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Exam Preview:

1. According to the reference material, the introduction of water to a slope is not a common cause of slope failures. The addition of water often results in an increase in shear strength of unsaturated soils.
    a. True
    b. False
2. Using Table 10-1, Approximate Shrink/Swell Factors, which of the following materials has a in tiu wet unit weight (pcf) of 131?
    a. Loess
    b. Sandy Gravel
    c. Siltstone
    d. Sandstone
3. If a Geotechnical Designer determines that a slope stability study is necessary, information that will be needed for analysis include: an accurate cross section showing topography, proposed grade, soil unit profiles, unit weight and strength parameters (c', φ'), (c, φ), or Su (depending on soil type and drainage and loading conditions) for each soil unit, and location of the water table and flow characteristics.
    a. True
    b. False
4. According to the reference material, rock slope design heavily relies upon surface mapping and discontinuity logging in boreholes of rock structure to assess discontinuities (fracture/joint) patterns and conditions, as discontinuities strongly control rock slope stability.
    a. True
    b. False

5. According to the reference material, which of the following material soil or rock type matches the following description: generally, include cohesive soils with an unconfined compressive strength greater than 1000 psf but less than 3000 psf and granular cohesionless soils with a high internal angle of friction, such as angular gravel or glacially overridden sand and gravel soils.
   a. Type A soil
   b. Type B soil
   c. Type C soil
   d. Stable Rock
6. According to the reference material, for steel reinforced MSE walls, the design soil friction angle for the backfill shall not be greater than __ even if soil specific shear strength testing is conducted, as research conducted to date indicates that measured reinforcement loads do not continue to decrease as the soil shear strength increases.
   a. 25°
   b. 30°
   c. 45°
   d. 40°
7. According to the reference material, rocks walls are considered to act principally as erosion protection, and they are not considered to provide strength to a slope unless designed as a buttress. What is the maximum height, in feet, that a rock wall can be?
   a. 10
   b. 12
   c. 14
   d. 16
8. Table 15-7, WAC 296-155 Allowable Temporary Cut Slopes, presents the maximum allowable temporary cut slope inclinations based on soil or rock type. Which of the following material has a max slope of 1½H:1V, assuming the maximum height is 20 feet or less?
   a. Stable Rock
   b. Type A soil
   c. Type B soil
   d. Type C soil
9. According to the reference material, drainage ditches along the roadway should be constructed at least __ feet from the toe of the slope, and the ground surface should be gently sloped towards the ditch.
   a. 10
   b. 15
   c. 20
   d. 25
10. According to the reference material, the AL or alignment load used for all permanent ground anchors must only hold 1 minute according to the strength limit state control.
    a. True
    b. False

Washington State
Department of Transportation

Geotechnical Design Manual

M 46-03.14

June 2021

PART 3 OF 4

Environmental and Regional Operations
Construction Division
Geotechnical Office

10.1 Overview and Data Acquisition

10.1.1 Overview

During the project definition phase, the project designer provides a description of the proposed cuts to the Region Materials Engineer (RME) as outlined in the *Design Manual* M 22-01 Chapter 510. The designer may prepare preliminary cross sections using the criteria presented in *Design Manual* M 22-01 Section 640.07. For side hill conditions the cross sections should extend up to the top of the hill or a controlling feature such as a rock outcrop or level bench. The RME with assistance from the HQ Geotechnical Division as needed, reviews existing information, performs a site reconnaissance and provides conceptual recommendations.

During the project design phase the subsurface investigation is completed and the cut slope design recommendations are prepared. Included in the recommendations are the slope inclinations required for stability, mitigation requirements if needed and the usability of excavated cut material. Typically for cut slope design, adequate geotechnical information is provided during the project design phase to complete the PS&E Development. Additional geotechnical work might be needed when right of way cannot be obtained or design requirements change.

10.1.2 Site Reconnaissance

General procedures for site reconnaissance are presented in Chapter 2. Special considerations for cut slopes should be made during the office and site review. The office review of aerial photos from different dates may reveal if there has been any change in slope angle or vegetation over time. Landforms identified on the photos should be field checked to determine if they can be related to geologic processes and soil type.

The existing natural and cut slopes in the project vicinity should be inspected for performance. Measure the inclination and height of existing cut slopes, and look for erosion or slope stability problems. Ask the regional maintenance engineer about any stability/erosion problems with the existing cut slopes. In general, if stable slopes will be cut back into an existing slope 10 feet or less and at the same or flatter angle of inclination, the slope height does not increase significantly because of the cut, there is no evidence of instability, there is no evidence the material type is likely to be different at the excavation face, and there is no potential for seepage to be encountered in the cut, then typically no further exploration will be required.

Observation of existing slopes should include vegetation, in particular the types of vegetation that may indicate wet soil. Indirect relationships, such as subsurface drainage characteristics may be indicated by vegetative pattern. Assess whether tree roots may be providing anchoring of the soil and if there are any existing trees near the top of the proposed cut that may become a hazard after the cut is completed.

Changes in ground surface slope angle may reflect differences in physical characteristics of soil and rock materials or the presence of water.

For cuts that are projected to be less than 10 feet in height, determine if further exploration is warranted based on soil type and extent.

10.1.3 Field Exploration

10.1.3.1 Test Borings

A minimum of one boring should be performed for each proposed soil cut slope greater than about 10 feet in height. For longer cuts, horizontal spacing for borings parallel to the cut should generally be between 200 to 400 feet, based on site geology. Wider spacing may be considered if, based on existing data and site geology, conditions are likely to be uniform and of low impact to construction and long-term cut slope performance. Each landform should be explored, and the borings should be spaced so that the extent of each soil type present is reasonably determined. At critical locations where slope stability analysis is necessary, additional borings perpendicular to the cut should be provided in order to model existing geologic conditions for use in slope stability analysis. The exploration program should also be developed with consideration to the potential for use of the removed material as a source for fill material elsewhere on the project. If the construction contract is set up with the assumption that the cut material can be used as a materials source for fill or other uses on the project, it is important to have adequate subsurface information to assess how much of the cut material is useable for that purpose. A key to the establishment of exploration frequency for embankments is the potential for the subsurface conditions to impact the construction of the cut, the construction contract in general, and the long-term performance of the finished project. The exploration program should be developed and conducted in a manner that these potential problems, in terms of cost, time, and performance, are reduced to an acceptable level. The boring frequency described above may need to be adjusted by the geotechnical designer to address the risk of such problems for the specific project.

Borings should extend a minimum of 15 feet below the anticipated depth of the cut at the ditch line to allow for possible downward grade revision and to provide adequate information for slope stability analysis. Boring depths should be increased at locations where base stability is a concern due to groundwater and/or soft or weak soil zones. Borings should extend through any weak zones into competent material.

Hand augers, test pits, trenches or other similar means of exploration may be used for investigating subsurface conditions for sliver cuts (additional cut in an existing natural or cut slope) or shallow cuts, if the soil conditions are known to be fairly uniform.

10.1.3.2 Sampling

For soil cuts, it is important to obtain soil samples in order to perform laboratory index tests such as grain size analysis, natural moisture content and Atterberg limits. This is generally the best way to define site stratigraphy. In situ testing can be used to augment the exploration program. However, information obtained from site specific samples is necessary to verify and place in proper context soil classification, strength and compressibility parameters obtained from in situ tests. Sampling should be performed for the purpose of cut stability assessment and assessment of the cut material as a materials source, if the cut material is needed as a materials source. Special considerations for loess slopes are discussed later in this chapter.

For granular soils, SPT samples at 5 feet intervals and at changes in strata are generally sufficient. A combination of SPTs and undisturbed thin-wall push tube (i.e. WSDOT undisturbed or Shelby tube) should be used in cohesive soil. The vane shear test (VST) may also be performed in very soft to soft cohesive soil. In general, the VST should be used in conjunction with laboratory triaxial testing unless there is previous experience with the VST at the site. The pressuremeter test (PMT) and dilatometer test (DMT) are expensive and generally have limited applicability for cut slope design, but are useful for determining shear strength and overconsolidation ratio in stiff to hard cohesive soil.

Because it is generally desirable to obtain samples for laboratory testing, the static cone penetration test (CPT) is not often used for routine exploration of cut slopes. However, the CPT provides continuous data on the stratigraphic profile and can be used to evaluate in situ strength parameters in very soft to medium stiff cohesive soil and very loose to medium dense sands.

10.1.3.3 Groundwater Measurement

Knowledge of groundwater elevations is critical for the design of cut slopes. The presence of groundwater within or just below a proposed cut will affect the slope angle required to achieve and maintain stability. For example, the presence of groundwater near the base of a proposed cut slope in loess will preclude making a near vertical slope. Substantially more right-of-way may be required to construct a flatter slope. Measurement of groundwater and estimates of its fluctuations are also important for the design of appropriate drainage facilities. Groundwater that daylights within a proposed cut slope may require installation of horizontal drains (generally for coarser grained cohesionless soils) or other types of drainage facilities. Groundwater near the toe of slopes may require installation of underdrains. Groundwater measurements are also important if slope stability analysis is required.

In granular soil with medium to high permeability, reliable groundwater levels can sometimes be obtained during the drilling program. At a minimum, groundwater levels should be obtained at completion of drilling after the water level has stabilized and 12 hours after drilling is completed for holes located in medium to high permeability soils. In low permeability soils false water levels can be recorded, as it often takes days for water levels to reach equilibrium; the water level is further obscured when drilling fluid is used. In this case piezometers should be installed to obtain water levels after equilibrium has been reached. Piezometers should be installed for any major cuts, or as determined by the geotechnical designer, to obtain accurate water level information.

If slope stability analysis is required or if water levels might be present near the face of a cut slope, piezometers should be installed in order to monitor seasonal fluctuations in water levels. Monitoring of piezometers should extend through at least one wet season (typically November through April). Continuous monitoring can be achieved by using electrical piezometers such as vibrating wire type in conjunction with digital data loggers.

Values of permeability and infiltration rates are generally determined based on correlations with grain size and/or knowledge of the site soil based on previous experience. However, borehole permeability tests, such as slug or pump tests, may be performed in order to design drainage facilities, especially if horizontal drains may be used.

10.1.4 Laboratory Testing

Standard classification tests should be performed on representative samples for all soil cut slopes. These tests include gradation analysis, moisture content, and Atterberg limits. These tests will provide information to aid in determining appropriate slope inclinations, drainage design, and usability of the cut material as a materials source for earthwork on the project. Additional tests will often be required to determine the suitability of reusing soil excavated from a cut for other purposes throughout the project. Examples include organic content to determine if a soil should be classified as unsuitable and compaction testing to aid in determining the optimum moisture content and shrink/swell factors for earthwork calculations. pH and corrosivity tests should also be performed on samples at locations for proposed drainage structures.

If it is determined by the geotechnical designer that slope stability analysis should be performed, laboratory strength testing on undisturbed samples may be required. Slope stability analysis requires accurate information of soil stratigraphy and strength parameters, including cohesion ($\underline{c}$'), friction angle (ϕ'), undrained shear strength (S_u), and unit weight for each layer. In-place density measurements can be determined from WSDOT undisturbed, Dames and Moore, or Shelby tube samples.

Cohesive soil shear strength parameters should be obtained from undisturbed soil samples using consolidated undrained triaxial tests with pore pressure measurement if portions of the proposed slope are saturated or might become saturated in the future. Effective strength parameters from these tests should be used to analyze cohesive soil cut slopes and evaluate long term effects of soil rebound upon unloading. Unconsolidated undrained (UU) triaxial tests or direct shear tests can be used to obtain undrained shear strength parameters for short term stability analysis, or when it is determined by the geotechnical designer that total stress/strength parameters are sufficient. The choice of which test to perform should be determined by the expected stress condition in the soil in relation to the anticipated failure surface. It should be understood, however, that strength parameters obtained from unsaturated tests are dependent on the moisture content at which the tests are performed. If the moisture content of the soil in question increases in the future, even to levels still below saturation, the shear strength might be significantly reduced, especially for cohesive soils. <u>Ring</u> shear tests can be performed to determine residual shear strength parameters for soils located in existing landslide areas. <u>Repeated direct shear tests have been used in the past to obtain residual strength parameters, but research has shown that this approach tends to over-estimate the residual strength, unless a slickensided surface in the specimen can be oriented such that the direct shear test fails the specimen on that pre-existing surface (Sabatini, et al., 2002).</u> Residual strength parameters should also be obtained for cuts in heavily overconsolidated clays, such as the Seattle clays (e.g., Lawton formation), as the removal of soil can release locked in stresses and allow the clay to deform, causing its strength to drop to a residual value.

