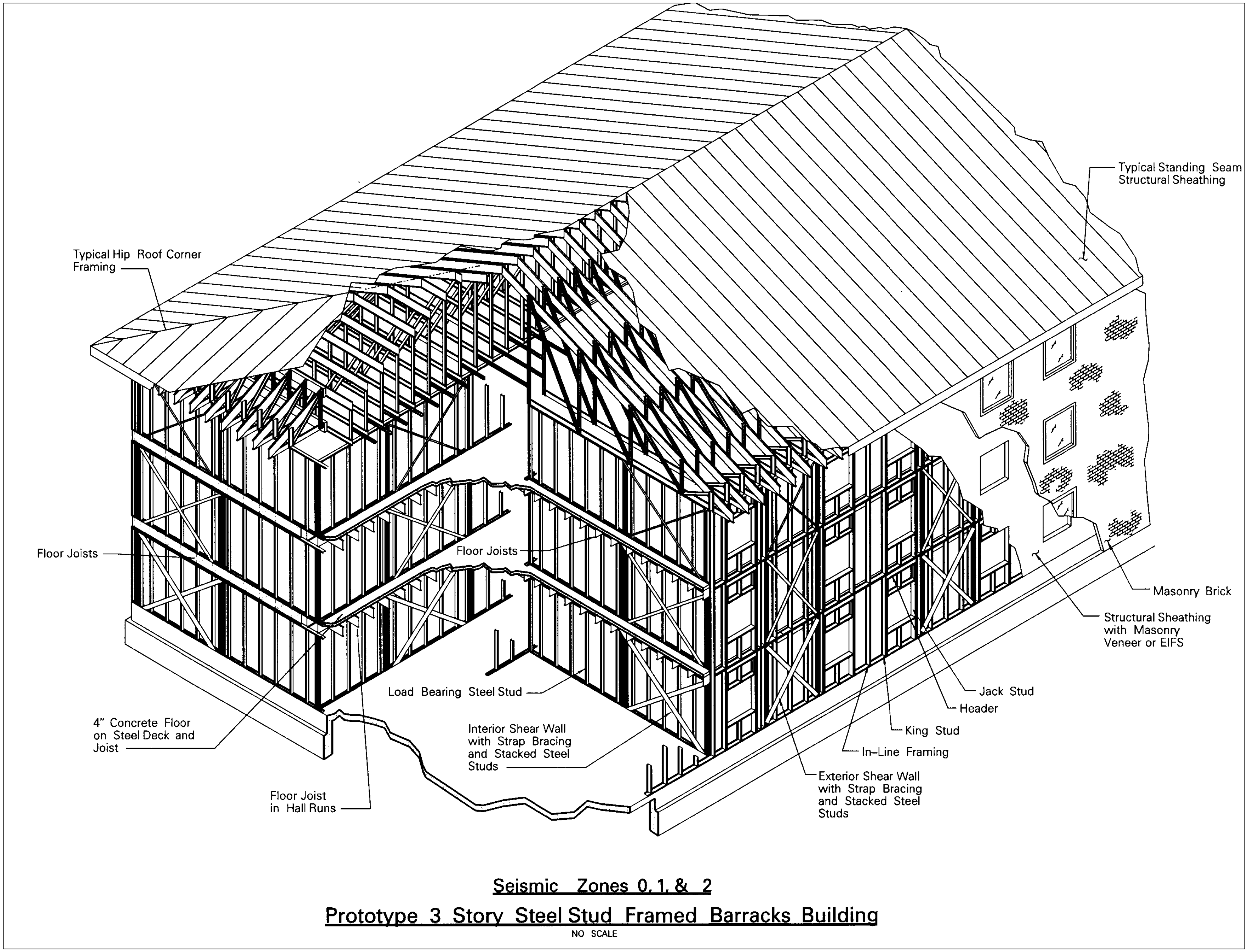


Figure A-1. Prototype 3-story barracks 3-D drawing.

Figure A-2. Test panel A1.

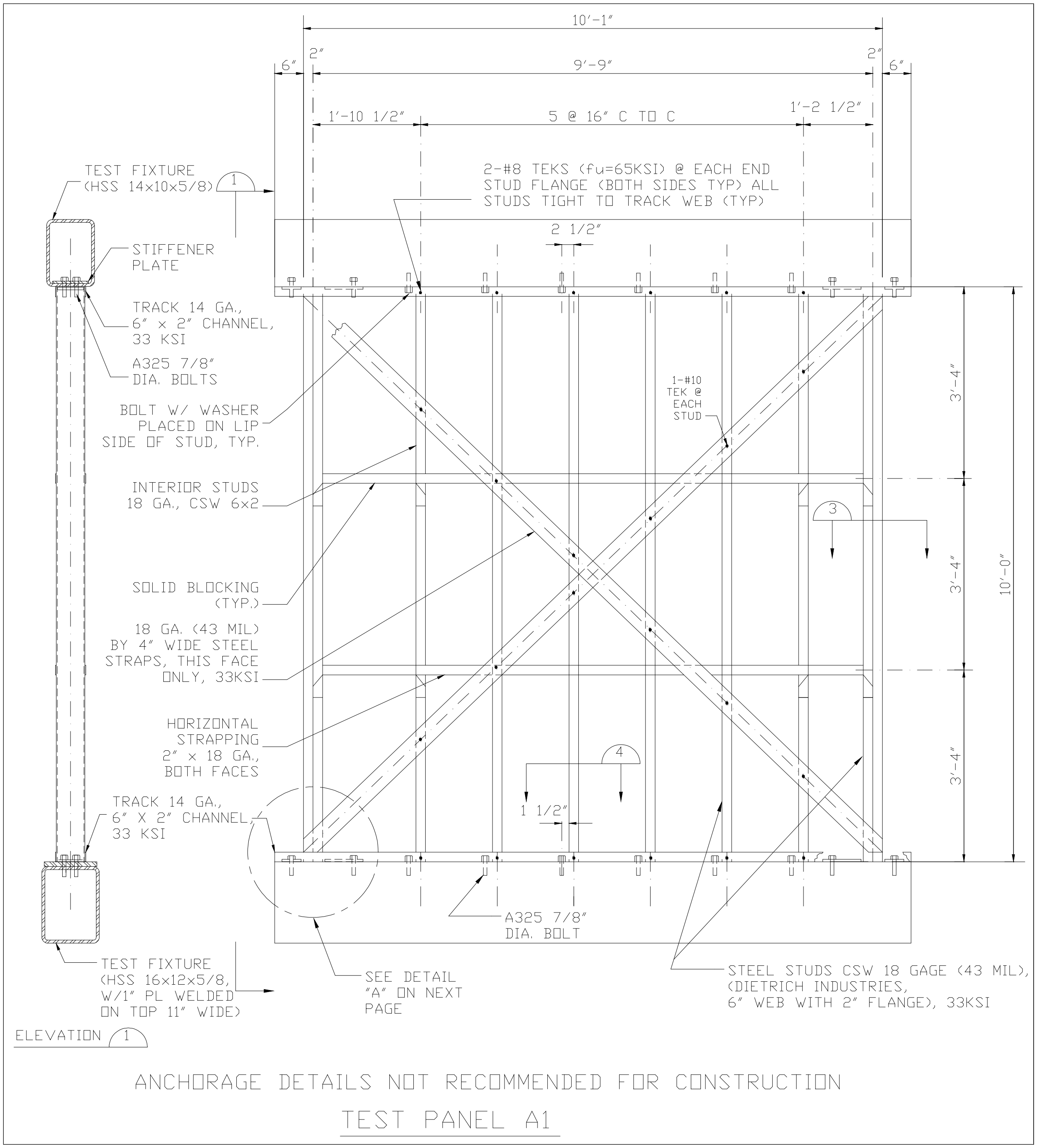


Figure A-3. Test panel A1 details.

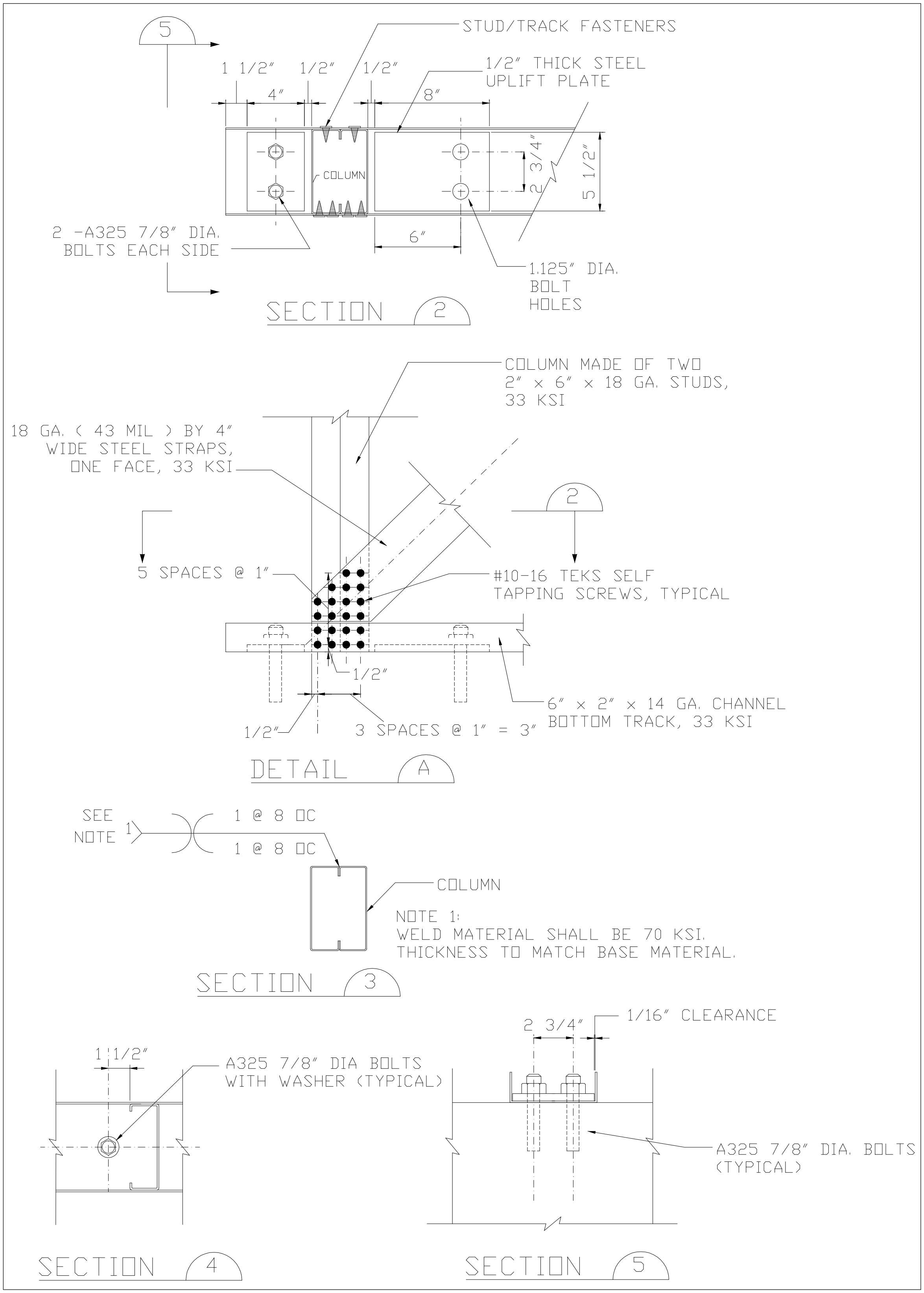


Figure A-4. Test panel A2.