It should be noted that for unsaturated soils, particularly cohesive soils, the natural moisture content of the soil at the time of testing must be determined since this will affect the results. Consideration should be given during stability analysis to adjusting strength parameters to account for future changes in moisture content, particularly if field testing was performed during the dry summer months and it is possible that the moisture content of the soil will likely increase at some point in the future. In this case using the values obtained from the field directly may lead to unconservative estimates of shear strength.

10.2 Overall Design Considerations

10.2.1 Overview

Small cut slopes are generally designed based on past experience with similar soils and on engineering judgment. Cut slopes greater than 10 feet in height usually require a more detailed geotechnical analysis. Relatively flat (2H:1V or flatter) cuts in granular soil when groundwater is not present above the ditch line, will probably not require rigorous analysis. Any cut slope where failure would result in large rehabilitation costs or threaten public safety should obviously be designed using more rigorous techniques. Situations that will warrant more in-depth analysis include large cuts, cuts with irregular geometry, cuts with varying stratigraphy (especially if weak zones are present), cuts where high groundwater or seepage forces are likely, cuts involving soils with questionable strength, or cuts in old landslides or in formations known to be susceptible to landsliding.

A major cause of cut slope failures is related to the release of stress within the soil upon excavation. This includes undermining the toe of the slope and oversteepening the slope angle, or as mentioned previously, cutting into heavily overconsolidated clays. Careful consideration should be given to preventing these situations for cut slopes by keeping the base of the slope as loaded as possible, by choosing an appropriate slope angle (i.e. not oversteepening), and by keeping drainage ditches near the toe a reasonable distance away. For heavily overconsolidated clays, retaining walls rather than an open cut may be needed that will prevent the deformation necessary to allow the soil strength to go to a residual value.

Consideration should also be given to establishing vegetation on the slope to prevent long-term erosion. It may be difficult to establish vegetation on slopes with inclinations greater than 2H:1V without the use of erosion mats or other stabilization method.

10.2.2 Design Parameters

The major parameters in relation to design of cut slopes are the slope angle and height of the cut. For dry cohesionless soil, stability of a cut slope is independent of height and therefore slope angle becomes the only parameter of concern. For purely cohesive (ϕ= 0) soils, the height of the cut becomes the critical design parameter. For c'-ϕ' and saturated soils, slope stability is dependent on both slope angle and height of cut. Also critical to the proper design of cut slopes is the incorporation of adequate drainage facilities to ensure that future stability or erosional problems do not occur.

10.3 Soil Cut Design

10.3.1 Design Approach and Methodology

Safe design of cut slopes is based either on past experience or on more in-depth analysis. Both approaches require accurate information regarding geologic conditions obtained from standard field and laboratory classification procedures. Cut slope heights and inclinations provided in the *Design Manual* M 22-01 can be used unless indicated otherwise by the Geotechnical Designer. If the Geotechnical Designer determines that a slope stability study is necessary, information that will be needed for analysis include: an accurate cross section showing topography, proposed grade, soil unit profiles, unit weight and strength parameters (c', ϕ'), (c, ϕ), or S_u (depending on soil type and drainage and loading conditions) for each soil unit, and location of the water table and flow characteristics.

Generally, the design factor of safety for static slope stability is 1.25. For pseudo-static seismic analysis the factor of safety can be decreased to 1.1. Cut slopes are generally not designed for seismic conditions unless slope failure could impact adjacent structures. These factors of safety should be considered as minimum values. The geotechnical designer should decide on a case by case basis whether or not higher factors of safety should be used based the consequences of failure, past experience with similar soils, and uncertainties in analysis related to site and laboratory investigation.

Initial slope stability analysis can be performed using simple stability charts. See Abramson et al. (1996) for example charts. These charts can be used to determine if a proposed cut slope might be subject to slope failure. If slope instability appears possible, or if complex conditions exist beyond the scope of the charts, more rigorous computer methods such XSTABL, PCSTABL, SLOPE/W, etc. can be employed (see Chapter 7). As stated previously, effective use of these programs requires accurate determination of site geometry including surface profiles, soil unit boundaries, and location of the water table, as well as unit weight and strength parameters for each soil type.

Because of the geology of Washington, many soil cuts will likely be in one of five typical types of deposits. These soils can be grouped based on geologic history and engineering properties into residual soil, alluvial sand and gravel, glacially overconsolidated soil, colluvial deposits, and loess deposits. A design procedure has been developed for loess slopes and is presented later in this chapter. A brief discussion of the other three soil types follows:

Residual Soil – The most typical residual soil is encountered in the Coast Range in the southwest part of the state. Other residual soil units weathered from rock formations such as the Renton, Cowlitz, Ellensburg and Ringold are also encountered in other parts of the state. However, the soil in the coast range is the most extensive residual soil found in the state and is the focus of this discussion. These soils have formed from weathering of siltstone, sandstone, claystone and tuff, and typically consist of soft to stiff silt, elastic silt and lean clay with varying amounts of rock fragments, sand and fat clay. Because of the cohesive nature of the soil and the angular rock fragments, the soils often form fairly steep natural slopes. Root strength from dense vegetation also contributes to the steep slopes. Logging a slope can often cause it to become unstable within a few years. These slopes are likely to become at least partially saturated during the winter and spring months. Groundwater also tends to move unevenly through the soil mass following zones of higher permeability such as sand layers and relict bedding and joint planes. For this reason, determination of representative groundwater elevations with the use of open standpipe piezometers may be difficult.

These slopes should generally be designed using total stress parameters to assess short-term strength during initial loading, and also using effective stress parameters to assess long-term stability; however, laboratory testing in these soils can be problematic because of variability and the presence of rock fragments. Shallow surface failures and weak zones are common. Typical design slopes should generally be 2H:1V or flatter. Vegetation should be established on cut slopes as soon as possible.

Alluvial Sand and Gravel Deposits – Normally consolidated sand and gravel deposits in Washington are the result of several different geologic processes. Post glacial alluvial deposits are located along existing rivers and streams and generally consist of loose to medium dense combinations of sand, gravel, silt and cobbles. In the Puget Sound region, extensive recessional outwash deposits were formed during the retreat of glacial ice. These deposits generally consist of medium to very dense, poorly graded sand and gravel with cobbles, boulders and varying amounts of silt.

In eastern Washington, extensive sand and gravel deposits were deposited during catastrophic outburst floods from glacially dammed lakes in Montana. These deposits often consist of loose to dense, poorly graded sand and gravel with cobbles and boulders and varying amounts of silt. Slopes in sand and gravel deposits are generally stable at inclinations of from 1.5H:1V to 2H:1V, with the steeper inclinations used in the more granular soil units with higher relative densities. Perched water can be a problem, especially in western Washington, when water collects along zones of silty soil during wet months. These perched zones can cause shallow slope failures. If significant amounts of silt are not present in the soil, vegetation is often difficult to establish.

Glacially Overconsolidated Deposits – Glacially consolidated soils are found mainly in the Puget Sound Lowland and the glacial valleys of the Cascades. For engineering purposes, these deposits can generally be divided into cohesionless and cohesive soil. The cohesionless soil deposits are poorly sorted and consist of very dense sand and gravel with silt, cobbles, and boulders. The soil units exhibit some apparent cohesion because of the overconsolidation and fines content. If little or no groundwater is present, slopes will stand at near vertical inclinations for fairly long periods of time. However, perched groundwater on low permeability layers is very often present in these slopes and can contribute to instability. Typical inclinations in these soils range from 1.75H:1V to 1H:1V; although, the steeper slope inclinations should be limited to slopes with heights of about 20 feet or less. These slopes also work well with rockeries at slopes of 1H:6V to 1H:4V.

Overconsolidated cohesive soils such as described in Section 5.13.3 consist of very stiff to very hard silt and clay of varying, and may contain fissures and slickensides. These soils may stand at near vertical inclinations for very limited periods of time.

The relaxation of the horizontal stresses cause creep and may lead to fairly rapid failure. Slopes in these soils should be designed based on their residual friction angle and often need to be laid back at inclinations of 4H:1V to 6H:1V. See Section 5.13.3 for specific requirements regarding the design of slopes in this type of deposit.

10.3.2 Seepage Analysis and Impact on Design

The introduction of water to a slope is a common cause of slope failures. The addition of water often results in a reduction in shear strength of unsaturated soils. It raises the water table and adds to seepage forces, raising pore pressures and causing a corresponding reduction in effective stress and shear strength in saturated soil. Finally, it adds weight to the soil mass, increasing driving forces for slope failures. In addition, it can cause shallow failures and surface sloughing and raveling. These problems are most common in clay or silt slopes. It is important to identify and accurately model seepage within proposed cut slopes so that adequate slope and drainage designs are employed.

For slope stability analysis requiring effective stress/strength parameters, pore pressures have to be known or estimated. This can be done using several methods. The phreatic (water table) surface can be determined by installing open standpipes or observation wells. This is the most common approach. Piezometric data from piezometers can be used to estimate the phreatic surface, or peizometric surface if confined flow conditions exist. A manually prepared flow net or a numerical method such as finite element analysis can be used provided sufficient boundary information is available. The pore pressure ratio (r_u) can also be used. However, this method is generally limited to use with stability charts or for determining the factor of safety for a single failure surface.

10.3.3 Drainage Considerations and Design

The importance of adequate drainage cannot be overstated when designing cut slopes. Surface drainage can be accomplished through the use of drainage ditches and berms located above the top of the cut, around the sides of the cut, and at the base of the cut. The following section on cut slopes in loess contains a more in-depth discussion on surface drainage.

Subsurface drainage can be employed to reduce driving forces and increase soil shear strength by lowering the water table, thereby increasing the factor of safety against a slope failure. Subsurface conditions along cut slopes are often heterogeneous. Thus, it is important to accurately determine the geologic and hydrologic conditions at a site in order to place drainage systems where they will be the most effective. Subsurface drainage techniques available include cut-off trenches, horizontal drains and relief wells.

Cut-off trenches are constructed by digging a lateral ditch near the top of the cut slope to intercept ground water and convey it around the slope. They are effective for shallow groundwater depths. If the groundwater table needs to be lowered to a greater depth, horizontal drains can be installed, if the soils are cohesionless and granular in nature. Horizontal drains are generally not very effective in finer grained soils. Horizontal drains consist of small diameter holes drilled at slight angles into a slope face and backfilled with perforated pipe wrapped in drainage geotextile. Installation might be difficult in soils containing boulders, cobbles or cavities. Horizontal drains require periodic maintenance as they tend to become clogged over time. Relief wells can be used in situations where the water table is at a great depth. They consist of vertical holes cased with perforated pipe connected to a disposal system such as submersible pumps or discharge channels similar to horizontal drains. They are generally not common in the construction of cut slopes.

Whatever subsurface drainage system is used, monitoring should be implemented to determine its effectiveness. Typically, piezometers or observation wells are installed during exploration. These should be left in place and periodic site readings should be taken to determine groundwater levels or pore pressures depending on the type of installation. High readings would indicate potential problems that should be mitigated before a failure occurs.

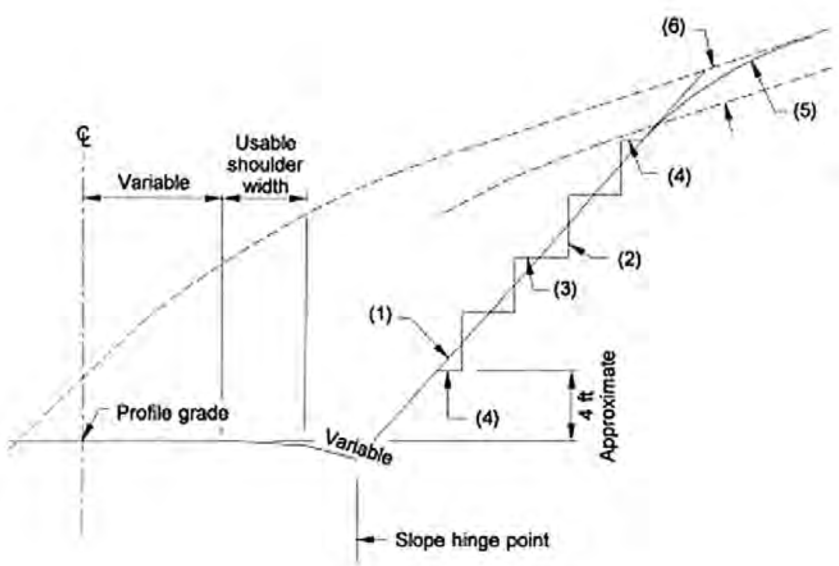
Surface drainage, such as brow ditches at the top of the slope, and controlling seepage areas as the cut progresses and conveying that seepage to the ditch at the toe of the cut, should be applied to all cut slopes. Subsurface drainage is more expensive and should be used when stability analysis indicates pore pressures need to be lowered in order to provide a safe slope. The inclusion of subsurface drainage for stability improvement should be considered in conjunction with other techniques outlined below to develop the most cost effective design meeting the required factor of safety.

10.3.4 Stability Improvement Techniques

There are a number of options that can be used in order to increase the stability of a cut slope. Techniques include:

- Flattening slopes
- Benching slopes
- Lowering the water table (discussed previously)
- Structural systems such as retaining walls or reinforced slopes.

Changing the geometry of a cut slope is often the first technique considered when looking at improving stability. For flattening a slope, enough right-of-way must be available. As mentioned previously, stability in purely dry cohesionless soils depends on the slope angle, while the height of the cut is often the most critical parameter for cohesive soils. Thus, flattening slopes usually proves more effective for granular soils with a large frictional component. Benching will often prove more effective for cohesive soils. Benching also reduces the amount of exposed face along a slope, thereby reducing erosion. Figure 10.1 shows the typical configuration of a benched slope. Structural systems are generally more expensive than the other techniques, but might be the only option when space is limited.



Notes:
(1) Staked slope line - Maximum slope 1H:1V.
(2) Step rise - heaight variable 1 foot to 2 feet.
(3) Step tread - width = staked slope ratio × step rise.
(4) Step termini - width = 1/2 step tread width.
(5) Slope rouding.
(6) Overburden area - variable slope ratio.

Typical Roadway Section With Stepped Slopes
(From Design Manual Figure 1230-8)
Figure 10-1

Shallow failures and sloughing can be mitigated by placing 2 to 3-foot thick rock drainage blanket over the slope in seepage areas. Moderate to high survivability permanent erosion control geotextile should be placed between native soil and drain rock to keep fines from washing out and/or clogging the drain rock.

In addition, soil bioengineering can be used to stabilize cut slopes against shallow failures (generally less than 3 feet deep), surface sloughing and erosion along cut faces. Refer to the *Design Manual* M 22-01 Chapter 940 for uses and design considerations of soil bioengineering.

10.3.5 Erosion and Piping Considerations

Surface erosion and subsurface piping are most common in clean sand, nonplastic silt and dispersive clays. Loess is particularly susceptible. However, all cut slopes should be designed with adequate drainage and temporary and permanent erosion control facilities to limit erosion and piping as much as possible. See Sections *10.3.3* and *10.5* for more information on drainage structures.

The amount of erosion that occurs along a slope is a factor of soil type, rainfall intensity, slope angle, length of slope, and vegetative cover. The first two factors cannot be controlled by the designer, but the last three factors can. Longer slopes can be terraced at approximate 15- to 30-foot intervals with drainage ditches installed to collect water. Best Management Practices (BMPs) for temporary and permanent erosion and stormwater control as outlined in the WSDOT *Highway Runoff Manual* and WSDOT *Roadside Manual* should always be used. Construction practices should be specified that limit the extent and duration of exposed soil. For cut slopes, consideration should be given to limiting earthwork during the wet season and requiring that slopes be covered as they are exposed, particularly for highly erodable soils mentioned above.

10.4 Use of Excavated Materials

The suitability of soil excavated from a roadway cut section for reuse should be determined by a combination of site reconnaissance, boring information and laboratory testing. Soil samples obtained from SPT testing are generally too small to be used for classifying soils as gravel borrow, select borrow, etc. Bulk soil samples obtained from test pits are more appropriate to determine the appropriate engineering characteristics, including compaction characteristics, of all soil units.

Based on the exploration and laboratory testing program, the geotechnical designer should determine the extent of each soil unit, the preferred uses for each unit (i.e. common fill, structural fill, drain rock, riprap, etc.), and any measures necessary for improvement of soil units to meet a particular specification. Soil excavated from within the roadway prism intended for use as embankment fill should generally meet, as a minimum, *Standard Specification* 9-03.14(3) for common borrow. However, both common borrow and select borrow are not usable as an all weather material. If all weather use is desired, the material should meet the specifications for gravel borrow per the WSDOT Standard Specifications. Any soil units considered unsuitable for reuse such as highly plastic soil, peat, and muck should be identified.

Consideration should be given to the location and time of year that construction will likely take place. In western Washington, in place soil that is more than a few percentage points over optimum moisture content is often impractical to aerate and dry back and must be wasted, stockpiled for later use or conditioned with admixtures. Even glacially overconsolidated soil with a high fines content that is near the optimum moisture content may become too wet for proper compaction during excavation, haul and placement. Laboratory testing consisting of the standard and modified Proctor (ASSHTO T 99 and T 180, respectively) tests should be performed on bulk samples, if the fines content indicates the soil may be moisture sensitive (generally more than about 10 percent). The *Standard Specification Section* 2-03.3(14)D requires that maximum density for soil with more than 30 percent by weight retained on the U.S. No. 4 sieve be determined by WSDOT Test Method 606. Test Method 606 does not provide reliable information on the optimum moisture content for placement. Therefore, the modified Proctor test should be performed to determine the optimum moisture.

Techniques such as adding portland cement to stabilize wet soil have been used on WSDOT projects in the past. The addition of cement can lower the moisture content of soil a few percent and provide some strength. However, concerns regarding the pH of runoff water from the project site may limit the use of this technique on some sites. The FHWA Publication "Soil and Base Stabilization and Associated Drainage Considerations, Volumes 1 and 2" (SA-93-004 & SA-93-005) provide additional information on soil amendments.

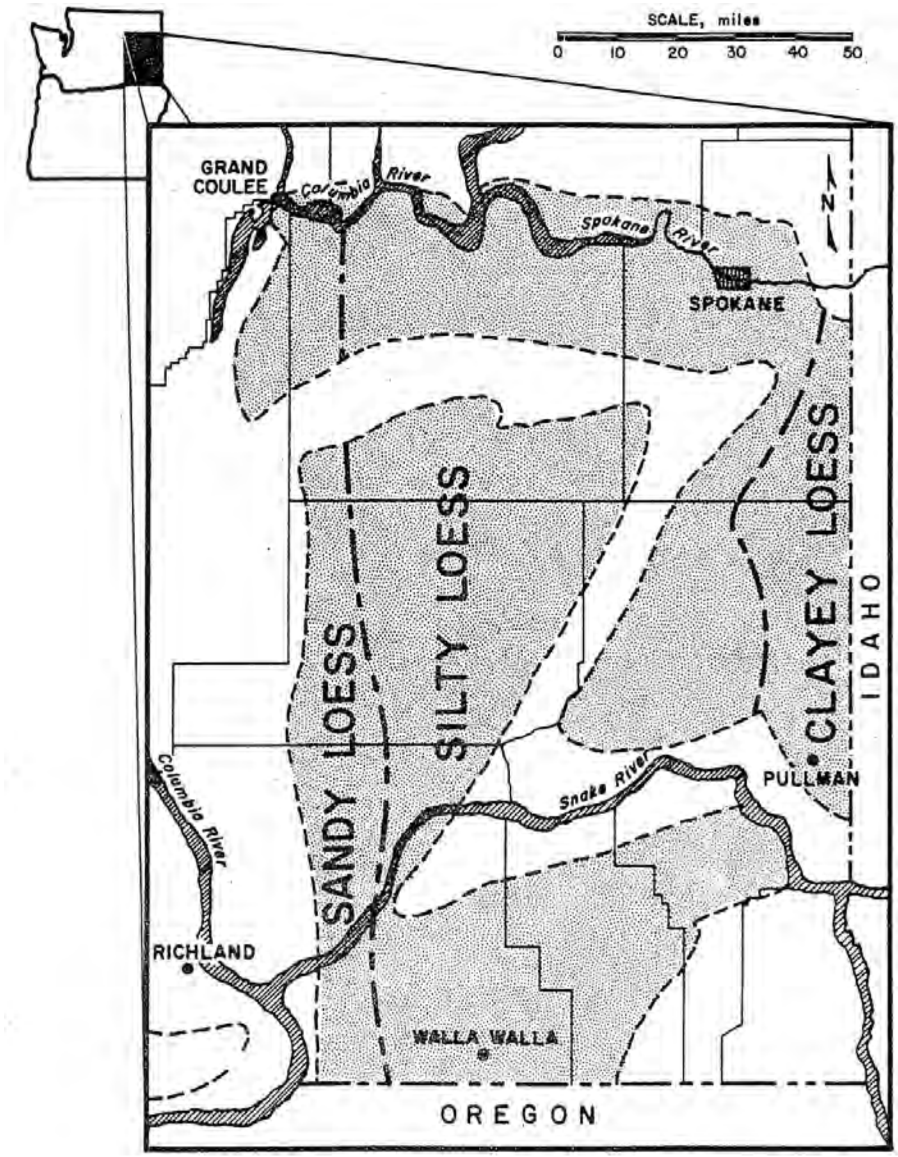
The RME or geotechnical designer should provide guidance in determining shrink/swell factors for earthwork computations. Soil excavated from cuts and then compacted for embankment construction typically has a shrinkage factor. Values vary based on soil type, in-place density, method of fill construction and compactive effort. Soil wasted typically has a swell factor because material is often end-dumped at the waste site. The shrink/swell factor for soil that will be reused can be estimated by determining the ratio of in situ density versus compacted density determined from Proctor tests. Corrections may need to be applied for oversize particles screened out of xcavated material. Local experience with similar soil also can be used to determine shrink/swell factors. Typical shrink/swell factors for various soils and rock are presented in Table 10-1.

Material	In situ wet unit weight (pcf)	Percent Swell	Loose Condition wet unit weight (pcf)	Percent Shrink (-) or Swell (+)	Compacted wet unit weight (pcf)
Sand	114	5	109	-11	129
Sandy Gravel	131	5	124	-7	141
Silt	107	35	79	-17	129
Loess	91	35	67	-25	120
Rock/Earth Mixtures					
75% R/25 % E	153	25	122	+12	136
50% R/50% E	139	29	108	-5	146
25% R/75% E	125	26	99	-8	136
Granite	168	72	98	+28	131
Limestone	162	63	100	+31	124
Sandstone	151	61	94	+29	117
Shale-Siliceous	165	40	118	+25	132
Siltstone	139	45	96	+9	127

Approximate Shrink/Swell Factors
(From Alaska DOT Geotechnical Procedures Manual, 1983)
Table 10-1