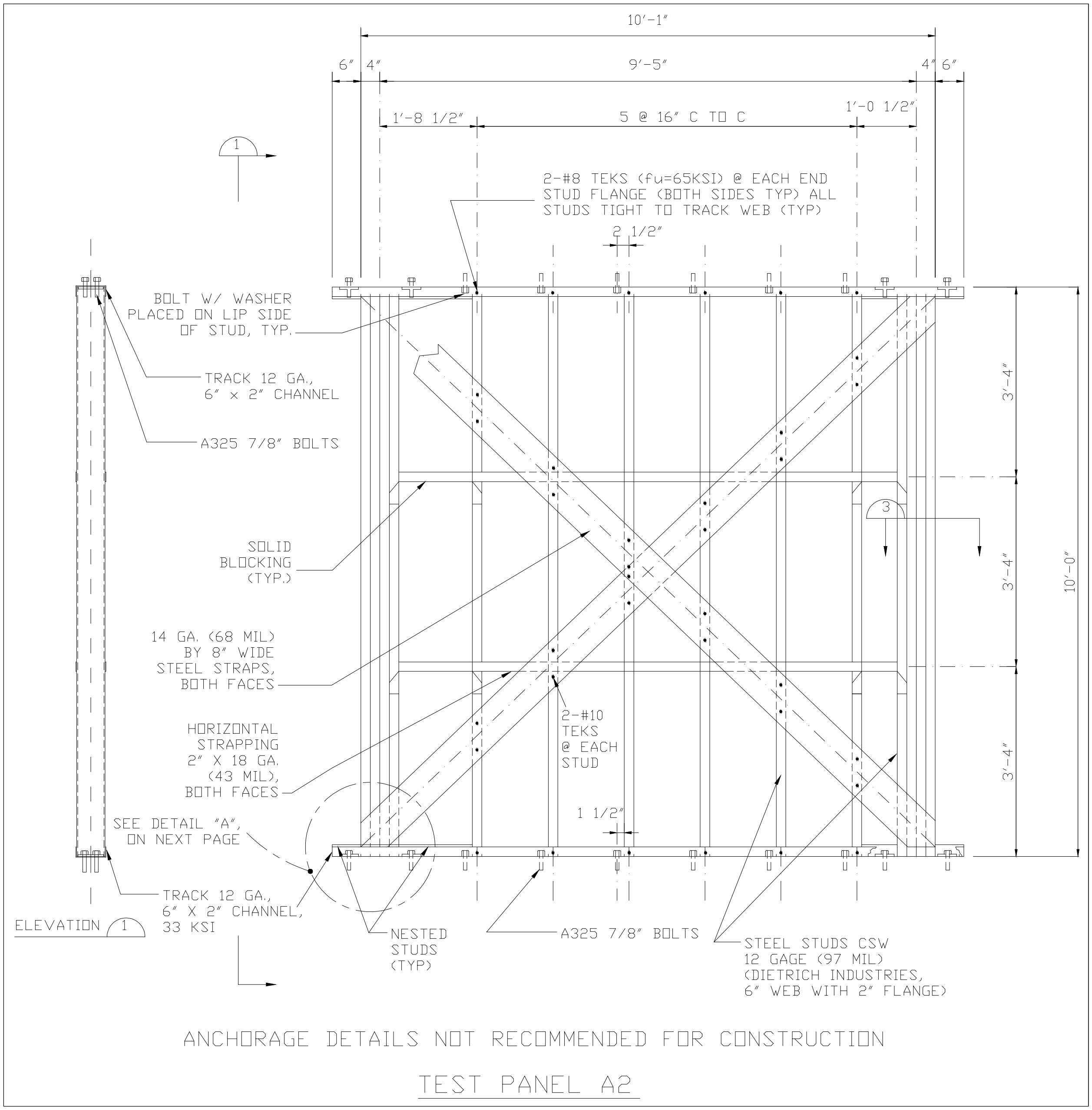


Figure A-5. Test panel A2 details.

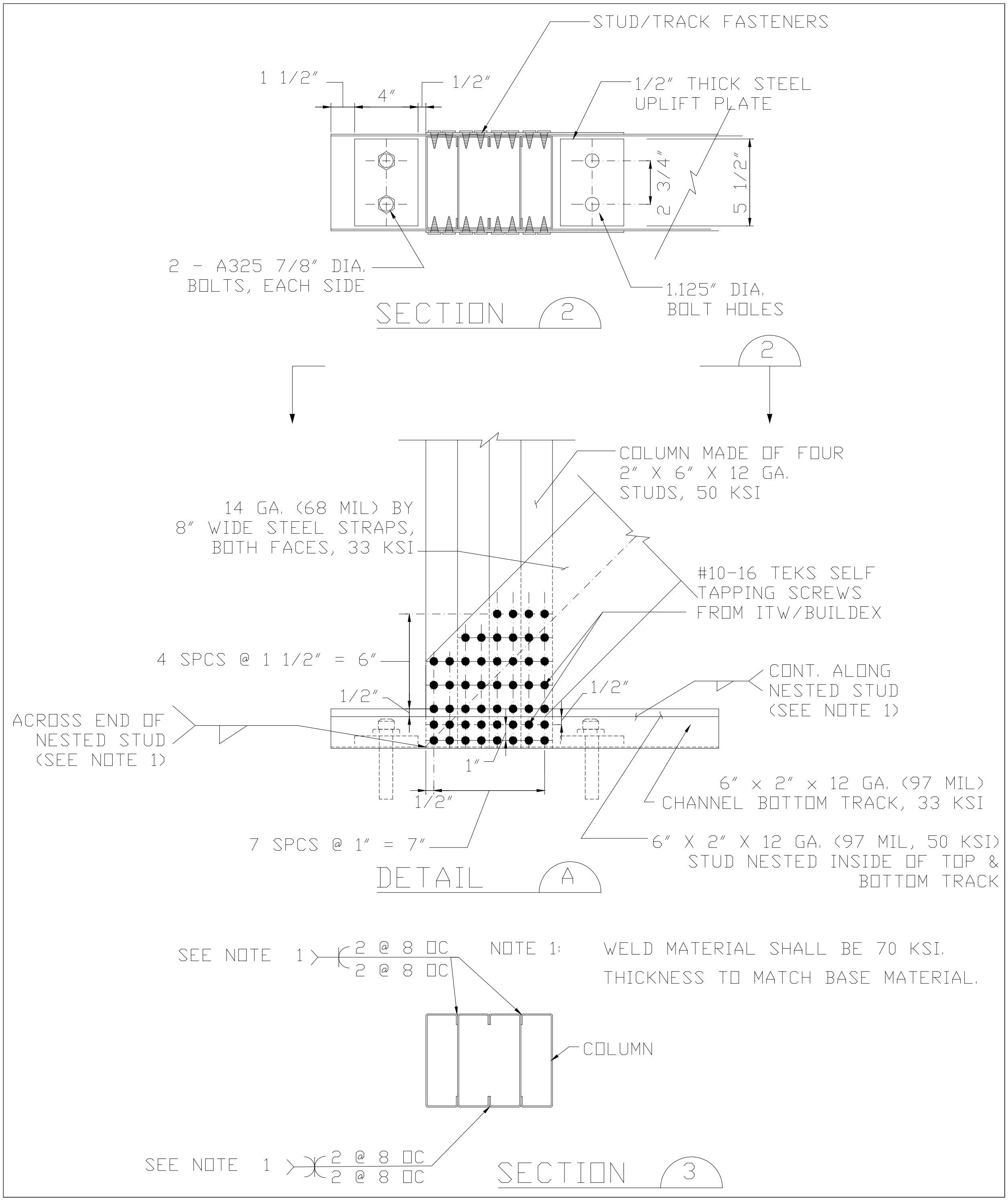


Figure A-6. Test panel A3.

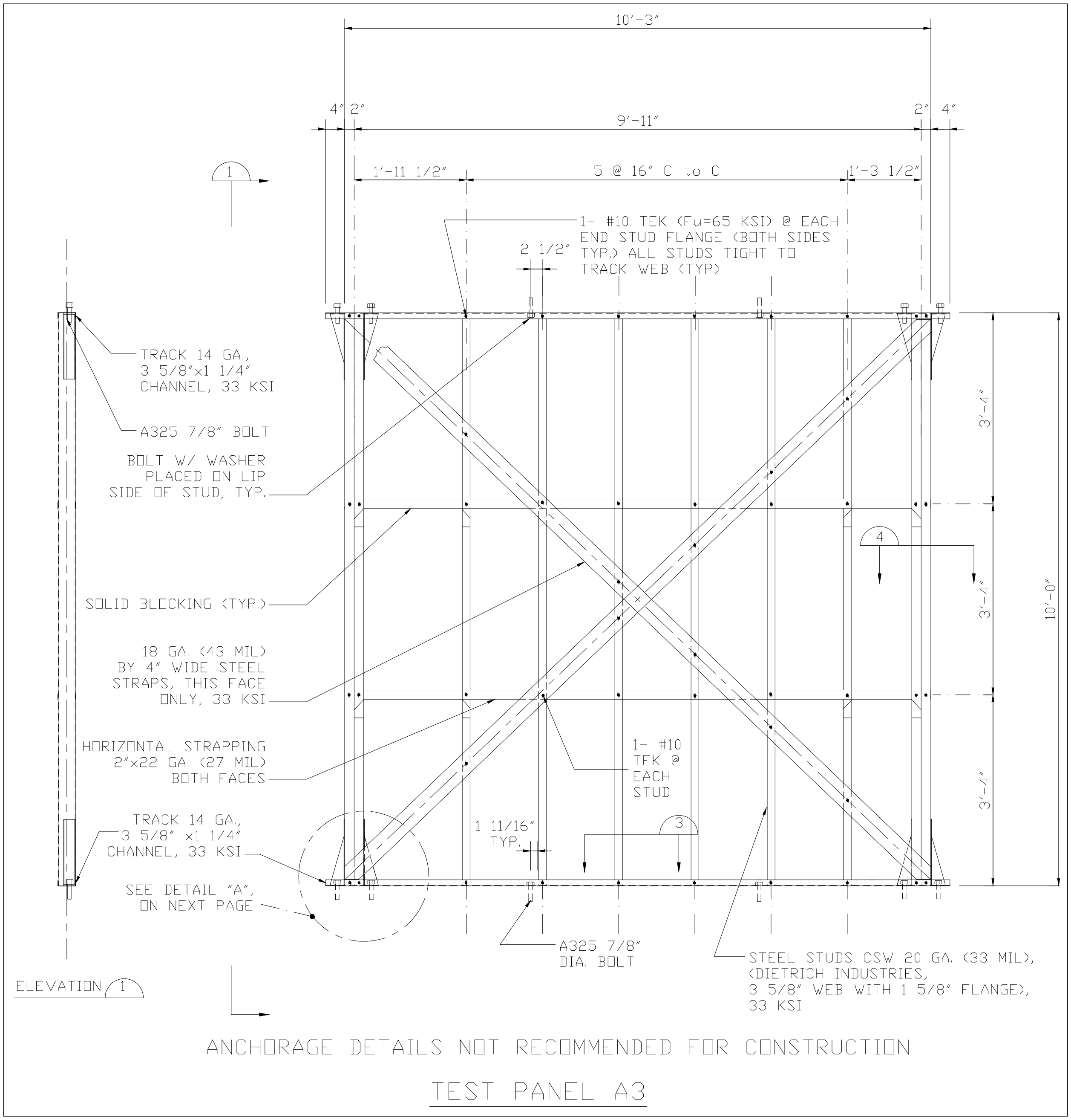


Figure A-7. Test panel A3 details.

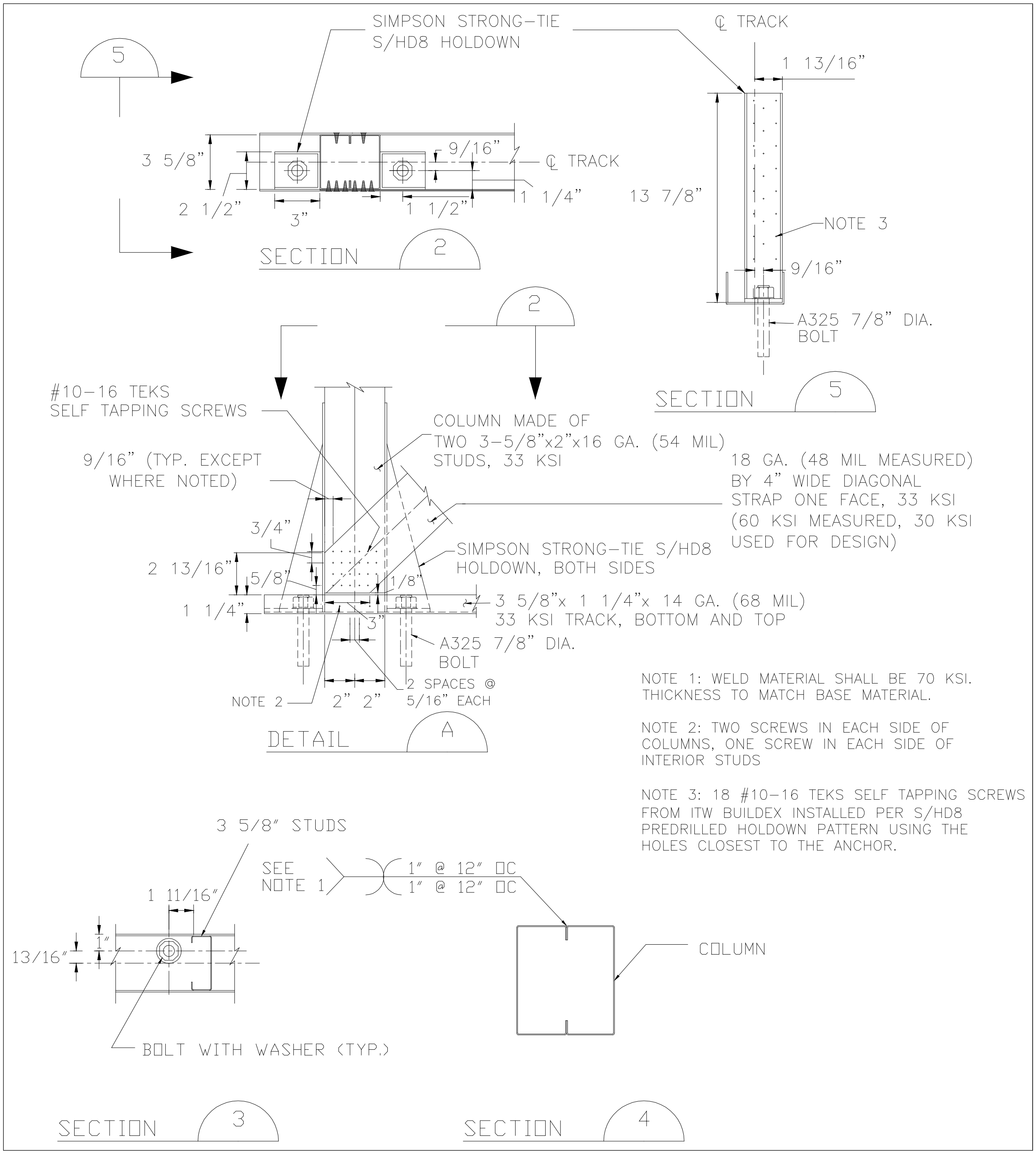


Figure A-8. Test panel C1.

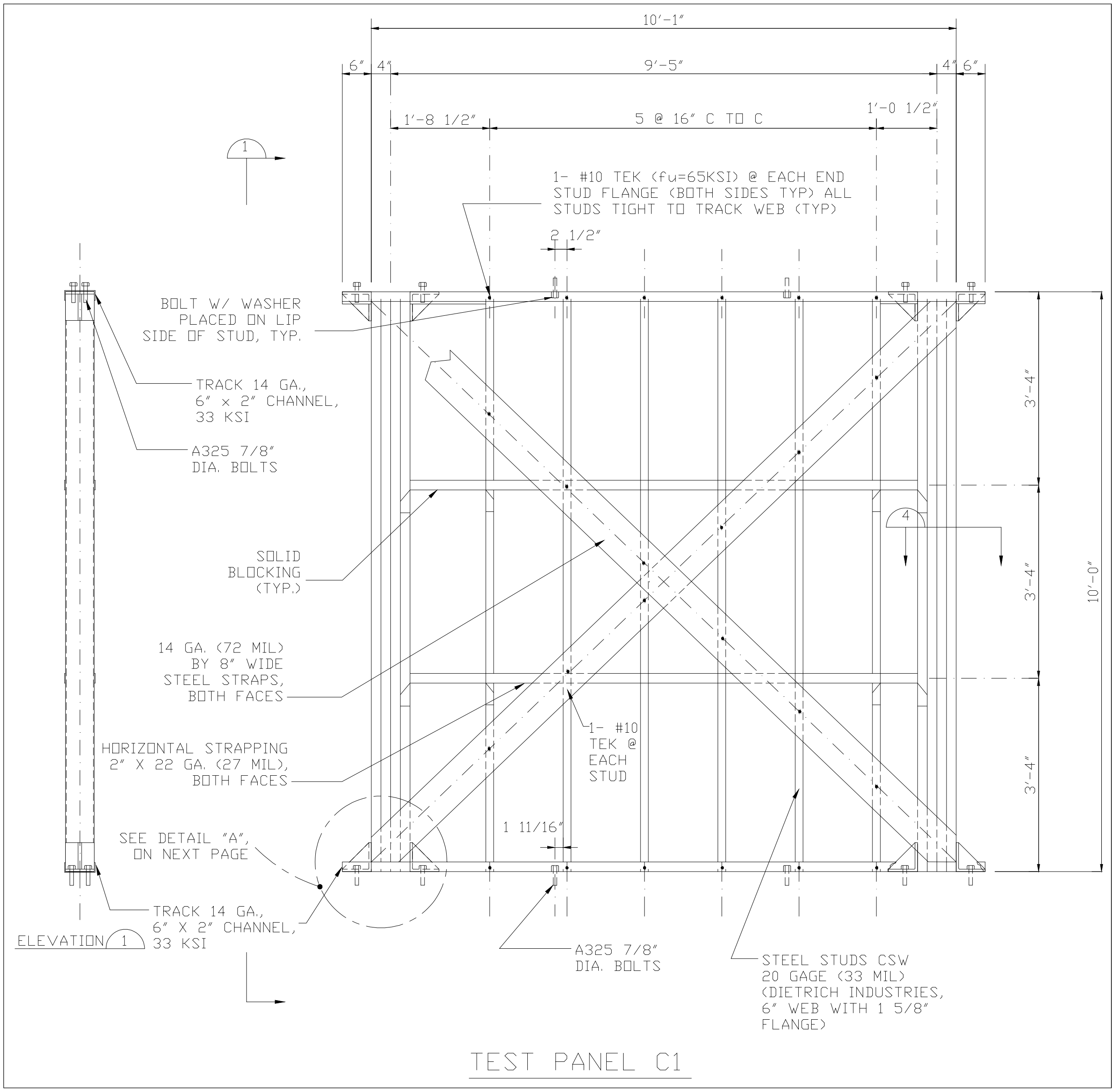


Figure A-9. Test panel C1 details.

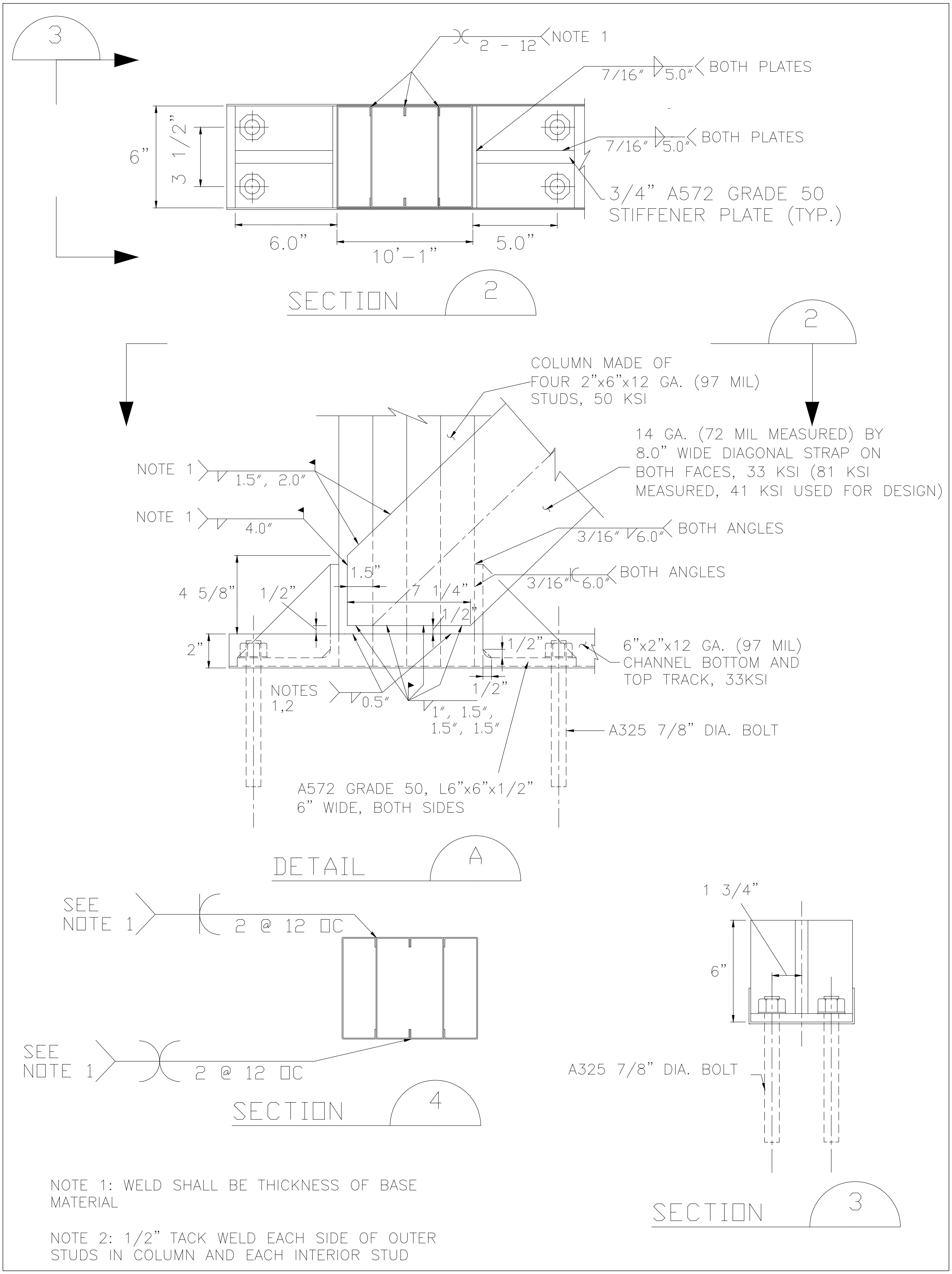


Figure A-10. Test panel D1.

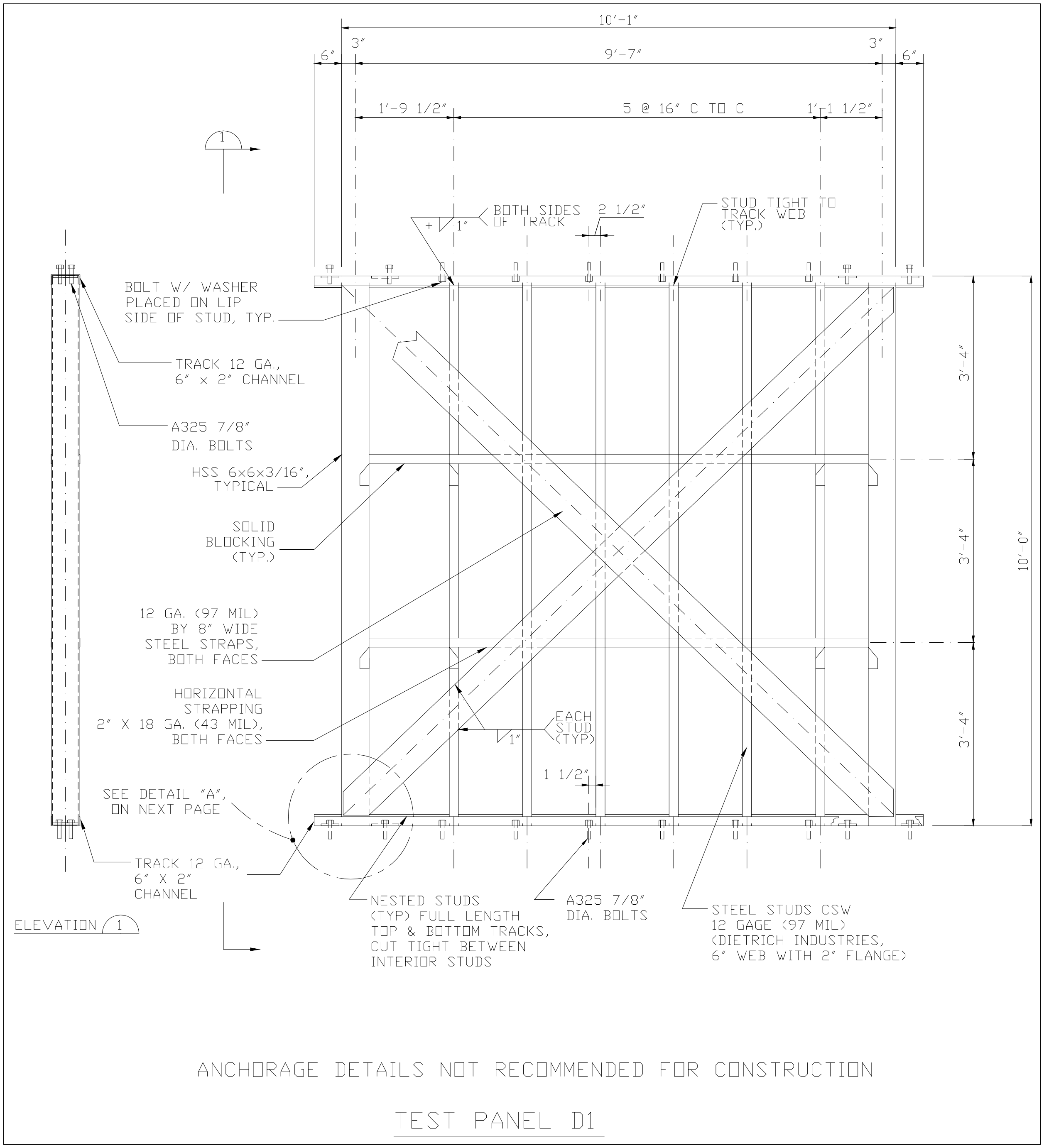


Figure A-11. Test panel D1 details.