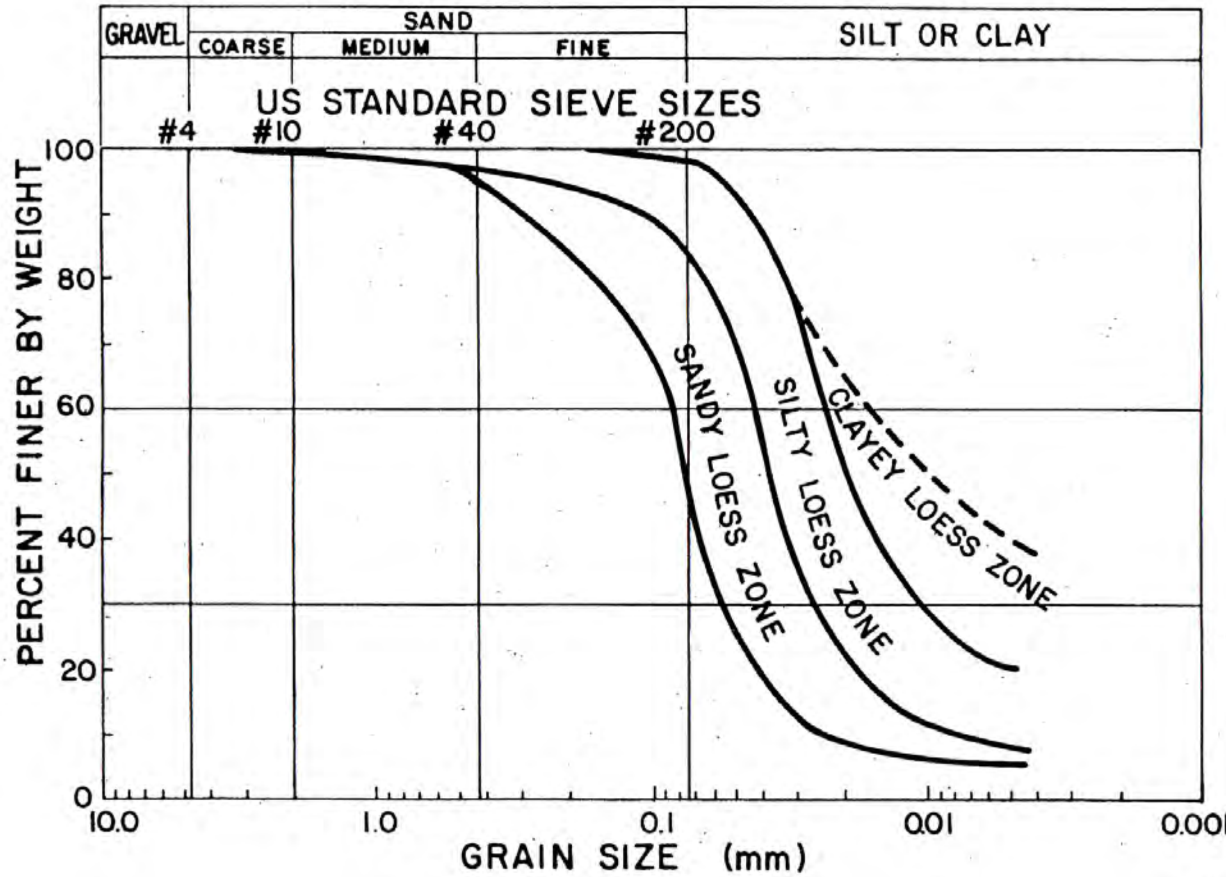
10.5 Special Considerations for Loess

Loess is an aeolian (wind deposited) soil consisting primarily of silt with fine sand and clay, generally found in the southeastern part of the state. See Figure 10-2 for general extents of loess deposits found within Washington state. Loess contains a large amount of void space, and particles are held together by the clay component. It can stand at near vertical slopes indefinitely provided its moisture content remains low. However, upon wetting it loses strength and because of its open structure can experience large rapid deformations that can result in slope failures. Slope failures in loess soil can occur as either shallow slides or flows or rotational slides. Loess is also highly prone to erosion and piping.



Approximate Gradation of Boundaries for Washington Loess
(After Higgins and Fragaszy, WA-RD 145.2)
Figure 10-2

Loess Can be Broken Down into Three Main Types – Clayey loess, silty loess, and sandy loess, based on grain size analysis (see Figure 10-3). Past research indicates that cuts in silty loess deposits with low moisture contents can stand at near vertical slopes (0.25H:1V), while cuts in clayey loess deposits perform best at maximum slopes of 2.5H:1V. Soils characterized as sandy loess can be designed using conventional methods. WSDOT manual "Design Guide for Cut Slopes in Loess of Southeastern Washington" (WA-RD 145.2) provides an in-depth discussion on design of cut slopes in loess.



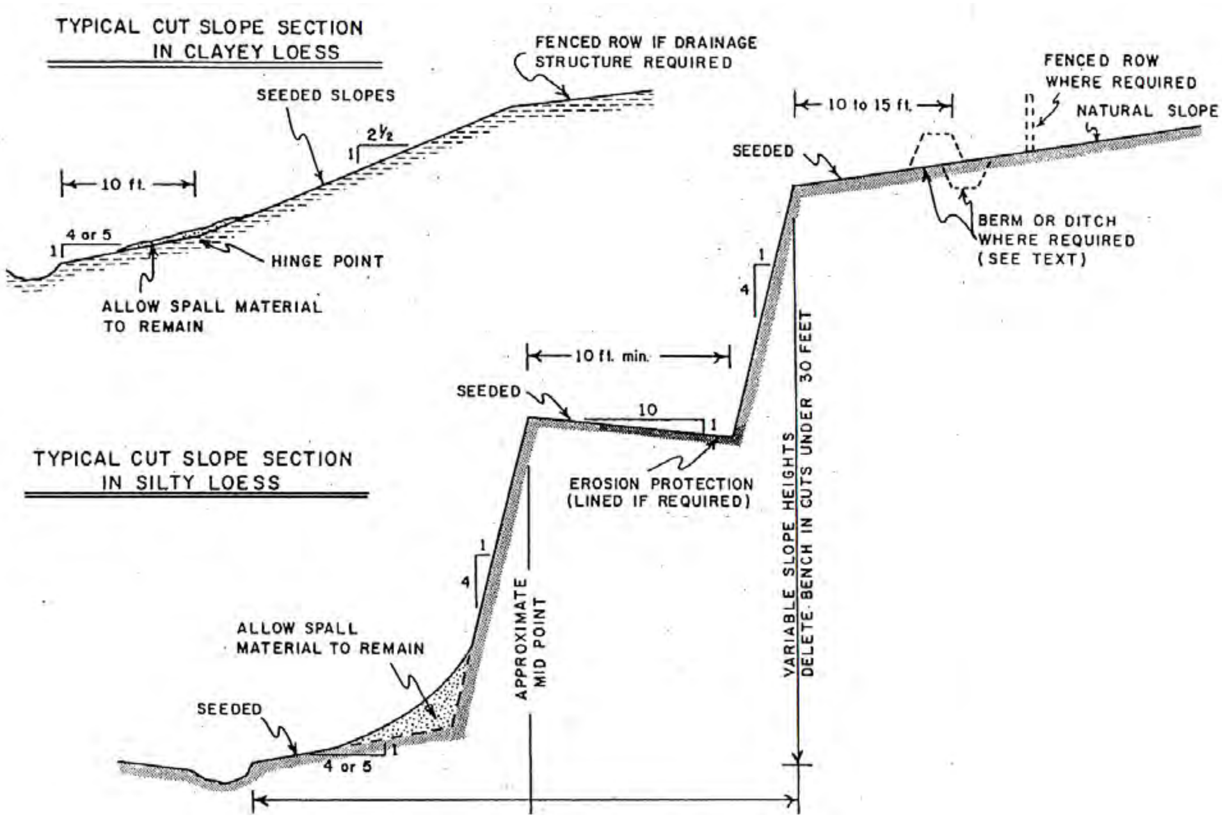
Definition of Sandy, Silty, and Clayey Loess for Southeastern Washington (After Higgins and Fragaszy, 1988)
Figure 10-3

The two most important factors affecting performance of cut slopes in loess are gradation and moisture content. Moisture content for near vertical slopes is crucial. It should not be over 17 percent. There should be no seepage along the cut face, especially near the base. If there is a possibility of groundwater in the cut, near vertical slopes should not be used. Maintenance of moisture contents below critical values requires adequate drainage facilities to prevent moisture migration into the cut via groundwater or infiltration from the surface.

The design of cut slopes in loess should include the following procedures that have been adapted from WA-RD 145.2 (Higgins and Fragaszy, 1988):

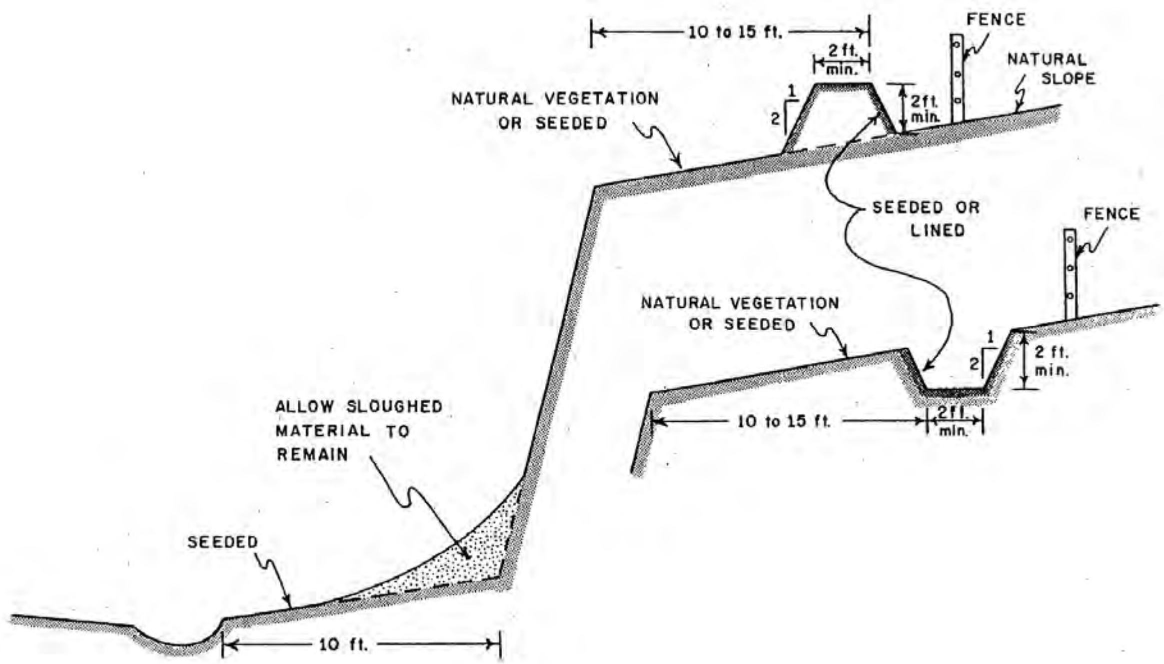
1. Perform office studies to determine possible extents of loess deposits along the proposed road alignment.
2. Perform field reconnaissance including observation of conditions of existing cut slopes in the project area.
3. Perform field exploration at appropriate locations. For loess slope design, continuous sampling in the top 6 feet and at 5 foot intervals thereafter should be used.

4. Perform laboratory grain-size analysis on representative samples throughout the depth of the proposed cut and compare the results with Figure 10-3. If the soil falls within the zone of sandy loess, or if sandy layers or other soils are encountered that do not classify as silty or clayey loess, design using conventional soil mechanics methods. If the soil falls within the zone of clayey loess, design using a maximum slope inclination of 2.5H:1V. If the soil falls within the zone of silty loess, the slope may be designed using a 0.25H:1V inclination provided that moisture contents will be within allowable levels as described in subsequent steps. See Figure 10-4 for typical sections in silty and clayey loess. If deep cuts (greater than about 50 feet) are to be used, or if moisture contents during the design life of the slope greater than 17 percent are expected, it is recommended that laboratory shear strength testing be run in order to perform slope stability analysis. If moisture contents below 17 percent are expected, total stress analysis can be used. If moisture contents above 17 percent are expected, effective stress analysis should be used. Care should be taken when using laboratory shear strength data because of the difficulty obtaining undisturbed samples in loess.
5. Determine if groundwater or seasonal perched water might be present. If so, the cut slope should be designed for a maximum slope of 2.5H:1V and appropriate drainage design applied. Slopes flatter than 2.5H:1V might be necessary because of seepage forces. In this case a drainage blanket may be required. See step 4 if slope stability analysis is required.
6. Perform moisture content analysis on representative samples. Moisture contents within the proposed slope above 17 percent indicate the soil structure is potentially unstable and prone to collapse. If moisture contents are below 17 percent and the soil classifies as silty loess, design for near vertical slopes. Otherwise, design for maximum slopes of 2.5H:1V. See step 4 if slope stability analysis is required.
7. Near vertical slopes should be benched on approximately 20 feet vertical intervals when the total height of the cut exceeds 30 feet. Benches should be 10 to 15 feet wide and gently sloped (10H:1V) towards the back of the cut to prevent water from flowing over the cut face. Benches should maintain a gradient for drainage not exceeding 3 to 5 percent. See number 4 if slope stability analysis is required.
8. Adequate drainage control is extremely important in loess soil due to its strength dependence on moisture content and high potential for erosion. The following section outlines general drainage design considerations for loess slopes. These designs can also be employed for cut slope design in other soils. However, as stated previously, loess soils are generally more susceptible to erosion and wetting induced slope failures, so the design of drainage structures for loess slopes might be overconservative when applied to other soils.



Typical Sections for Cut Slopes in Silty and Clayey Loess
(After Higgins and Fragaszy, 1988)
Figure 10-4

Drainage at Head of Slopes – For silty loess, a drainage ditch or berm should be constructed 10 to 15 feet behind the top of the slope prior to excavation. Provided the gradient is less than about 5 percent, a flat bottomed, seeded drainageway will be adequate. A mulch or geotextile mat should be used to protect the initial seeding. If the slope is located where adequate vegetation will not grow, a permanent erosion control geotextile covered with crushed rock or coarse sand can be used. The sizing of cover material should be based on flow velocities. The geotextile should be chosen to prevent erosion or piping of the underlying loess and strong enough to withstand placement of the cover material. Gradients greater than about 5 percent will require a liner similar to those used to convey water around the sides of cut slopes as described below. For clayey loess a drainage way behind the top of a cut slope is necessary only when concentrated flows would otherwise be directed over the slope face. In this case drainage should be the same as for silty loess. See Figure 10-5 for drainage details at the head of cut slopes in silty loess.