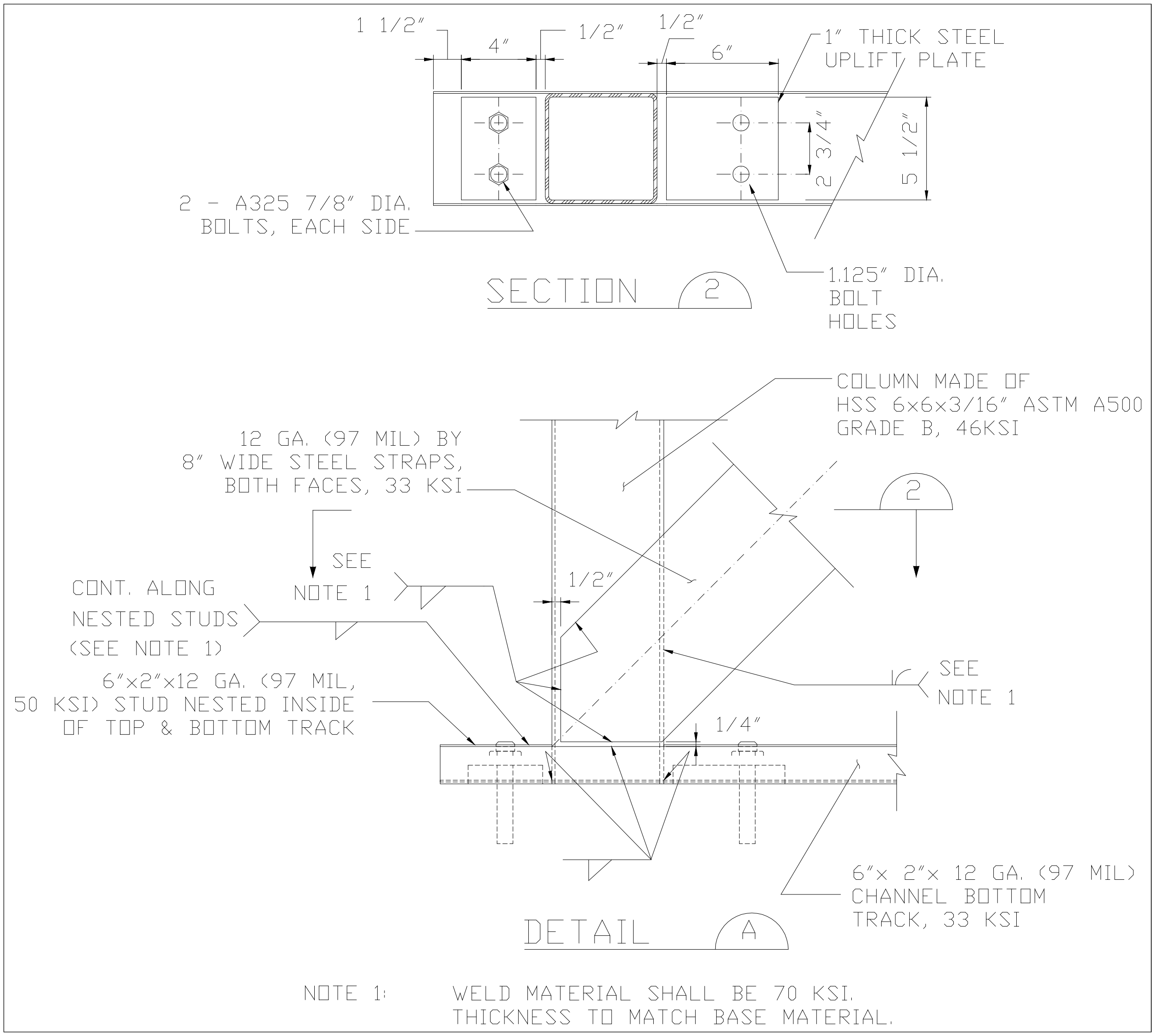


Figure A-12. Test panel D2.

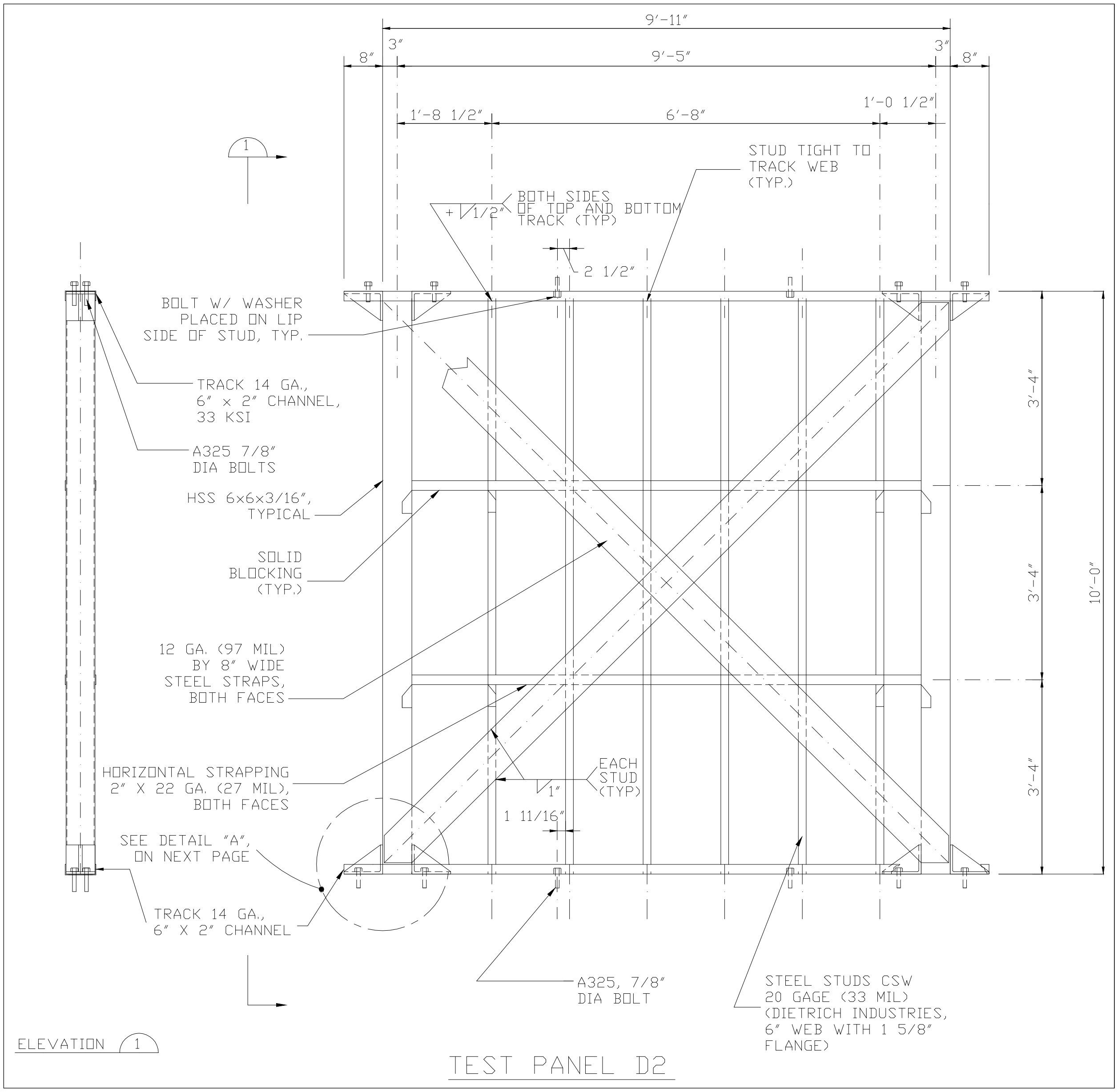
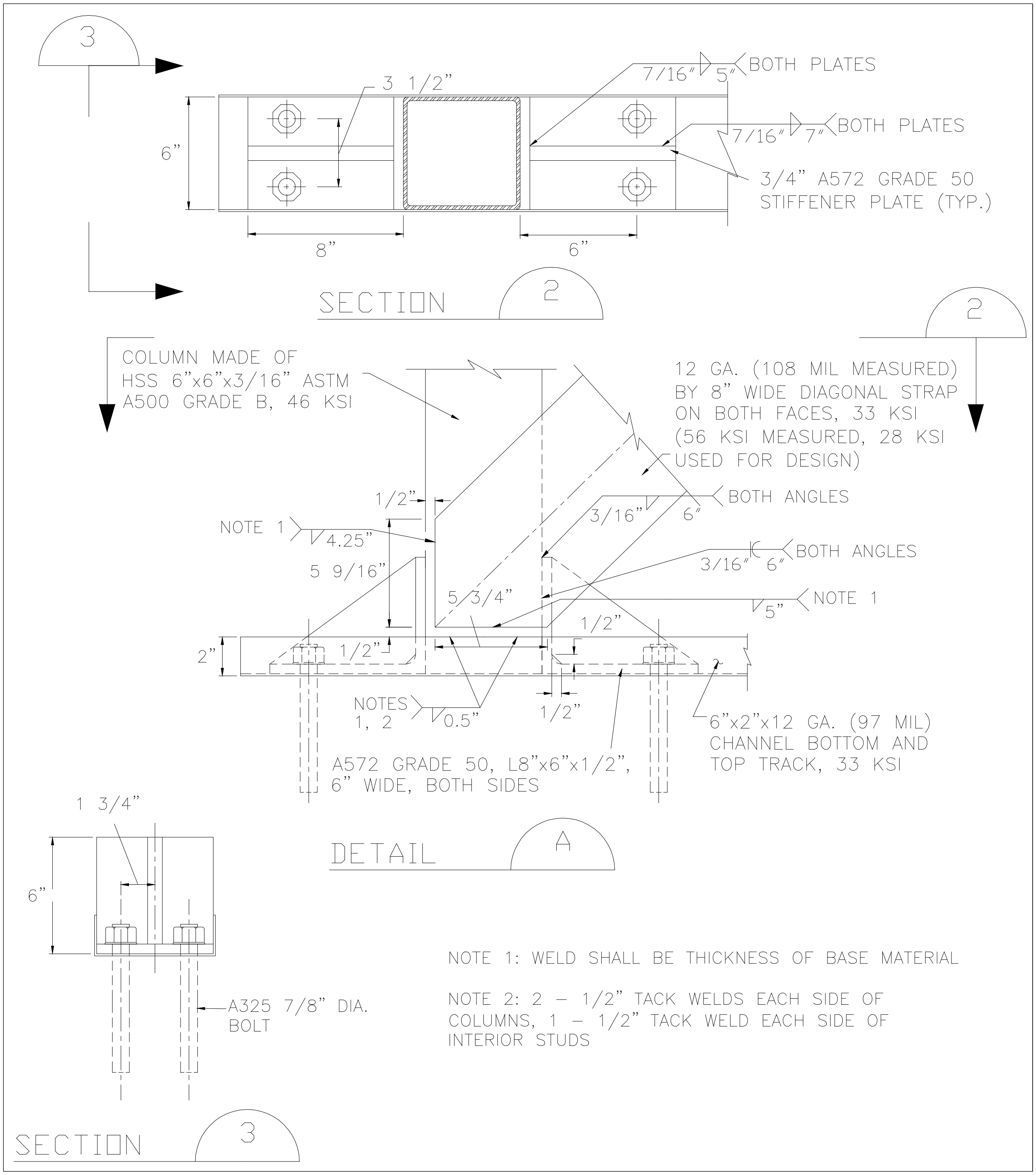


Figure A-13. Test panel D2 details.



Appendix B: Cold-Formed Steel Test Observations

The following tables provide details on damage progression with respect to lateral deformation for all monotonically and cyclically loaded test panels.

Table B-1. Panel A1a monotonic observations.

Shear Deflection (in.)	Location	Failure or Other Observation
1.0	North (right) column, bottom corner	Local buckling at column knockouts – effectively redistributed loads to other portions of column cross-section.
1.4	South column, top corner	Top of tension strap began detaching, screws failed in shear at column/track connection. Column tearing in shear at column/track connection. Column began to twist at this connection because of column/track failure at the diagonal strap face of column while the other column face continued to carry shear forces to the track.
2.0	Top horizontal strap at south column	Buckling of horizontal strap.
2.1	South column, top corner	All screws at south column – top strap/column connection failed in shear.
3.5	Bottom of third stud in from north	Interior stud twisted and buckled.
3.5	North column	Buckling at two knockouts at the center and near the top of the exterior face of the north column.
3.8	First and third stud in from south column	Buckling of interior studs near their top at the diagonal and horizontal straps.
3.85	Second & third stud in from north column	Buckling of interior studs near their bottom at the diagonal and horizontal straps.
4.5	Second, third & fourth stud in from south column	Buckling of interior studs near their bottom at the horizontal strap.
4.8	North column, top corner	Column bending at the top (local buckling on the north face??).
5.0	South column, top corner	Shear failure of screws at back face of column/track connection.
5.4	North column	Buckling at two knockouts at the center and near the bottom of the interior face of the north column.
6.9	First stud in from south column – top	Screws at stud/track connection failed in shear.
7.8	Second stud in from south column – top	Screws at stud/track connection failed in shear.
8.3	Bottom of north column	Buckling at knockout at the south interior face of the north column.
9.0	Top of south column	Screws (3 or 4) failed in shear at the column/track connection.
9.7	North column, 1 ft down from top	Studs making up the column begin to separate.
10.4	Entire panel	Gross buckling of the columns and interior studs.

Table B-2. Panel A1b cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.4	Diagonal straps	Straps yield.
0.6	North and south columns	Buckling at column knockouts.
1.2	Second stud	Buckling of knockout near bottom of stud.
1.2	South column, upper corner	Column is twisting at top track (exterior face).
1.2	South column, lower corner	One screw sheared at lower south corner of column/strap connection
1.6	South column, upper and lower strap connections	Column is tearing between track flange tip and strap connection. There is large twisting at the column's midspan.
1.6	North column, upper corner	Buckling of column at the top track.
2.4	North column, upper corner	Kinking of column at top corner.
2.4	South column, upper and lower corners	Major tearing of column at track flange tip; screws, strap and track connection holding well.
2.4	South column at blocking stiffeners	Large buckling of columns at stiffeners.
2.4	6th stud	Buckling of middle knockout.
3.2	South column, upper corner	Tear halfway across column face between strap and track connections. All screws on strap side of track connection have failed. Screws on opposite side are beginning to fail.
3.2	South column, lower corner	All track screws have failed; only 2 strap screws have failed.
3.2	All studs	Twisting of interior studs near the strap connections (torsional buckling). All stud/bottom track connections have failed.
3.2	Fourth stud	Kinking of the column in the front face of stud near the knockout. Buckling of stud near the bottom track.
4.0	Bottom track	Buckling of stud flanges (front).
4.8	Bottom track	Buckling of stud flanges (back).
4.8	South column, upper corner	All track screws have failed; part of torn column still attached.
4.8	North column, upper corner	Kinking of exterior column flange (back).
5.6	Columns & interior studs	Total collapse of structure.

Table B-3. Panel A1c cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.4	Diagonal straps	Yielding of straps.
0.4	Sixth stud – top	Flange buckling in front; possible fabrication error.
0.4	South column, lower corner	Slight elastic shift in alignment of bottom track fasteners from strap connection fasteners. Column is still twisting at the connection.
0.6	North column, lower corner	See 0.4 in., south lower corner.
0.6	Fourth stud – top	Distortion around knockout.
0.6	North column	Column starting to twist at mid-height.
0.8	North column, upper corner	Local buckling of column knockout (interior face).
0.8	South column, lower corner	Permanent offset between strap and track connection.
1.2	North column, upper corner	Buckling of knockout on exterior face of column.
1.2	North column, lower corner	Buckling of column base at bottom track connection.

Shear Deflection (in.)	Location	Failure or Other Observations
1.2	South column, upper corner	Fasteners shearing in strap/column connection. Gaps are forming between the studs in the column. The top edge of the column/strap connection is pulling away from the plane.
1.2	South column, lower corner	Rotation and translation of bottom outer edge of column.
1.2	Strap on back of panel	Strap yielding on back of panel.
1.6	South column	Column has torn at top and bottom track connections.
1.6	North column, lower corner	Local deformation of top row of fasteners at joint. Pictures were taken of interior of column through the knockout.
1.6	South column, upper and lower column connections	Column is torn on face between track and strap connection.

Table B-4. Panel A2a monotonic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.8	Diagonal strap	Buckling of strap.
2.1	North column, lower corner	Weld cracking at base (exterior front) of column at column/bottom track connection.
2.6	South column	Weld failure at lower column stiffener.
2.9	South column, upper corner	Weld fracture at column/track connection.
3.0	South column, upper corner	Fasteners breaking.
3.0	South column, lower column	Local buckling of knockout at exterior face.
3.6	North column, lower corner	Holes yielding at base of column.
4.1	Sixth stud – bottom	Stud rotated counterclockwise to the south.
4.4	North column	Local buckling of knockouts in exterior face of column.
5.3	North column, lower corner	Weld cracking at base of column (exterior face -back).
5.9	North column, lower corner	Weld failure along entire base of column/track connection.
6.2	South column, lower corner	Fasteners in column/strap connection failing in shear. Large buckling of column at track flange.
6.2	South column, upper column	Buckling of column flange at track.
7.1	First stud – top	Weld failure at nested stud connection.
7.4	North column, lower corner	Buckling of bottom track (back).
7.9	South column, upper corner	Weld failure at column/track connection.
8.2	North column, lower corner	Horizontal straps buckling.
9.2	South column, upper corner	Column is tearing at top track.
9.4	Interior studs – top	Fasteners failing in shear at top of studs (stud/track connection).
10.3	North column, lower corner	Bottom track yielding (buckling up) between sixth stud and column.
10.5	North column, lower corner	Tearing of web column at the base.
10.7	North column, upper corner	Fasteners connecting strap to studs on back shearing off.
11.2	North column, lower corner	Bottom track beginning to tear at column connection. (Front)
11.7	South column, upper corner	Column is pulling out of the connection.
12.6	Sixth stud – top	Fasteners shearing off at stud/top track connection.
13.2	Second stud	Second stud is twisting and fasteners are failing at both connections.

Shear Deflection (in.)	Location	Failure or Other Observations
14.2	South column	Local buckling of the knockouts.
14.2	South column, upper corner	Top track fractured at first column of fasteners.
Conclusion of test	North column, upper column	Weld at column/upper track connection fractured. Many fasteners at top and bottom of column sheared but the fastener heads remained affixed to the form.

Table B-5. Panel A2b trial cyclic observations (data are incomplete).

Shear Deflection (in.)	Location	Failure or Other Observations
0.5	Diagonal Straps	Buckling in both straps.
0.5	Sixth stud – bottom	Fasteners bending away from column at stud/ track connection.
0.5	Second stud – bracing	Fasteners on strap beginning to pull out.
0.5	First stud – bottom	Fasteners pulling away from base.
0.5	North column, lower corner	Slight bowing of second knockout from the bottom (exterior). Weld cracking at base of column at exterior face. Local buckling of knockouts near the bottom of the column.
0.5	North column, lower corner	Welds fracturing at top and base of column interior face. Buckling of all knockouts along exterior face.
0.5	South column, lower corner	Crack at welds in two directions into the column web (exterior).
0.5	South column, upper corner	Warping of knockouts. Local buckling of track near the back edge.
0.5	North column, upper corner	Large crack width at top of column across top weld. Buckling of top track near the back edge. Strap fasteners pulling out from studs.
6.4	North column, lower corner	Fracture through column at base.
6.4	Second stud – strap	Fasteners pulling out of studs.
6.4	North column, upper corner	Fasteners popping out of column/strap connection.
6.4	Fourth and fifth stud	Noticeable deformation of fastener holes in strap connections.
9.6	South column, upper corner	Bolts popping out of joint. Connection failure (back). Large web fracture at top of column at exterior face.
9.6	South column, lower corner	Buckling of column at base. Fasteners pulling out from column/strap connection.
9.6	Diagonal straps	Excessive buckling of bracing.
9.6	North column, upper corner	Fracture through column web on exterior face.
13.2	South column, upper corner	Column fracture completely through web. Fasteners popping out at joint
13.2	First, second, & third studs	Top fasteners in studs.
13.2	Stud –top	Failed.
15.0	South column, lower corner	Weld failure along bottom of track in nested stud. Column buckling on interior face.
15.0	Fourth & fifth studs - top	Studs twisting.

Table B-6. Panel A2b cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.6	Diagonal straps	Straps yielding.
1.2	South column, lower corner	Weld crack at stiffener front and back.
1.6	North column, upper/lower corner	Weld fracture at column.
2.4	South column, lower corner	Weld crack through stiffener.
3.2	North column, lower corner	Screw head sheared off (top row upper). Buckling of bottom track. Failure of 2 track screws at buckle point.
4.0	North column, lower corner	Buckling at column cutout. Buckling at column base.
4.8	North column, lower corner	Screw failure at top track (top row). Screw shearing at bottom track.
5.6	South column, upper corner	Fracture of column weld at top track. Tearing of column.
6.4	South column, lower corner	More weld failures at bottom of column. Column base beginning to bend.
6.4	South column, upper corner	Weld of nested stud failed.
6.4	South column	Weld fracture of column at track connection (interior).
6.4	South column, lower corner	Tearing of lower track near column intersection.
7.2	North column, lower corner	Upward buckling of track web at base of column at outer face. Tearing of track flange near weld at column base.
7.2	South column, lower corner	Fastener failure at column base. Bottom track lifting up off base beam (back).
8.0	South column, lower corner	Fasteners failing at column base (back).
8.0	Sixth stud - top	Fasteners failing at stud/strap connection.
8.0	South column, upper corner	Top track tearing at column intersection.
8.0	North column, lower corner	Fastener failed at track. Uplift of track is causing tearing of bottom track flange. Total failure of column base weld. Multiple fastener failure at north end (back).
8.8	North column - exterior	Buckling of exterior face between knockouts.
8.8	Sixth stud – top	Fastener connecting strap failed (back).
8.8	North column, upper corner	Weld tearing at column/top track connection (exterior).
9.6	North column, upper corner	Tearing of weld at column/top track connection (back).
9.6	Front diagonal strap	Various fasteners shearing at stud connections.
9.6	North column, upper corner	Four fasteners have failed in shear at top track/column connection.
9.6	North column, lower corner	Almost all fasteners in column/bottom track have failed (front).
10.4	Third stud	Fasteners shearing at stud connections.
11.2	North column, upper corner	Fasteners failing at column connection to upper stud.
11.2	Sixth stud – bottom	Fasteners failing at bracing connections.
11.2	North column, upper corner	Weld at column/top track connection failing (still maintaining some load). Fastener failed in shear at column/top track connection.
11.2	Sixth stud – bottom	Fastener failed at strap connection.
11.2	South column, upper corner	Top track tearing at column intersection (back).
11.2	Interior studs	Fasteners failing (back).
11.2	South column, upper corner	Top track tearing at top (back).