Drainage Above a Cut Slope in Silty Loess
(After Higgins and Fragaszy, 1988)
Figure 10-5

Drainage Around Sides of Cut Slopes – Drainageways around the sides of slopes generally have higher gradients (about 5 to 10 percent) than those at the tops of slopes. WSDOT WA-RD 145.2 (Higgins and Fragaszy, 1988) recommends four general designs for drainageways within this gradient range:

1. Line the drainageway with permanent erosion control geotextile and cover with coarse crushed rock.
2. Line the drainageway with permanent erosion control geotextile under a gabion blanket.
3. Construct the drainageway with a half-rounded pipe. The pipe should be keyed into the top of the slope to prevent erosional failure, and adequate compaction should be provided around the pipe to prevent erosion along the soil/pipe interface. Care should be taken to prevent leakage at pipe joints.
4. Line the drainageway with asphalt or concrete. This approach is expensive, and leakage can lead to piping and eventual collapse of the channel.

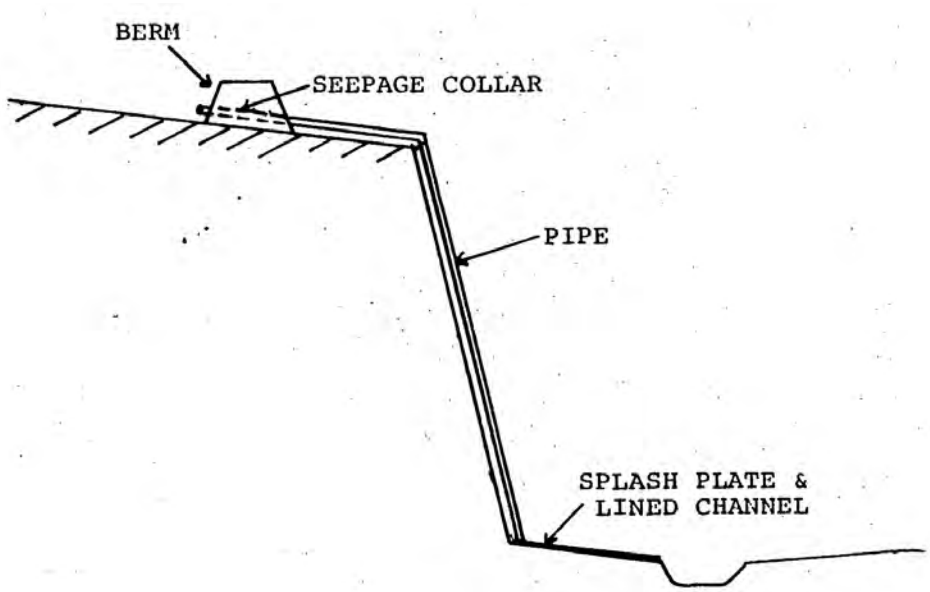
Drainage Over the Face of Cut Slopes – Where cuts will truncate an existing natural drainage basin, it is often necessary to convey water directly over the face of slopes due to the excessive ROW required to convey water around the sides. At no point should water be allowed to flow freely over the unprotected face of a cut slope. WSDOT WA-RD145.2 (Higgins and Fragaszy, 1988) lists three possible designs for this scenario in clayey loess and two possible designs in silty loess. For clayey loess:

1. Cut a shallow, flat bottomed ditch into the slope face. The ditch should be lined with permanent erosion control geotextile and covered with a gabion mat or coarse rock

2. Use a half-rounded pipe as described previously.
3. Use an asphalt or concrete liner.

For silty loess with a near vertical slope:

1. Intercept the drainage high enough above the cut to channel it around the sides using techniques described previously for drainage around the sides of cut slopes.
2. Convey water over the slope face using a PVC pipe connected to a collection area impounded by a berm located above the head of the slope. The pipe should be installed above the ground and sealed against the berm to prevent seepage along the outside of the pipe. The pipe also should be anchored both above and below the slope face, and a splash plate should be provided at the bottom to prevent undercutting of the slope. Figure 10-6 shows details of drainage over a cut face. This design is best suited for low to moderate flow volumes in conjunction with berm drainage. It should not be used with ditches.



Drainage Over a Cut Slope
(After Higgins and Fragaszy, 1988)
Figure 10-6

Drainage at the Toe of slopes – Drainage ditches along the roadway should be constructed at least 10 feet from the toe of the slope, and the ground surface should be gently sloped toward the ditch.

Sufficient right-of-way should be available to ensure that future agricultural activities are kept away from the top of the cut slope to keep drainageways from being filled in and to limit excessive disturbance around the cut slope.

Finally, proper construction control should be implemented. Construction equipment should be kept away from the top of the slope once the cut has been made. The following recommendations all have the same focus, to limit the amount of water that might reach the slope face. Construction should be performed during the summer, if possible. Drainage ways above the top of the cut should be constructed prior to opening up the cut. Seeding or other slope protection should be implemented immediately following construction of the cut. All cut slopes should be uniform, i.e. compound slopes should not be allowed. If animal holes are present that would create avenues for piping, they should be backfilled with low permeability fines or grout.

A design checklist taken from WA-RD 145.2 (Higgins and Fragaszy, 1988) is included in *Appendix 10-A*.

10.6 PS&E Considerations

Considerations concerning PS&E and construction generally consist of specifying the extents and periods during which earthwork is permitted in order to limit soil disturbance and erosion. Specifications should also be included that require construction of adequate drainage structures prior to grubbing and that construction equipment stay away from the tops of completed cut slopes.

In general, excavation for slopes should proceed in the uphill direction to allow surface or subsurface water exposed during excavation to drain without becoming ponded. Cut slopes should not be cut initially steeper, and then trimmed back after mass excavation. This procedure can result in cracks and fissures opening up in the oversteepened slope, allowing infiltration of surface water and a reduction in soil shear strength.

Both permanent and temporary cuts in highly erodable soil should be covered as they are excavated. Vegetation should be established on permanent slopes as soon as feasible. Only uniform slopes should be constructed in loess or other erodable soil (no compound slopes) in order to prevent erosion and undercutting.

10.7 References

AASHTO "Standard Specifications for Transportation Materials and Methods of Sampling and Testing," Part 2: Tests.

Alaska DOT, *Geotechnical Procedures Manual*, 1983.

Abramson, L., Boyce, G., Lee, T., and Sharma, S., 1996, *Slope Stability and Stabilization Methods*, Wiley, ISBN 0471106224.

FHWA, 1993, *Soil and Base Stabilization and Associated Drainage Considerations*, Vol 1, FHWA-SA-93-004.

FHWA, 1993, *Soil and Base Stabilization and Associated Drainage Considerations*, Vol 2, FHWA-SA-93-005

Higgins, J. D., and Fragaszy, R. J., and Martin, T., 1987, *Engineering Design in Loess Soils of Southeastern Washington*, WSDOT Research Report WA-RD 145.1, 121 pp.

Higgins, J. D., and Fragaszy, R. J., 1988, *Design Guide for Cut Slopes in Loess of Southeastern Washington*, WSDOT Research Report WA-RD 145.2, 57 pp.

Design Manual M 22-01

Highway Runoff Manual M 31-16, 2004

Roadside Manual.

Standard Specifications for Road, Bridge, and Municipal Construction M 21-01, 2004

Washington State Department of Transportation Loess Slope Design Checklist

The Loess Site Design Checklist has been prepared to aid the geotechnical engineer in the preliminary site investigation, field investigation layout, and design evaluation of highway construction in a loess soil region where cut slopes are required. This checklist was adapted from the Design Guide for Cut Slopes in Loess of Southeastern Washington, WA-RD 142.5 (Higgins and Fragaszy, 1988).

The checklist has been organized into five categories. The five categories include:

1. Project Definition
2. Project Field Data
3. Geotechnical Investigation
4. Laboratory Testing
5. Design Evaluation and Recommendations

Project Definition	Yes	No	N/A
1. Is the proposed construction within a loess region?	❒	❒	❒
If yes, what loess type is present? (Figure 10.3) ❒ Sandy Loess ❒ Silty Loess ❒ Clayey Loess			
2. Does the proposed construction involve complete realignment?	❒	❒	❒
3. Does the proposed construction involve minor realignment?	❒	❒	❒
4. Has an assessment been made of the current land management activities, e.g. review recent aerial photography?	❒	❒	❒
5. Has an assessment been made of the potential for land use changes, e.g. converting dryland farming to irrigation farming?	❒	❒	❒

Project Field Data	Yes	No	N/A
1. Is a county soil survey report available for review? If yes, answer the following:	❒	❒	❒
a. Have major soil types along the proposed route been identified?	❒	❒	❒
b. Have important soil parameters of those major soil types been identified? i.e. grain size distribution, percent clay vs. depth, permeability, drainage, depth to bedrock, agricultural use, irrigation potential.	❒	❒	❒
2. Have plans, profiles and cross sections been reviewed?	❒	❒	❒
3. Do the cross sections show the existing ground line beyond the top of the proposed cut?	❒	❒	❒
4. Have all major cut and fill slopes been located?	❒	❒	❒
5. What cut slope inclinations are desired by the Region: ____ ¼:1 ____2.5:1 or ____other If other, identify proposed cut slope angle and reason.	❒	❒	❒
6. If ¼:1 cuts area proposed, is there sufficient right-of-way to accommodate the required drainage facilities and fencing?	❒	❒	❒
7. Are there any existing or proposed structures present near the top of the proposed backslope?	❒	❒	❒

Geotechnical Investigation	Yes	No	N/A
1. Does the site investigation meet the minimum requirements established by WSDOT and FHWA, e.g. frequency of sampling holes, depth of holes, sample of frequency, hole locations, etc.?	❐	❐	❐
2. Were all major cuts represented by samples taken at depth in the loess?	❐	❐	❐
3. Were all cut slope aspects represented in the sampling process?	❐	❐	❐
4. On projects where minor sliver cuts are required, did sampling (hand auger holes) along the face of the existing cut extend a minimum of 4 feet into the face?	❐	❐	❐
5. Has the soil sampling been continuous in the top 6 feet and then every 5 feet thereafter?	❐	❐	❐
6. Was the soil investigation conducted during the wet time of year?	❐	❐	❐
7. Was natural field moisture determined from samples sealed in soil sample cans?	❐	❐	❐
8. Was groundwater encountered in any of the test borings?	❐	❐	❐
If yes, were piezometers installed for monitoring purposes?	❐	❐	❐
9. Is the groundwater perched on an impermeable layer (i.e. bedrock)?	❐	❐	❐
10. Will the proposed cut daylight the groundwater table?	❐	❐	❐
11. Has a field review of the condition of existing loess slope cuts been made?	❐	❐	❐
12. What is the repose of the existing cuts in the vicinity of the proposed project?	❐	❐	❐
13. Are the existing cuts in ____good, ____average, ____poor condition? Explain in detail.	❐	❐	❐

Laboratory Testing	Yes	No	N/A
1. Have Atterberg limits been performed?	❐	❐	❐
2. Have hydrometer tests been performed?	❐	❐	❐
3. Have sieve analyses been performed?	❐	❐	❐
4. Has field moisture been calculated?	❐	❐	❐
5. Has the shear strength been determined on representative samples from cuts exceeding 50 feet in height?	❐	❐	❐

Design Evaluation and Recommendations	Yes	No	N/A
1. Has the laboratory data been summarized, i.e. graphs representing percent clay vs. depth, and percent field moisture with depth?	❒	❒	❒
2. Based on criteria in Figure 10.3 and Section 10.5 of this chapter has the project loess soil been appropriately classified as to type and critical moisture?	❒	❒	❒
3. Are the recommended cuts based on guidelines in Section 10.5 of this chapter?	❒	❒	❒
If answer is no, is a justification given?			
4. Were there specific recommendations made of erosion control, e.g. backslopes, sideslopes, ditches? (This is <u>absolutely critical</u> to the successful use of cut slopes in loess; surface runoff must be collected and discharge so as not to saturate and erode the cut face.)	❒	❒	❒
5. If ¼:1 cut slopes are recommended, answer the following:	❒	❒	❒
a. Has a drainage profile along the proposed ditch been established?	❒	❒	❒
b. Does the ditch extend to a cut/fill transition or to a drainage structure?	❒	❒	❒
c. If the gradient of the ditch exceeds 5 percent is there the provision for ditch erosion protection i.e. asphalt or concrete or rock/geotextile lined ditch?	❒	❒	❒
d. Is there the provision for discharging water (without saturating the cut slope) from the ditch to the road grade line at low water collection points along the ditch profile?	❒	❒	❒
e. Is the proposed drainage ditch a minimum of 10 feet from the face of the ¼:1 cut slope?	❒	❒	❒
f. Does the design include the construction of a controlled access fence?	❒	❒	❒
6. If 2.5:1 cut slopes are recommended answer the following:	❒	❒	❒
a. If the cut intersects a natural drainageway have provisions been made to discharge the water over or around the face?	❒	❒	❒
b. Where soil is exposed to concentrated flow, such as in a ditch, is there provision for erosion protection?	❒	❒	❒

11.1 Overview

Ground improvement is used to address a wide range of geotechnical engineering problems, including, but not limited to, the following:

- Improvement of soft or loose soil to reduce settlement, increase bearing resistance, and/or to improve overall stability for structure and wall foundations and/or for embankments.
- To mitigate liquefiable soils.
- To improve slope stability for landslide mitigation.
- To retain otherwise unstable soils.
- To improve workability and usability of fill materials.
- To accelerate settlement and soil shear strength gain.