Table B-7. Panel A2c cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.4	Diagonal straps	Straps yielding.
0.6	North column, upper corner	Exterior weld crack at column/top track connection.
0.6	South column, lower corner	Weld beginning to fail at column/bottom track connection at panel exterior.
0.6	South column, lower corner	Fastener at strap/column connection failure.
1.2	North column, upper corner	Exterior weld continues to crack at column/top track connection.
1.6	Sixth stud - top	Strap fastener shear (front and back).
1.6	South column, lower corner	Bottom track beginning to buckle upwards, followed by screw failure.
2.4	North column, lower corner	Weld fracture at column/base track connection.
2.4	Track	Fasteners pulling out (back).
2.4	South column, lower corner Exterior	Weld at column/bottom track connection failed.
3.2	North column, lower corner	Base track torn from corner to fastener. Track buckling out at column intersection at back face.
3.2	South column, upper corner	Screws sheared at column/top track connection.
4.0	Diagonal strap	Strap buckling (front).
4.0	South column	Top track beginning to tear at column connections. Top weld at upper track/column connection (exterior).
4.0	South column, lower corner	Weld failure along bottom track connection (interior). Entire bottom track failing along exterior column face.
4.0	North column, upper corner	Top track buckling at column. Top track tearing along fastener line.
4.0	North column, lower corner	Welds fracturing at base. Fasteners shearing at bottom of track at back face.
4.0	Upper/north lower corners	Bending of top track away from beam at top and bottom.
4.8	North column, lower corner	Brittle weld fracture along entire base. Bottom track torn along fastener line at back face.
4.8	Sixth stud - bottom	Strap fastener failing.
5.6	Second stud	Fastener failed at strap connection.
5.6	North column, lower corner	Bottom track pulling up from beam.
5.6	North column, upper corner	Weld failure along base (exterior).
5.6	Right column	Local buckling near knockouts.
5.6	North column, lower corner	Column tearing at base near exterior weld.
5.6	South column, lower corner	Bottom track shearing and weld failure through track.
6.4	South column, upper corner	Exterior weld at upper track/column connection failed completely.
6.4	South column, lower corner	Slight buckling of column near base. Bottom track has failed (Back)
6.4	North column, lower corner	Buckling of interior track/column connection.
6.4	North column, upper corner	Column tearing near weld (exterior).
6.4	Sixth stud - top	Stud fastener to top track sheared off. Fasteners have sheared at top track – front and back face.
7.2	South column, upper corner	Fasteners failing at column connection – back face
8.0	South column, lower corner	Track is beginning to uplift from beam.
8.0	Third stud	Fastener failure at strap connection.
8.0	North column, lower corner	Column buckling at track (exterior). Column starting to gap at base.

Shear Deflection (in.)	Location	Failure or Other Observations
8.0	North column, upper corner	Fasteners shearing at track/column connection.
8.8	Third stud – strap	Fasteners shearing at strap connection.
9.6	Third stud – strap	Fasteners shearing at strap connection.
9.6	North column, lower corner	Bottom track flange beginning to tear.
9.6	South column, lower corner	Fasteners at column/strap connection failing.
10.4	North column, lower corner	Bottom track at column base has sheared. Tear continuing along the bottom track.
11.2	First stud- top	Fasteners at strap connection shearing.
12.8	Interior studs - bottom	Studs fail along bottom track at screws.

Table B-8. Panel A3a monotonic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
Loading to the South – Positive Direction on Data Plots		
1.3	South column, top corner	Screws failed between column and track.
2.8	South column, top corner	Major distortion of column.
3.5	South column, top corner	Column pulled away from anchor.
4.0	South column, top corner	Screws between column and strap failing.
5.0	South column, top corner	Strap failed.
Loading to the North – Negative Direction on Data Plots		
0.7	North column, top corner	Screws failed between column and track.
1.15	South column, bottom corner	Screws failed between column and track.
2.2	South column, bottom corner	Column pulling away from strong-tie.
2.8	South column, bottom corner	Column buckling around strong-tie.
5.9	Interior	Studs buckle.
6.3	South column, bottom corner	Column pulling away from strong-tie.
10.0	South column, bottom corner	Slow progression of crushing of double stud between anchors.

Table B-9. Panel A3b cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.45	South column, bottom corner	Screws failed between column and track.
0.9	North column, bottom corner	Screws failed between column and track.
1.2	South column, top corner	Strap pulling away from column.
1.2	North column, bottom corner	Bowing of column.
1.8	South column, top corner	Buckling of column.
1.8	South column, bottom corner	Strap pulling away from column.
2.4	South column, bottom corner	Buckling of column.
2.4	North column, top corner	Buckling of column.
3.6	South column, top corner	Screws fail between strap and column.
3.6	Interior	Buckling of interior channels and partial screw pullout.

Shear Deflection (in.)	Location	Failure or Other Observations
3.6	North column, bottom corner	Screws fail between strap and column. Major tearing of column away from anchor
4.8	North column, top corner	Crushing of column against anchor.
4.8	South column, top corner	Channels of column start pulling apart.
4.8	South column, bottom corner	Screws pull out.
6.0	South column, bottom corner	Strap failure by pullout.
6.0	North column, bottom corner	Strap failure by pullout.

Table B-10. Panel A3c cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.45	North column, bottom corner	Column flexing with strap tension.
0.6	North column, top corner	Screws failed between column and track.
0.9	South column, Top/bottom corner	Screws failed between column and track.
0.9	North column, top corner	Screws failed between column and track.
1.2	North column, top corner	As connection is stressed, back of column wraps back around anchor.
2.4	North column, top/bottom corner	Screws between column and strap nearly pulling out.
3.6	North column, bottom corner	Strap net area failure at connection.
4.8	North column, top corner	Screws between column and strap pull out.
6.0	South column, bottom corner	Lots of screws showing between strap and column.
7.2	Interior	Interior studs well buckled.

Table B-11. Panel C1a monotonic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0	Unknown	Weld between track and column failed at application of vertical load.
0.5	North column, top corner	Weld between track and column failed.
0.75	South column, top corner; North column, bottom corner	Major deflection of strap (compression).
2.25	South column, bottom corner	Tearing of the strap.
3.5	South column, bottom corner	Rear strap failed.

Table B-12. Panel C1b cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.6	North column, top corner	Weld crack.
0.8	North column, bottom corner	Weld cracked on both sides
1.2	North column, bottom corner	Small cracking at top of angle/column connection.

Shear Deflection (in.)	Location	Failure or Other Observations
1.6	North column, bottom corner	First three welds cracked.
1.6	South column, bottom corner	Weld failure.
2.4	North column, top corner; North column, bottom corner	Complete tear of strap.
3.2	North column, bottom corner	Angle splitting from column.
4.8	North column, bottom corner	Complete tear at angle/column connection.

Table B-13. Panel C1c cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.3	North column, top corner	Track weld failure (front).
0.4	North column, top corner	Track weld failure (back).
0.6	North column, bottom corner	Track weld failure (front/back).
1.6	South column, bottom corner	Track weld failure (back).

Table B-14. Panel D1a monotonic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.95	Diagonal straps	Straps yielding.
1.0	South column, upper corner	Crack forming in weld at column/top track connection. (Exterior) Track bowing away from beam.
1.37	North column, lower corner	Track pulling up.
1.6	North column, lower corner	Welds fracture at nested studs.
2.1	North column, lower corner	Bottom track tearing at weld (back). Weld at base of column fracture at exterior face.
3.8	South column, upper corner	Top track bowing away from beam.
4.0	South column, upper corner	Welds at nested stud have failed.
4.0	North column, lower corner	Track torn through to base.
4.43	North column, lower corner	Track base tearing. Track flanges bowing out between welds.
6.7	South column, lower corner	Bottom track pulling away from beam.
8.0	South column, lower corner	Weld failing at nested stud/column connection.
8.0	South column, upper corner	Top Track beginning to tear. Buckling of top track is causing it to crush against tube column.
8.46	Sixth stud - straps	Strap exhibiting a hump between welds on same stud.
9.3	North column, lower corner	Nested stud flanges are buckling against tube.
10.2	North column, lower corner - Strap	Strap tearing on back face.
10.68	North column, lower corner – Strap	Strap tom through on back.
10.68	Interior studs - bottom	Weld at base of interior stud fail.
11.24	North column, lower corner-strap	Front strap torn through on front.

Shear Deflection (in.)	Location	Failure or Other Observations
11.24	North column, lower corner	Bottom track bowing up.
11.24	Third stud	Strap weld fails.
12.0	First stud - bottom	Weld failure at strap connection.
12.5	Fifth stud - bottom	Stud buckling at base.
13.14	North column, lower corner	Bottom track buckling at base.
13.6	Interior studs - bottom	Massive buckling at stud/bottom track connections.
14.0	North column, lower corner	Weld fracture/tearing through track.
14.45	Diagonal strap	Strap weld failures at stud connections.
14.45	Base beam	Dishing effects at bolts.

Table B-15. Panel D1b cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.4	Diagonal straps	Straps yielding.
0.6	North column, upper corner	Weld begins to fail at column/top track connection at exterior face.
1.2	North column, upper corner	Welds across flanges fail at nested studs.
1.2	South column, upper corner	Welds across flanges fail at nested studs.
1.6	South column, lower corner	Welds across flanges fail at nested studs.
2.4	South column, lower corner	Bottom track beginning to uplift under outside of column.
2.4	South column, upper track	Tear in top track beginning to propagate in track near column at exterior face.
3.2	North column, lower corner	Weld begins to fail at flange of nested stud. Bottom track lifting up.
3.2	Second stud - strap	Weld at strap connection begins to fail at front and back face.
3.2	Third stud - strap	Weld failure at strap connection at back face.
4.0	North column, lower corner	Track tearing near column connection at front face. Tearing of bottom track along side at front face.
4.0	Second stud - strap	Weld at strap connection fails at front face.
4.0	North column, upper corner	Top track tearing. Flange of nested stud beginning to buckle.
4.0	South column, upper corner	Tears forming at column/top track weld connection at front and back faces. Buckling of nested stud flange due to prying action against column.
4.8	Sixth stud - bottom	Weld failure at strap connection.
4.8	Diagonal straps	Straps bowing between welds of same stud.
4.8	Second stud - bottom	Weld at strap connection failed at front and back faces.
5.6	North column, lower corner	Bottom track buckling out at column.
5.6	Sixth strap - bottom	Weld beginning to fail at strap connection.
5.6	North column, upper corner	Top track pulling away from beam.
5.6	South column, lower corner	Weld failure of column/bottom track connection at exterior face.
6.4	South column, upper corner	Weld failure of column/bottom track connection at exterior face.
6.4	North column, lower corner	Sudden weld fracture at column/bottom track connection at exterior face. Strap/column connection beginning to fail at back face. Bottom track buckling out at back face.
7.2	North column, upper corner	Strap/column connection beginning to fail. Weld failure at column/top track connection at exterior face.
7.2	Sixth stud - bottom	Stud beginning to tear near bottom track. Stud buckling at base.

Shear Deflection (in.)	Location	Failure or Other Observations
7.2	Fifth stud - bottom	Weld fails around strap.
8.0	South column, upper corner	Top track flanges beginning to buckle near column/top track connection.
8.0	Interior studs	Welds at strap/stud connection fail. Studs tearing near bottom track.
9.6	North column, lower corner	Strap beginning to tear.
9.6	North column, upper corner	Weld failure at column/top track connection at exterior face.
10.4	North column, lower corner	Strap tearing near column in two places. Weld failure near tear column/strap connection.
11.2	North column, lower corner	Sudden failure of back strap.
11.2	North column, upper corner	Strap beginning to tear.
12.0	North column, upper corner	Strap beginning to tear on back near column.
13.6	Interior studs	Local buckling near knockouts.
13.6	Diagonal straps	Three straps have failed.

Table B-16. Panel D1c cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.4	Diagonal straps	Straps yielding.
0.8	South column, lower corner	Inside weld of nested stud beginning to fracture.
1.2	South column, upper corner	Weld on top of nested stud beginning to fracture.
1.6	South column, lower corner	Tearing of bottom track at column interior.
1.6	North column, upper corner	Crack in weld at column/nested stud connection at front face.
2.4	North column, Lower Column	Weld at column/bottom track failed. Tear forming across nested stud near column connection at exterior face. Buckling of bottom track away from the nested studs. Uplift of track at column (front).
2.4	First stud	Strap weld fracturing (back).
3.2	North column, lower corner	Tear at weld propagating into bottom track.
3.2	South column, upper corner	Top track beginning to tear (exterior).
4.0	First & second stud	Welds to strap beginning to fail (back).
4.0	Third stud	Weld to strap failed (front).
4.0	North column, lower corner	Large crack through weld at the top of Bottom track near column connection at exterior face.
4.0	North column, Upper Column	Tear forming in top track at column connection at exterior face.
4.8	South column, lower corner	Bottom track beginning to lift off beam. Bottom track beginning to tear near column interior face.
4.8	First stud - bottom	Weld to bottom track beginning to fail.
4.8	South column, lower corner	Buckling of track at base.
4.8	First stud - top	Strap weld fails at back face.
5.6	North column, upper corner	Weld beginning to tear at strap/column connection at back face.
5.6	South column, lower corner	Weld at column base continues to fail.
5.6	First stud - bottom	Strap weld failure (back).
5.6	Third stud	Strap weld failure (front).
6.4	Sixth stud - top	Weld failure at strap connection at front and back face.
6.4	Fifth stud - top	Top track beginning to tear and twist at top track/stud connection.

Shear Deflection (in.)	Location	Failure or Other Observations
6.4	South column, upper corner	Top track tearing around column interior. Buckling of top track around column interior.
6.4	South column, lower corner	Small weld fracture of column/strap connection (front).
7.2	South column, lower corner	Total weld failure of column to bottom track connection at back face.
7.2	North column, upper corner	Weld failure along column/top track connection at back face.
7.2	Fourth and fifth stud	Welds to strap fails.
8.0	First stud	Strap bowing between welds on same stud. Short panel welds fail. Short panel near stud is rotating down.
8.0	Interior studs - bottom	Studs buckling near bottom track.
8.0	Second stud - strap	Strap beginning to tear (back).
8.0	First stud - bottom	Stud tearing near bottom track.
8.0	Second stud - top	Weld failing at stud/top track connection.
8.0	Fourth stud - top	Stud tearing near top track (front).
8.0	First stud - top	Stud tearing near top track (front).
8.0	Fifth stud - bottom	Stud tearing at stud/bottom track connection.
8.0	Sixth stud - top	Weld at strap connection failed (back).
8.0	Diagonal strap at fifth stud	Strap beginning to tear at stud connection on front face.
8.8	Second and third stud – top	Weld beginning to fail (track/top track connection).
8.8	Interior studs	All studs tearing along bottom track.
8.8	Diagonal strap – north column, upper corner	Strap beginning to tear at column connection (front).
9.6	North column, upper corner	Welds at strap connections beginning to tear at front & back faces.
9.6	South column, upper corner	Sudden weld failure at column/top track connection.
9.6	Interior studs	All studs tearing along top track.
9.6	South column, upper corner	Top track pulling away from beam.
12.0	North column, upper corner	Strap beginning to tear near column at back and front face.
12.8	Sixth stud at strap	Tear is propagating at first stud weld.
13.6	North column, lower corner	Two tears forming in straps near column at front face.
13.6	First stud	Short panel weld fails causing panel to swing down.

Table B-17. Panel D2a monotonic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
Loading to the North – Positive Direction on Data Plot		
0.7	South column, bottom corner	Track weld failure.
0.8	North column, top corner	Track weld failure.
1.7	North column, top corner	Brittle fracture of strap.
1.9	South column, bottom corner	Brittle fracture of strap (back side).
Loading to the South – Negative Direction on Data Plot		
0.5	South column, top corner	Track weld failure.
1.2	South column, top corner	Vibration noise.

Table B-18. Panel D2b cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.4	North column, bottom corner	Track welds failed (back – north side of column).
0.6	North column, top corner	Track welds failed (back).
0.8	North column, bottom corner	Track welds failed (back – south side of column).
2.4	South column, top corner	Back strap failed.
2.4	South column, bottom corner	Front strap broke.
4.0	Unknown	Last strap broke.

Table B-19. Panel D2c cyclic observations.

Shear Deflection (in.)	Location	Failure or Other Observations
0.3	North column, top/bottom corner	Track weld failure (front and back).
2.4	North column, bottom corner	Brittle tear of strap.

Appendix C: Prototype Shear Panels for Cold-Formed Steel Seismic Design

This appendix provides tabular data for the selection of possible prototype shear panels that may be used in the seismic design of cold-formed steel structures. These panels were developed for the example problem presented in Chapter 12, using the design recommendations presented in Chapter 11. Each shear panel given in Table C-1 is defined in Figure 12-9, Figure 12-11, Figure 12-13 and Figure 12-12, as indicated in Table C-1. The panel shown in Figure 12-14 was not selected for the example problem, but meets all the requirements of these design recommendations.

Definition of terms

The prototype shear panels given in Table C-1 shall be used based on the following definition of terms. For these panels, the values of GL_{max} and GL_{min} were defined at which the demand reached the capacity for one of the limiting equations given below.

$\varphi_t Q_{sy}$ = the lateral shear panel design strength that must exceed the maximum story shear per shear panel, including the effects of torsion, defined and limited by Equation 11-37.

GL_{max} = the maximum gravity load per shear panel, defined by Equation 11-18 and limited by Equations 11-55 or 11-56.

GL_{min} = the minimum gravity load per shear panel, defined by Equation 11-19 and limited by Equations 11-100, 11-108, 11-116, 11-117, 11-118, 11-120, or 11-122.

Prototype panel load table

Table C-1 provides the tabular data needed to select prototype shear panels.

Table C-1. Prototype shear panel load capacities.

Panel Figure	Lateral Design Strength $\phi_t Q_{sy}$ (kips)	Max Gravity Load/ Panel GL_{max} (kips)	Min Gravity Load/ Panel GL_{min} (kips)
Figure 12-9	8.1	35.2	-3
Figure 12-11	19.0	36.5	-4
Figure 12-13	24.2	88.5	10
Figure 12-14	24.2	224	5

Appendix D: Seismic Qualification Procedure and Acceptance Criteria for Other Shear Panel Configurations

This appendix presents the test procedure, acceptance criteria, and documentation requirements needed to demonstrate the acceptability of cold-formed steel shear panel configurations that are different from the specific system defined in Chapter 11. Acceptable configurations are limited to cold-formed steel shear panels that use diagonal straps or full panel sheets as the lateral-load-resisting elements. The columns shall be constructed with cold-formed or hot-rolled structural steel. This procedure applies to the qualification of a prototype of the specific panel that will be used in construction. Qualification requires the testing of three specimens. All panel tests shall represent full panel system tests of all the panel components including connections and anchors.

Coupon tests of all test panel materials

Coupon tests shall be performed on all materials that may contribute to the structural performance of the test panels. At least three coupons shall be tested from each lot of each type of material. Coupons shall be prepared and tested following the provisions of ASTM A370 (ASTM 2014b). Materials that contribute to the ductility of the shear panels shall have a total elongation of at least 10% for a 2 in. gage length. All coupon test results shall be plotted in a test report, in terms of stress versus strain. All coupon test results shall also be summarized in a table in the format shown in Table D-1. The data in this table shall be the average value of the three or more coupons of the particular component.

Table D-1. Tabular format for coupon test results.

Structural Component of Coupon	Design Yield Stress (MPa or ksi)	0.2% Offset Yield Strain* (mm/mm)	0.2% Offset Yield Stress* (MPa or ksi)	Maximum Load Strain (mm/mm)	Maximum Stress (MPa or ksi)	Max Stress 0.2% Offset Yield Stress
Component #1						
Component #2						

See Chapter 4 for the definitions of 0.2% offset yield strain and stress.

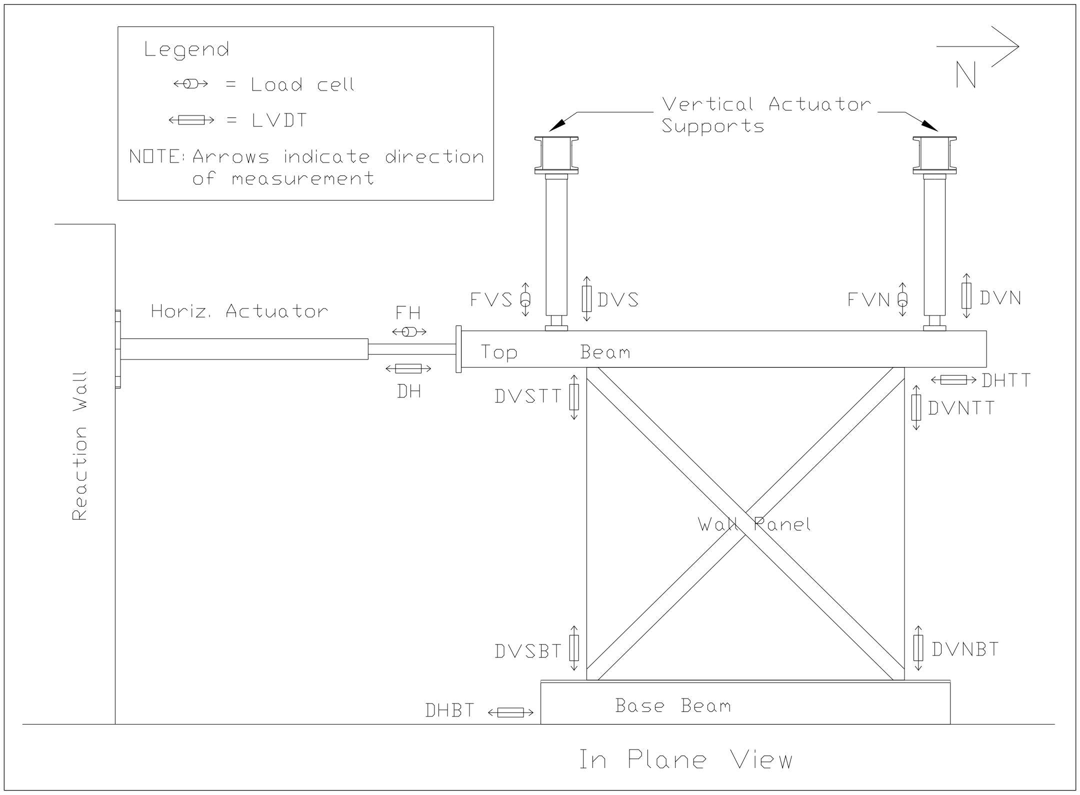
Coupon test of all field panel materials

Coupon tests shall be performed on all materials that contribute to the structural performance of the field panels. The field panels shall be identical to the prototype-tested panels. At least three coupons of each material shall be tested. Coupons shall be prepared and tested following the provisions of ASTM A370 (ASTM 2014b). Materials that contribute to the ductility of the shear panels shall have a total elongation of at least 10% for a 2 in. gage length. All coupon test results shall be plotted in a test report, in terms of stress versus strain. All coupon test results shall also be summarized in a table in the format shown in Table D-1. The data in this table shall be the average value of the three or more coupons of the particular component. The field diagonal straps or full panel sheets shall have a coupon yield stress (0.2% offset) not greater than 5% above or not less than 10% below the test panel coupon yield stress (0.2% offset). The field material coupons for all other structural elements shall have coupon yield stress (0.2% offset) not less than the test panel coupon yield stress (0.2% offset).

Test configuration

Full-scale test panels shall be tested with both monotonic (push-over in one direction) and cyclic loading. The panels shall be anchored to a base beam and top beam in a manner representative of the field installation. The base beam shall resist any slippage, out-of-plane movement or rotation in any direction. Vertical load shall be applied to the shear panel through the top beam, at a level representative of potential gravity loads in the field. The amount of vertical load applied should consider the worst-case condition for the most vulnerable panel components. For example, the minimal vertical load may provide the most severe loading for the anchors, while the maximum vertical would provide the worst-case loading for column buckling. This vertical load shall be held constant throughout each test. The top beam shall be held horizontal during all tests, as this represents the field conditions when the panel is assembled in a building. Figure D-1 shows the test configuration and instrumentation plan for shear panels tested at ERDC-CERL, to illustrate the load configuration. In the ERDC-CERL tests, stroke control was used to keep the two vertical actuators at the same length, which held the top beam horizontal. The combined vertical force was held constant by using the test control system (which was done manually for earlier tests).

Figure D-1. Schematic drawing showing sensor locations.



Instrumentation

Table D-2 defines the instrumentation required for all shear panel tests. Figure D-1 shows the location and orientation of all sensors, and Table D-2 describes the purpose of each sensor. The purpose of most gages is to ensure that no unwanted motion takes place and for test control. The only data used in reporting panel performance are the first, second, third, and fourth channels in Table D-2. The vertical actuator force measurements (FVS and FVN in Table D-2 and Figure D-1) are required to define total shear force when deflections reach large amplitudes, at which point the horizontal components of these forces become significant. This total shear force, TSF, is determined as follows:

$$TSF = FH - TVF\left\{sin\left[arctan\left(\frac{DH}{L}\right)\right]\right\} \quad \text{(Eq D1)}$$

where

FH = the measured horizontal actuator force (see Table D-2 or Figure D-1).

TVF = the total vertical actuator force, equal to FVS plus FVN (Table D-2 or Figure D-1).

DH = the measured horizontal displacement (Table D-2 or Figure D-1).

L = the length of the vertical actuators, with vertical load applied but no horizontal displacement.

Table D-2. Cold-formed steel shear panel instrumentation.

Channel #	Sensor Type	Measurement, Direction, Location and Symbol	Purpose
1	Load cell	Force Horizontal, FH	Horizontal actuator load measurement
2	LVDT	Deflection Horizontal, DH	Horizontal deflection, shear panel deformation
3	Load cell	Force Vertical South, FVS	Manual vertical load control (25k total load w/#5)
4	LVDT	Deflection Vertical South, DVS	Stroke (tied to #6)
5	Load cell	Force Vertical North, FVN	Load (summed with #3, for 25k total load)
6	LVDT	Deflection Vertical North, DVN	Controlled by #4 stroke feedback
7	LVDT	Defl Horiz Bot Track, DHBT	To ensure no slippage
8	LVDT	Defl Vert South Bot Track, DVSBT	To ensure no uplift
9	LVDT	Defl Vert North Bot Track, DVNBT	To ensure no uplift
10	LRDG* (20")	Defl Horiz Top Track, DHTT	Check for shear panel deformation - same as #2
11	LRDG (10")	Defl Vert South Top Track, DVSTT	Vertical panel/column deformation & rotation check
12	LRDG (10")	Defl Vert North Top Track, DVNTT	Vertical panel/column deformation & rotation check

*Linear resistance deflection gage, or cable-extension position transducer.