Types of ground improvement techniques include the following:

- Vibrocompaction techniques such as stone columns and vibroflotation, and other techniques that use vibratory probes that may or may not include compaction of gravel in the hole created to help densify the soil
- Deep dynamic compaction
- Blast densification
- Geosynthetic reinforcement of embankments
- Wick drains, sand columns, and similar methods that improve the drainage characteristics of the subsoil and thereby help to remove excess pore pressure that can develop under load applied to the soil
- Grout injection techniques and replacement of soil with grout such as compaction grouting, jet grouting, and deep soil mixing
- Lime or cement treatment of soils to improve their shear strength and workability characteristics
- Permeation grouting and ground freezing (temporary applications only)

Each of these methods has limitations regarding their applicability and the degree of improvement that is possible.

Rock mass improvement techniques such as bolting dowelling, shotcreting, etc., are not presented in this chapter, but are addressed in Chapter 12.

11.2 Development of Design Parameters and Other Input Data for Ground Improvement Analysis

In general, the geotechnical investigation conducted to design the cut, fill, structure foundation, retaining wall, etc., that the improved ground is intended to support will be adequate for the design of the soil improvement technique proposed. However, specific soil information may need to be emphasized depending on the ground improvement technique selected.

For example, for vibro-compaction techniques, deep dynamic compaction, and blast densification, detailed soil gradation information is critical to the design of such methods, as minor changes in soil gradation characteristics could affect method feasibility. Furthermore, the in-situ soil testing method used (e.g., SPT testing cone testing, etc.) will need to correspond to the technique specified in the contract to verify performance of the ground improvement technique, as the test data obtained during design will be the baseline to which the improved ground will be compared. Other feasibility issues will need to be addressed if these types of techniques are used. Critical is the impact the vibrations caused by the improvement technique will have on adjacent structures. Investigation of the foundations and soil conditions beneath adjacent structures and utilities may be needed, in addition to precondition surveys of the structures to enable identification of any damage caused by the ground improvement technique, if the risk of damage to adjacent structures and utilities is estimated to be acceptably low.

For wick drains, the ability to penetrate the soil with the wick drain mandrel, in addition to obtaining good rate of settlement information, must be assessed. Good Atterberg limit and water content data should be obtained, as well as any other data that can be useful in assessing the degree of overconsolidation of the soil present, if any.

Grout injection techniques (not including permeation grouting) can be used in a fairly wide range of soils, provided the equipment used to install the grout can penetrate the soil. The key here is to assess the ability of the equipment to penetrate the soil, assign the soil density and the potential for obstructions such as boulders.

Permeation grouting is more limited in its application, and its feasibility is strongly dependent on the ability of the grout to penetrate the soil matrix under pressure. Detailed grain size characterization and permeability assessment must be conducted, as well as the effect ground water may have on these techniques, to evaluate the feasibility of these techniques. An environmental assessment of such techniques may also be needed, especially if there is potential to contaminate groundwater supplies. These techniques are highly specialized and require the approval of the State Geotechnical Engineer before proceeding with a design based on using these techniques.

Similarly, ground freezing is a highly specialized technique that is strongly depending on the soil characteristics and groundwater flow rates present. Again, approval of the State Geotechnical Engineer is required before proceeding with a design based on using this technique.

11.3 Design Requirements

The design requirements provided in FHWA manual No. FHWA-SA-98-086 “Ground Improvement Technical Summaries” (Elias, et al., 2000) shall be followed. In addition, for stone column design, FHWA Report No. FHWA/RD-83/O2C “Design and Construction of Stone Columns” (Barkdale and Bachus, 1983) shall be used, for deep dynamic compaction, FHWA manual No. FHWA-SA-95-037, Geotechnical Engineering Circular No. 1, “Dynamic Compaction” (Lukas, 1995) shall be used, and for wick drain design, FHWA manual FHWA/RD-86/168 “Prefabricated Vertical Drains – A design and Construction Guidelines Manual” (Rixner, et al., 1986) shall be used.

For blast densification, the methodology and general approach described in Kimmerling (1994), and the additional design guidelines provided by Mitchell (1981) should be used. For lime and cement treatment of soils, Alaska DOT/FHWA Report No. FHWA-AK-RD-01-6B “Alaska Soil Stabilization Design Guide” (Hicks, 2002) shall be used for design. Design of geosynthetic base reinforcement and reinforced slopes are addressed in Chapters 9 and 15, respectively.

11.4 References

Barkdale, R. D., and Bachus, R. C., 1983, *Design and Construction of Stone Columns – Vol. 1*, Federal Highway Administration, FHWA/RD-83/02C.

Elias, V., Welsh, J., Warren, J., and Lukas, R., 2000, *Ground Improvement Technical Summaries – Vol. 1 and 2*, Demonstration Project 116, Federal Highway Administration, FHWA-SA-98-086.

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Kimmerling, R. E., 1994, *Blast Densification for Mitigation of Dynamic Settlement and Liquefaction*, WSDOT Research Report WA-RD 348.1, 114 pp.

Lukas, R. G., 1995, *Geotechnical Engineering Circular No. 1 – Dynamic Compaction*, Federal Highway Administration, FHWA-SA-95-037.

Mitchell, J. K., 1981, *Soil Improvement: State-of-the-Art Report*, Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Sweden, pp. 509-565.

Rixner, J. J., Kraemer, S. R., and Smith, A. D., 1986, *Prefabricated Vertical Drains – Vol. 1: Engineering Guidelines*, Federal Highway Administration, FHWA/RD-86/168.

12.1 Overview

This chapter addresses the assessment of stable slopes for rock cuts, including planning for excavation (e.g., blasting plan development), and rock mass improvement techniques such as bolting, dowelling, shotcreting, etc., to produce a stable slope.

12.2 Development of Design Parameters and Other Input Data for Rock Cut Stability Analysis

In addition to the site reconnaissance and geotechnical investigation requirements described in Chapter 2, rock slope design heavily relies upon surface mapping and discontinuity logging in boreholes of rock structure to assess discontinuities (fracture/joint) patterns and conditions, as discontinuities strongly control rock slope stability. In some cases, test hole data should also obtained, especially if surface mapping is not feasible due to the presence of overburden soil or for other reasons. Assessment of ground water present in the rock discontinuities, as is true of any slope, is critical to the assessment of stability. The detailed requirements for site investigation and analysis of rock cuts provided in FHWA HI-99-007 "Rock Slopes Reference Manual" (Munfakh, et al., 1998) shall be used. In addition to the requirements provided in the FHWA manual, design parameters shall be developed in accordance with Chapter 5.

12.3 Design Requirements

The detailed requirements for design of rock cuts provided in FHWA HI-99-007 "Rock Slopes Reference Manual" (Munfakh, et al., 1998) shall be used. In addition, for the development of blasting plans for rock cut excavation, the FHWA manual entitled "Rock Blasting and Overbreak Control,"FHWA-HI-92-001 (Konya and Walter, 1991) shall be used.

12.4 References

Konya, C. J., and Walter, E. J., 1991, *Rock Blasting and Overbreak Control*, Federal Highway Administration, FHWA-HI-92-001.

Munfakh, G., Wyllie, D., and Mah, C. W., 1998, *Rock Slopes Reference Manual*, Federal Highway Administration, FHWA HI-99-007.

Chapter 13 Landslide Analysis and Mitigation

13.1 Overview

This chapter addresses the assessment of landslides in soil and rock, and the development of the mitigating measures needed to stabilize the landslide.

13.2 Development of Design Parameters and Other Input Data for Landslide Analysis

In addition to the site reconnaissance and geotechnical investigation requirements described in Chapter 2, the exploration requirements provided in Special TRB Report 247 “Landslides Investigation and Mitigation”, Turner and Schuster, editors (1996) or “Landslides in Practice” by Cornforth (2005). Soil and rock properties for use in landslide analysis and mitigation shall be developed in accordance with Chapter 5.

13.3 Design Requirements

For landslides in soil and soft rock, the slope stability analysis methods and design requirements specified in Chapter 7 shall be used. For rockslides, the stability analysis method specified in Chapter 12 shall be used. The detailed requirements for analysis and mitigation design of landslides shall in addition be conducted in accordance with Special TRB Report 247 “Landslides Investigation and Mitigation”, Turner and Schuster, editors (1996) or “Landslides in Practice” by Cornforth (2005).

13.4 References

Cornforth, D. H., 2005, Landslides in Practice, John Wiley and Sons, Hoboken, NJ, 596 pp.

Turner, A. K., and Schuster, R. L., editors, 1996, Landslides Investigation and Mitigation, Transportation Research Board, TRB Special Report 247, National Academy Press, Washington, DC, 673 pp.

Chapter 14 Unstable Rockslope Analysis and Mitigation

14.1 Overview

This chapter addresses the assessment of unstable rockslopes and the development of the mitigating measures needed to stabilize the rockslope or to safely prevent the rockfall from reaching the traveled way.

14.2 Development of Design Parameters and Other Input Data for Unstable Rockslope Analysis

In addition to the site reconnaissance and geotechnical investigation requirements described in Chapter 2, assessment of unstable rockslopes heavily relies upon surface mapping of rock structure to assess fracture/joint patterns and conditions, as rock fractures and joints strongly control rock slope stability, and observations from past rockfall events. The detailed requirements for investigation of unstable rockslopes provided in FHWA manual No. FHWA SA-93-085, "Rockfall Hazard Mitigation Methods" (Brawner, 1994).

14.3 Design Requirements

The design requirement specified in Chapter 12 for Rock cut design are applicable to assessment and stabilization of unstable rockslopes. In addition, to address the prediction of rockfall and its mitigation, the design requirements provided in FHWA manual No. FHWA SA-93-085, "Rockfall Hazard Mitigation Methods" (Brawner, 1994) shall be used.

14.4 References

Brawner, C.O., 1994, *Rockfall Hazard Mitigation Methods*, Federal Highway Administration,FHWA SA-93-085.

Chapter 15 Abutments, Retaining Walls, and Reinforced Slopes

15-1 Introduction and Design Standards

This chapter addresses the geotechnical design of the abutments as well as retaining walls and reinforced slopes. Abutments for bridges have components of both foundation design and wall design. Retaining walls and reinforced slopes are typically included in projects to minimize construction in wetlands, to widen existing facilities, and to minimize the amount of right of way needed in urban environments. Projects modifying existing facilities often need to modify or replace existing retaining walls or widen abutments for bridges.

There tends to be confusion regarding when they should be incorporated into a project, what types are appropriate, how they are designed, who designs them, and how they are constructed. The roles and responsibilities of the various WSDOT offices and those of the Department's consultants further confuse the issue of retaining walls and reinforced slopes, as many of the roles and responsibilities overlap or change depending on the wall type. This chapter does not fully address the roles and responsibilities of the various WSDOT offices with regard to wall and abutment design, and the design process that should be used. The *Design Manual* M 22-01 Chapter 730, should be consulted for additional guidance on these issues.

All abutments, retaining walls, and reinforced slopes within WSDOT Right of Way or whose construction is administered by WSDOT shall be designed in accordance with the *Geotechnical Design Manual* (GDM) and the following documents:

- *Bridge Design Manual (LRFD)* M 23-50
- *Design Manual* M 22-01
- AASHTO *LRFD Bridge Design Specifications*, U.S.