Test requirements

For each shear panel qualified, three specimens shall be fabricated and tested. This requirement assumes only minor variation in panel performance for a given shear panel. If large variations occur, more than three specimens shall be tested and a statistical evaluation of panel performance may be required. For panels with minor variation, one specimen shall be tested monotonically and two shall be tested cyclically, as defined below. All tests, both monotonic and cyclic, shall use stroke control, loading the panels laterally at a constant displacement per minute. The vertical load shall be held constant and the top beam shall be held horizontal throughout each test, as described previously under “Test Configuration.” Both monotonic and cyclic tests shall be conducted up to deflections that cause ultimate failure of the shear panels or reach the limits of the test equipment, but shall not be less than 10 times the lateral yield displacement of the test panel, δ_y. These deflections are very large (well beyond acceptable drift limits), but they are needed to ensure that brittle failures (sudden loss of lateral or vertical load-carrying capacity) do not occur near the useful deflection range of the panel.

Monotonic test protocol

A single specimen of each shear panel shall be loaded in one direction (monotonic) at a constant stroke rate that is slow enough to allow careful observation of panel performance and failure progression.[25] These observations shall include documentation of panel behavior through a log of observations with respect to displacement and photographs. Load versus deflection (TSF versus DH) shall be plotted to determine the measured lateral yield displacement, δ_y, and this value shall be used in defining the cyclic test protocol.

Cyclic test protocol

A minimum of two specimens of each panel configuration shall be loaded cyclically at a constant stroke rate that is slow enough to allow careful observation of panel performance and failure progression[26]. These observations shall include documentation of panel behavior through a log of observations with respect to displacement and photographs. Load versus deflection (TSF versus DH) shall be plotted to create load/deflection hysteretic envelopes. The cyclic load protocol follows a standard method, so that test results may be compared with cyclic test results of other systems. The protocol defined here is similar to SAC Phase 2 guidelines (SAC 1997) that have been modified to scale to the lateral yield deflection, as described in ATC-24 (ATC 1992). The SAC-recommended loading histories call for loading with a deformation parameter based on interstory drift angle, θ, defined as interstory displacement over interstory height. The commentary to SAC (1997) explains that the interstory drift angle of 0.005 radians corresponds to a conservative estimate of the value that would cause yield deformation. Therefore, the load protocol defined by SAC in terms of drift angle is scaled to the measured lateral yield deflection, δ_y, to define the cyclic test steps shown in Table D-3. This protocol calls for a set number of cycles at each of the deformation amplitudes shown in Table D-3. This protocol is illustrated by the deformation time history shown in Figure D-2, which is based on a lateral yield deflection, δ_y of 0.4 in. and stroke rate of 6 in. per minute.

[25] Monotonic tests reported in Chapter 7 used a stroke rate of 0.5 in. per minute.

[26] Cyclic tests reported in Chapter 7 used a stroke rate of 3 and 6 in. per minute. The faster stroke rate was used for panels tested cyclically beyond 10 in. (20 in. peak to peak).

Table D-3. Cyclic test load protocol.

Load Step #	SAC-2		Modified SAC Amplitude
	Number of Cycles, n	Peak Deformation, θ (radians)	
1	6	0.00375	$0.75\delta_y$
2	6	0.005	$1.0\delta_y$
3	6	0.0075	$1.5\delta_y$
4	4	0.01	$2\delta_y$
5	2	0.015	$3\delta_y$
6	2	0.02	$4\delta_y$
7	2	0.03	$6\delta_y$
8	2	0.04	$8\delta_y$
9	2	0.05	$10\delta_y$
10	2	0.06	$12\delta_y$
11	2	0.07	$14\delta_y$
12	2	0.08	$16\delta_y$
13	2	0.09	$18\delta_y$
14	2	0.10	$20\delta_y$
15	2	0.11	$22\delta_y$
16	2	0.12	$24\delta_y$
17	2	0.13	$26\delta_y$
18	2	0.14	$28\delta_y$
19	2	0.15	$30\delta_y$
20	2	0.16	$32\delta_y$

Shear panel performance documentation

Shear panel performance from both monotonic and cyclic tests shall be documented in terms of load versus deflection plots (TSF versus DH). Cyclic tests plot load versus deflection to define load-versus-deflection hysteretic envelopes. Observations of panel performance and failure progression with respect to lateral displacement shall be documented in a spreadsheet format. Photographs that document these observations shall be included in the test report. Test results for each specimen tested shall be summarized in the format shown in Table D-4. Repeatability of panel performance of a given configuration is critical so that if only two cyclic tests are conducted, the poorest performance of the two shall form the basis for design. Therefore, special consideration shall be given to large variations in panel performance, especially failure type or displacement amplitude of each type of failure. Test procedures and results shall be documented in a test report.

Figure D-2. Modified SAC cyclic test time history, with δ_y = 0.4 in. and 6 in./min stroke rate.

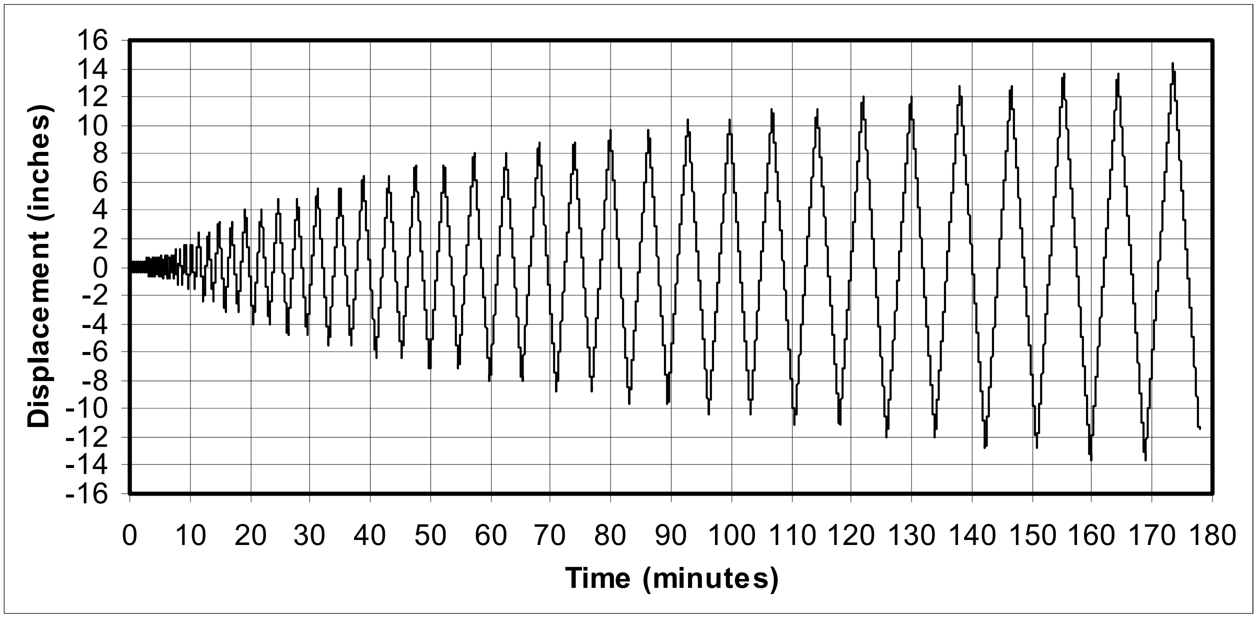


Table D-4. Summary of test panel performance (specified format).

Test Specimen	Load Type (Monotonic or Cyclic)	Load Rate (mm/min or in/min)	Linear Shear Stiffness (kN/mm) or (kips/in.)	Shear Load at δ_y Deflection (kip or kN)	Shear Deflection at Ultimate Shear Load (in. or mm)	Ultimate Shear Load (kip or kN)

Design recommendations

The measured load versus deflection data shall be used to define the design strength and stiffness of the shear panels. Resistance factors for each loading mechanism shall be defined that recognize the variation of the shear panel capacity. In other words, a panel shear capacity resistance factor, ϕ_v, shall reflect the variability of shear capacity of the tested panels. For example, $\phi_v = 0.9$ if the strength variability is small and both mode and displacement of failures are consistent. The following criteria shall be defined from the shear panel cyclic test data:

1. The panel ductility, μ, the ultimate lateral deflection without loss of lateral or vertical load capacity, δ_u, over yield lateral deflection, δ_y, defined as follows:

$$\mu = \frac{\delta_u}{\delta_y} \qquad \text{(Eq D2)}$$

2. The panel overstrength, Ω,[27] the maximum measured ultimate lateral panel capacity, Q_u, over the yield capacity, Q_y, defined as follows:

$$\Omega = \frac{Q_u}{Q_y} \qquad \text{(Eq D3)}$$

3. The panel redundancy factor, ρ_1, of the individual shear panel tested[28]. This redundancy can be seen by comparing shear panel load/deflection data with coupon data, to determine if overstrength, Ω is due to strain hardening of the primary load-carrying element or due to the action of a secondary lateral load-resisting element. An example of this would be a panel with diagonal straps acting as the primary element with the columns effectively working to provide a significant moment frame. In this case the moment frame would provide redundancy for the shear panel. If the diagonal straps fail, this moment frame capacity would provide lateral resistance for the moment from the P-delta effect of the gravity load. This redundancy is critical to preventing building collapse for a structure whose lateral load-resisting system has failed. The panel redundancy factor, ρ_1 is calculated as follows:

$$\rho_1 = \frac{Q_u}{Q_p} = \frac{Q_p + Q_q}{Q_p} \qquad \text{(Eq D4)}$$

where

Q_p = the portion of the shear panel ultimate lateral capacity carried by the primary lateral load-resisting element including the effects of strain hardening. For panels with full panel sheet(s), this contribution will increase with increasing deflection due to a widening of the panel tension field. This value can only be reasonably determined by measuring Q_c (as described below) and calculating Q_p as the difference between Q_u and Q_c.

Q_c = the portion of shear panel ultimate lateral capacity carried by the columns acting as moment frames. For panels with full

[27] This should not be confused with the system overstrength factor, Ω_0, as defined in ASCE 7-10 (ASCE 2010), Section 12.2.1.

[28] This should not be confused with the reliability factor, ρ or ρ_x, which is the extent of structural redundancy in the lateral-force-resisting system for an entire story of a building.

panel sheet(s), this value can only be obtained by testing the same exact panels with the full panel sheets removed. If these tests are not performed for full panel sheet shear panels, Q_c shall be set equal to zero.

4. The width of the cyclic test load/deflection hysteretic envelope. If the hysteretic envelope is significantly pinched (no or very little load resistance away from the peak excursions), much less energy is absorbed by the structural system so that building amplification grows. Pinched hysteretic envelopes occur when the primary lateral load-resisting element is stretched, and there is little redundant capacity from other elements to pick up load, so that little resistance is available away from the peak excursions of the load cycles. Panels with significantly pinched hysteretic envelopes, can experience high acceleration impact loading because the building will be free to sway with little resistance and then suddenly snap the lateral load-resisting element when another peak excursion is reached. This high acceleration snap can cause brittle failures. A shear panel with a great deal of redundancy within the panel, ρ_1 will tend to have a wide hysteretic envelope.

Table D-5 defines the acceptance criteria in terms of μ, Ω and ρ_1, based on data measured in the cyclic panel tests as defined by Equations D2 through D4.

Values for the system response modification coefficient, R; system overstrength factor, Ω_0; and deflection amplification factor, C_d, are defined in Table D-6. These values are used in the seismic design guidance defined in ASCE/SEI 7-10. Exceptions to these criteria shall require AISI approval or Corps of Engineers Headquarters (CEMP-ET) approval for Department of Defense construction.

Table D-5. Acceptance criteria for shear panels based on μ, Ω, and ρ_1.

Criteria	Acceptance Requirement
Panel ductility, μ	≥ 10
Panel overstrength, Ω	≥ 1.3
Panel redundancy factor, ρ_1	≥ 1.0
Hysteretic envelope width	Not required

Table D-6. Values for R, Ω_0, and C_d.

Factor	Value
System response modification coefficient, R	4
System overstrength factor, Ω_0	2
Deflection amplification factor, C_d	3.5