The most current versions or editions of the above referenced manuals including all interims or design memoranda modifying the manuals shall be used. In the case of conflict or discrepancy between manuals, the following hierarchy shall be used: Those manuals listed first shall supersede those listed below in the list.

The following manuals provide additional design and construction guidance for retaining walls and reinforced slopes and should be considered supplementary to the GDM and the manuals and design specifications listed above:

- Lazarte, C. A., Robinson, H., Gomez, J. E., Baxter, A., Cadden, A., and Berg, R., 2015. *Geotechnical Engineering Circular No. 7*, *Soil Nail Walls – Reference Manual*, U.S. Department of Transportation, Federal Highway Administration, FHWA-NHI-14-007, 425 pp.
- Porterfield, J. A., Cotton, D. A., Byrne, R. J., 1994, *Soil Nail Walls-Demonstration Project 103*, *Soil Nailing Field Inspectors Manual*, U.S. Department of Transportation, Federal Highway Administration, FHWA-SA-93-068, 86 pp.
- Samtani, N. C., and Nowatzki, E. A., 2006, *Soils and Foundations*, *Reference Manual-Volumes I and II*, Washington, D.C., National Highway Institute Publication, FHWA-NHI-06-088/089, Federal Highway Administration.

- Berg, R. R., Christopher, B. R., and Samtani, N. C., 2009, *Design of Mechanically Stabilized Earth Walls and Reinforced Slopes*, No. FHWA-NHI-10-024, Federal Highway Administration, 306 pp.
- Sabatini, P. J., Pass, D. G., and Bachus, R. C., 1999, *Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems*, FHWA-IF-99-015, 281 pp.

15-2 Overview of Wall Classifications and Design Process for Walls

The various walls and wall systems can be categorized based on how they are incorporated into construction contracts. Standard Walls comprise the first category and are the easiest to implement. Standard walls are those walls for which standard designs are provided in the WSDOT *Standard Plans*. The internal stability design and the external stability design for overturning and sliding stability have already been addressed in the Standard Plan wall design, and bearing resistance, settlement, and overall stability must be determined for each standard-design wall location by the geotechnical designer. All other walls are nonstandard, as they are not included in the *Standard Plans*.

Nonstandard walls may be further subdivided into proprietary or nonproprietary. Nonstandard, proprietary walls are patented or trademarked wall systems designed and marketed by a wall manufacturer. The wall manufacturer is responsible for internal stability. Sliding stability, eccentricity, bearing resistance, settlement, compound stability, and overall slope stability are determined by the geotechnical designer. Nonstandard, nonproprietary walls are not patented or trade marked wall systems. However, they may contain proprietary elements. An example of this would be a gabion basket wall. The gabion baskets themselves are a proprietary item.

However, the gabion manufacturer provides gabions to a consumer, but does not provide a designed wall. It is up to the consumer to design the wall and determine the stable stacking arrangement of the gabion baskets. Nonstandard, nonproprietary walls are fully designed by the geotechnical designer and, if structural design is required, by the structural designer. Reinforced slopes are similar to nonstandard, nonproprietary walls in that the geotechnical designer is responsible for the design, but the reinforcing may be a proprietary item.

A number of proprietary wall systems have been extensively reviewed by the Bridge and Structures Office and the HQ Geotechnical Office. This review has resulted in WSDOT preapproving some proprietary wall systems. The design procedures and wall details for these preapproved wall systems shall be in accordance with this manual and other manuals specifically referenced herein as applicable to the type of wall being designed, unless alternate design procedures have been agreed upon between WSDOT and the proprietary wall manufacturer. These preapproved design procedures and details allow the manufacturers to competitively bid a particular project without having a detailed wall design provided in the contract plans. Note that proprietary wall manufacturers may produce several retaining wall options, and not all options from a given manufacturer have been preapproved. The Bridge and Structures Office shall be contacted to obtain the current listing of preapproved proprietary systems prior to including such systems in WSDOT projects. A listing of the preapproved wall systems, as of the current publication date for this manual, is provided in Appendix 15-D. Specific preapproved details and system specific design requirements for each wall system are also included as appendices to Chapter 15. Incorporation of non-preapproved systems requires the wall supplier to completely design the wall prior to advertisement for construction.

All of the manufacturer's plans and details would need to be incorporated into the contract documents. Several manufacturers may need to be contacted to maintain competitive bidding. More information is available in chapters 610 and 730 of the *Design Manual* M 22-01.

If it is desired to use a non-preapproved proprietary retaining wall or reinforced slope system, review and approval for use of the wall or slope system on WSDOT projects shall be based on the submittal requirements provided in Appendix 15-C. The wall or reinforced slope system, and its design and construction, shall meet the requirements provided in this manual, including Appendix 15-A. For Mechanically Stabilized Earth (MSE) walls, the wall supplier shall demonstrate in the wall submittal that the proposed wall system can meet the facing performance tolerances provided in Appendix 15-A through calculation, construction technique, and actual measured full scale performance of the wall system proposed.

Note that MSE walls are termed Structural Earth (SE) walls in the *Standard Specifications* M 41-10 and associated General Special Provisions (GSPs). In the general literature, MSE walls are also termed reinforced soil walls. In this GDM, the term "MSE" is used to refer to this type of wall.

15-3 Required Information

15-3.1 Site Data and Permits

The *Design Manual* M 22-01 discusses site data and permits required for design and construction. In addition, chapters 610 and 730 provide specific information relating to geotechnical work and retaining walls.

15-3.2 Geotechnical Data Needed for Retaining Wall and Reinforced Slope Design

The project requirements, site, and subsurface conditions should be analyzed to determine the type and quantity of information to be developed during the geotechnical investigation. It is necessary to:

- Identify areas of concern, risk, or potential variability in subsurface conditions.
- Develop likely sequence and phases of construction as they may affect retaining wall and reinforced slope selection.
- Identify design and constructability requirements or issues such as:
  - *Surcharge loads from adjacent structures*
  - *Backslope and toe slope geometries*
  - *Right of way restrictions*
  - *Materials sources*
  - *Easements*
  - *Excavation limits*
  - *Wetlands*
  - *Construction Staging*
- Identify performance criteria such as:
  - *Tolerable settlements for the retaining walls and reinforced slopes*
  - *Tolerable settlements of structures or property being retained*
  - *Impact of construction on adjacent structures or property*
  - *Long-term maintenance needs and access*

- Identify engineering analyses to be performed:
  - *Bearing resistance*
  - *Settlement*
  - *Global stability*
  - *Internal stability*
- Identify engineering properties and parameters required for these analyses.
- Identify the number of tests/samples needed to estimate engineering properties.

Table 15-1 provides a summary of information needs and testing considerations for retaining walls and reinforced slope design.

Chapter 5 covers requirements for how the results from the field investigation, the field testing, and laboratory testing are to be used to establish properties for design. The specific tests and field investigation requirements needed for foundation design are described in the following sections.

Table 15-1 Summary of Information Needs and Testing Considerations

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Fill Walls/ Reinforced Soil Slopes	• internal stability • external stability • global and compound stability • limitations on rate of construction • settlement • horizontal deformation? • lateral earth pressures? • bearing capacity? • chemical compatibility with soil, groundwater, and wall materials? • pore pressures behind wall • borrow source evaluation (available quantity and quality of borrow soil) • liquefaction • potential for subsidence (karst, mining, etc.) • constructability • scour	• subsurface profile (soil, ground water, rock) • horizontal earth pressure coefficients • interface shear strengths • foundation soil/wall fill shear strengths? • compressibility parameters? (including consolidation, shrink/ swell potential, and elastic modulus) • chemical composition of fill/ foundation soils? • hydraulic conductivity of soils directly behind wall? • time-rate consolidation parameters? • geologic mapping including orientation and characteristics of rock discontinuities? • design flood elevations • seismicity	• SPT • CPT • dilatometer • vane shear • piezometers • test fill? • nuclear density? • pullout test (MSEW/ RSS) • rock coring (RQD) • geophysical testing	• 1-D Oedometer • triaxial tests • unconfined compression • direct shear tests • grain size distribution • Atterberg limits • specific gravity • pH, resistivity, chloride, and sulfate tests? • moisture content? • organic content • moisture-density relationships • hydraulic conductivity

Table 15-1 Summary of Information Needs and Testing Considerations

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Cut Walls	• internal stability • external stability • excavation stability • global and compound stability • dewatering • chemical compatibility of wall/soil • lateral earth pressure • down-drag on wall • pore pressures behind wall • obstructions in retained soil • liquefaction • see page • potential for subsidence (karst, mining, etc.) • constructability	• subsurface profile (soil, ground water, rock) • shear strength of soil • horizontal earth pressure coefficients • interface shear strength (soil and reinforcement) • hydraulic conductivity of soil • geologic mapping including orientation and characteristics of rock discontinuities • seismicity	• test cut to evaluate stand-up time • well pumping tests • piezometers • SPT • CPT • vane shear • dilatometer • pullout tests (anchors, nails) • geophysical testing	• triaxial tests • unconfined compression • direct shear • grain size distribution • Atterberg limits • specific gravity • pH, resistivity tests • organic content • hydraulic conductivity • moisture content • unit weight

15-3.3 Site Reconnaissance

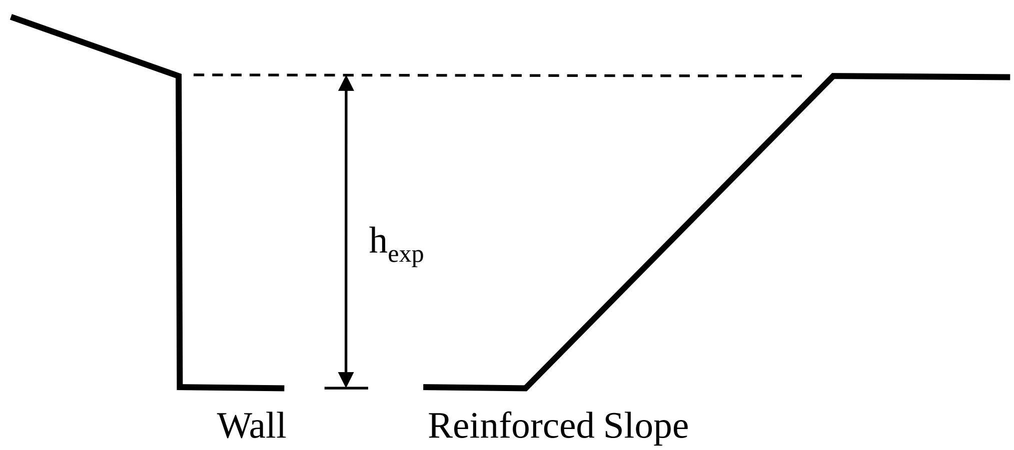
For each abutment, retaining wall, and reinforced slope, the geotechnical designer should perform a site review and field reconnaissance. The geotechnical designer should be looking for specific site conditions that could influence design, construction, and performance of the retaining walls and reinforced slopes on the project. This type of review is best performed once survey data has been collected for the site and digital terrain models, cross-sections, and preliminary wall profiles have been generated by the civil engineer (e.g., region project engineer). In addition, the geotechnical designer should have access to detailed plan views showing existing site features, utilities, proposed construction, and right or way limits. With this information, the geotechnical designer can review the wall/slope locations making sure that survey information agrees reasonably well with observed site topography. The geotechnical designer should observe where utilities are located, as they will influence where field exploration can occur and they may affect design or constructability. The geotechnical designer should look for indications of soft soils or unstable ground. Items such as hummocky topography, seeps or springs, pistol butted trees, and scarps, either old or new, need to be investigated further. Vegetative indicators such as equisetum (horsetails), cat tails, black berry, or alder can be used to identify soils that are wet or unstable. A lack of vegetation can also be an indicator of recent slope movement. In addition to performing a basic assessment of site conditions, the geotechnical designer should also be looking for existing features that could influence design and construction such as nearby structures, surcharge loads, and steep back or toe slopes. This early in design, it is easy to overlook items such as construction access, materials sources, and limits of excavation. The geotechnical designer needs to be cognizant of these issues and should be identifying access and excavation issues early, as they can affect permits and may dictate what wall type may or may not be used.

15-3.4 Field Exploration Requirements

A soil investigation and geotechnical reconnaissance is critical for the design of all abutments, retaining walls, or reinforced slopes. The stability of the underlying soils, their potential to settle under the imposed loads, the usability of any existing excavated soils for wall/reinforced slope backfill, and the location of the ground water table are determined through the geotechnical investigation. All abutments, retaining, walls and reinforced slopes regardless of their height require an investigation of the underlying soil/rock that supports the structure. Abutments shall be investigated like other bridge piers in accordance with Chapter 8.

Retaining walls and reinforced slopes that are equal to or less than 10 feet in exposed height, h_{exp}, as measured vertically from wall bottom to top or from slope toe to crest, as shown in Figure 15-1, shall be investigated in accordance with Sections 15-3.4.1 and 15.3.4.2. For all retaining walls and reinforced slopes greater than 10 feet in exposed height, the field exploration shall be completed in accordance with the AASHTO *LRFD Bridge Design Specifications* and this manual.

Figure 15-1 Exposed Height (H) for a Retaining Wall or Slope



Explorations consisting of geotechnical borings, test pits, hand holes, or a combination thereof shall be performed at each wall or slope location. Geophysical testing may be used to supplement the subsurface exploration and reduce the requirements for borings. If the geophysical testing is done as a first phase in the exploration program, it can also be used to help develop the detailed plan for second phase exploration. As a minimum, the subsurface exploration and testing program should obtain information to analyze foundation stability and settlement with respect to:

- Geological formation(s).
- Location and thickness of soil and rock units.
- Engineering properties of soil and rock units, such as unit weight, shear strength and compressibility.

- Ground water conditions.
- Ground surface topography.
- Local considerations (e.g., liquefiable, expansive or dispersive soil deposits, underground voids from solution weathering or mining activity, or slope instability potential).

In areas underlain by heterogeneous soil deposits and/or rock formations, it will probably be necessary to perform more investigation to capture variations in soil and/ or rock type and to assess consistency across the site area. In a laterally homogeneous area, drilling or advancing a large number of borings may be redundant, since each sample tested would exhibit similar engineering properties. In all cases, it is necessary to understand how the design and construction of the geotechnical feature will affect the soil and/or rock mass in order to optimize the exploration. The following minimum guidelines for frequency and depth of exploration shall be used. Additional exploration may be required depending on the variability in site conditions, wall/slope geometry, wall/slope type, and the consequences should a failure occur.

15-3.4.1 *Exploration Type, Depth, and Spacing*

Generally, walls 10 feet or less in height, constructed over average to good soil conditions (e.g., non-liquefiable, medium dense to very dense sand, silt or gravel, with no signs of previous instability) will require only a basic level of site investigation. A geologic site reconnaissance (see Chapter 2), combined with widely spaced test pits, hand holes, or a few shallow borings to verify field observations and the anticipated site geology may be sufficient, especially if the geology of the area is well known, or if there is some prior experience in the area.

The geotechnical designer should investigate to a depth below bottom of wall or reinforced slope at least to a depth where stress increase due to estimated foundation load is less than 10 percent of the existing effective overburden stress and between one and two times the exposed height of the wall or slope. Exploration depth should be great enough to fully penetrate soft highly compressible soils (e.g., peat, organic silt, soft fine grained soils) into competent material of suitable bearing capacity (e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock). Hand holes and test pits should be used only where medium dense to dense granular soil conditions are expected to be encountered within limits that can be reasonably explored using these methods, approximately 10 feet for hand holes and 15 feet for test pits, and that based on the site geology there is little risk of an unstable soft or weak layer being present that could affect wall stability.

For retaining walls and reinforced slopes less than 100 feet in length, the exploration should occur approximately midpoint along the alignment or where the maximum height occurs. Explorations should be completed on the alignment of the wall face or approximately midpoint along the reinforced slope, i.e., where the height, as defined in Figure 15-1, is 0.5H. Additional borings to investigate the toe slope for walls or the toe catch for reinforced slopes may be required to assess overall stability issues.

For retaining walls and slopes more than 100 feet in length, exploration points should in general be spaced at 100 to 200 feet, but may be spaced at up to 500 feet in uniform, dense soil conditions. Even closer spacing than 100 to 200 feet should be used in highly variable and potentially unstable soil conditions. Where possible, locate at least one boring where the maximum height occurs. Explorations should be completed on the alignment of the wall face or approximately midpoint along the reinforced slope, i.e., where the height is 0.5H. Additional borings to investigate the toe slope for walls or the toe catch for reinforced slopes may be required to assess overall stability issues. Exploration locations may be adjusted if geophysical testing conducted in accordance with Chapter 5 is done, provided enough borings are available to properly interpret the geophysical test results.

A key to the establishment of exploration frequency for walls is the potential for the subsurface conditions to impact the construction of the wall, the construction contract in general, and the long-term performance of the finished project. The exploration program should be developed and conducted in a manner that these potential problems, in terms of cost, time, and performance, are reduced to an acceptable level. The boring frequency described above may need to be adjusted by the geotechnical designer to address the risk of such problems for the specific project. Exploration locations may be adjusted if geophysical testing conducted in accordance with Chapter 5 is done, provided enough borings are available to properly interpret the geophysical test results.

15-3.4.2 *Walls and Slopes Requiring Additional Exploration*

15-3.4.2.1 *Soil Nail Walls*

Soil nail walls should have additional geotechnical borings completed to explore the soil conditions within the soil nail zone. The additional exploration points shall be at a distance of 1.0 to 1.5 times the height of the wall behind the wall to investigate the soils in the nail zone. For retaining walls and slopes more than 100 feet in length, exploration points should in general be spaced at 100 to 200 feet, but may be spaced at up to 500 feet in uniform, dense soil conditions. Even closer spacing than 100 to 200 feet should be used in highly variable and potentially unstable soil conditions. The depth of the borings shall be sufficient to explore the full depth of soils where nails are likely to be installed, and deep enough to address overall stability issues.

In addition, each soil nail wall should have at least one test pit excavated to evaluate stand-up time of the excavation face. The test pit shall be completed outside the nail pattern, but as close as practical to the wall face to investigate the stand-up time of the soils that will be exposed at the wall face during construction. The test pit shall remain open at least 24 hours and shall be monitored for sloughing, caving, and groundwater seepage. A test pit log shall be prepared and photographs should be taken immediately after excavation and at 24 hours. If variable soil conditions are present along the wall face, a test pit in each soil type should be completed. The depth of the test pits should be at least twice the vertical nail spacing and the length along the trench bottom should be at least one and a half times the excavation depth to minimize soil-arching effects. For example, a wall with a vertical nail spacing of 4 feet would have a test pit 8 feet deep and at least 12 feet in length at the bottom of the pit.

15-3.4.2.2 *Walls With Ground Anchors or Deadman Anchors*

Walls with ground anchors or deadman anchors should have additional geotechnical borings completed to explore the soil conditions within the anchor/deadman zone. For retaining walls more than 100 feet in length, exploration points should in general be spaced at 100 to 200 feet, but may be spaced at up to 500 feet in uniform, dense soil conditions. Even closer spacing than 100 to 200 feet should be used in highly variable and potentially unstable soil conditions. The borings should be completed outside the no-load zone of the wall in the bond zone of the anchors or at the deadman locations. The depth of the borings shall be sufficient to explore the full depth of soils where ground anchors or deadman anchors are likely to be installed, and deep enough to address overall stability issues.

15-3.4.2.3 *Wall or Slopes With Steep Back Slopes or Steep Toe Slopes*

Walls or slopes that have a back slopes or toe slopes that exceed 10 feet in slope length and that are steeper than 2H:1V should have at least one hand hole, test pit, or geotechnical boring in the backslope or toe slope to define stratigraphy for overall stability analysis and evaluate bearing resistance. The exploration should be deep enough to address overall stability issues. Hand holes and test pits should be used only where medium dense to dense granular soil conditions are expected to be encountered within limits that can be reasonably explored using these methods, approximately 10 feet for hand holes and 20 feet for test pits.

15-3.5 Field, Laboratory, and Geophysical Testing for Abutments, Retaining Walls, and Reinforced Slopes

The purpose of field and laboratory testing is to provide the basic data with which to classify soils and to estimate their engineering properties for design. Often for abutments, retaining walls, and reinforced slopes, the backfill material sources are not known or identified during the design process. For example, mechanically stabilized earth walls are commonly constructed of backfill material that is provided by the Contractor during construction. During design, the material source is not known and hence materials cannot be tested. In this case, it is necessary to design using commonly accepted values for regionally available materials and ensure that the contract will require the use of materials meeting or exceeding these assumed properties.

For abutments, the collection of soil samples and field testing shall be in accordance with chapters 2, 5, and 8.

For retaining walls and reinforced slopes, the collection of soil samples and field testing are closely related. Chapter 5 provides the minimum requirements for frequency of field tests that are to be performed in an exploration point. As a minimum, the following field tests shall be performed and soil samples shall be collected:

In geotechnical borings, soil samples shall be taken during the Standard Penetration Test (SPT). Fine grained soils or peat shall be sampled with 3-in Shelby tubes or WSDOT Undisturbed Samplers if the soils are too stiff to push 3-in Shelby tubes. All samples in geotechnical borings shall be in accordance with chapters 2 and 3.

In hand holes, sack soil samples shall be taken of each soil type encountered, and WSDOT Portable Penetrometer tests shall be taken in lieu of SPT tests. The maximum vertical spacing between portable penetrometer tests should be 5 feet.

In test pits, sack soil samples shall be taken from the bucket of the excavator, or from the spoil pile for each soil type encountered once the soil is removed from the pit. WSDOT Portable Penetrometer tests may be taken in the test pit. However, no person shall enter a test pit to sample or perform portable penetrometer tests unless there is a protective system in place in accordance with Washington Administrative Code (WAC) 296-155-657.

In soft soils, CPT tests or insitu vane shear tests may be completed to investigate soil stratigraphy, shear strength, and drainage characteristics.

All soil samples obtained shall be reviewed by a geotechnical engineer or engineering geologist. The geotechnical designer shall group the samples into stratigraphic units based on consistency, color, moisture content, engineering properties, and depositional environment. At least one sample from each stratigraphic unit should be tested in the laboratory for Grain Size Distribution, Moisture Content, and Atterberg limits. Additional tests, such as Loss on Ignition, pH, Resistivity, Sand Equivalent, or Hydrometer may be performed.

Walls that will be constructed on compressible or fine grained soils should have undisturbed soil samples available for laboratory testing, e.g., shelby tubes or WSDOT undisturbed samples. Consolidation tests and Unconsolidated Undrained (UU) triaxial tests should be performed on fine grained or compressible soil units. Additional tests such as Consolidated Undrained (CU), Direct Shear, or Lab Vane Shear may be performed to estimate shear strength parameters and compressibility characteristics of the soils.

Geophysical testing may be used for establishing stratification of the subsurface materials, the profile of the top of bedrock, depth to groundwater, limits of types of soil deposits, the presence of voids, anomalous deposits, buried pipes, and depths of existing foundations. Data from Geophysical testing shall always be correlated with information from direct methods of exploration, such as SPT, CPT, etc.

15-3.6 Groundwater

One of the principal goals of a good field reconnaissance and field exploration is to accurately characterize the groundwater in the project area. Groundwater affects the design, performance, and constructability of project elements. Installation of piezometer(s) and monitoring is usually necessary to define groundwater elevations. Groundwater measurements shall be conducted in accordance with Chapter 2, and shall be assessed for each wall. In general, this will require at least one groundwater measurement point for each wall. If groundwater has the potential to affect wall performance or to require special measures to address drainage to be implemented, more than one measurement point per wall will be required.

15-3.7 Wall Backfill Testing and Design Properties

The soil used as wall backfill may be tested for shear strength in lieu of using a lower bound value based on previous experience with the type of soil used as backfill (e.g., gravel borrow). See Chapter 5 (specifically Table 5-2) for guidance on selecting a shear strength value for design if soil specific testing is not conducted. A design shear strength value of 36° to 38° has been routinely used as a lower bound value for gravel borrow backfill for WSDOT wall projects. Triaxial tests conducted in accordance with AASHTO T296-95 (2000), but conducted on remolded specimens of the backfill compacted at optimum moisture content, plus or minus 3 percent, to 95 percent of maximum density per WSDOT Test Method T606, may be used to justify higher design friction angles for wall backfill, if the backfill source is known at the time of design. This degree of compaction is approximately equal to 90 to 95 percent of modified proctor density (ASTM D1557). The specimens are not saturated during shearing, but are left at the moisture content used during specimen preparation, to simulate the soil as it is actually placed in the wall. Note that this type of testing can also be conducted as part of the wall construction contract to verify a soil friction assumed for design.

Other typical soil design properties for various types of backfill and native soil units are provided in Chapter 5.

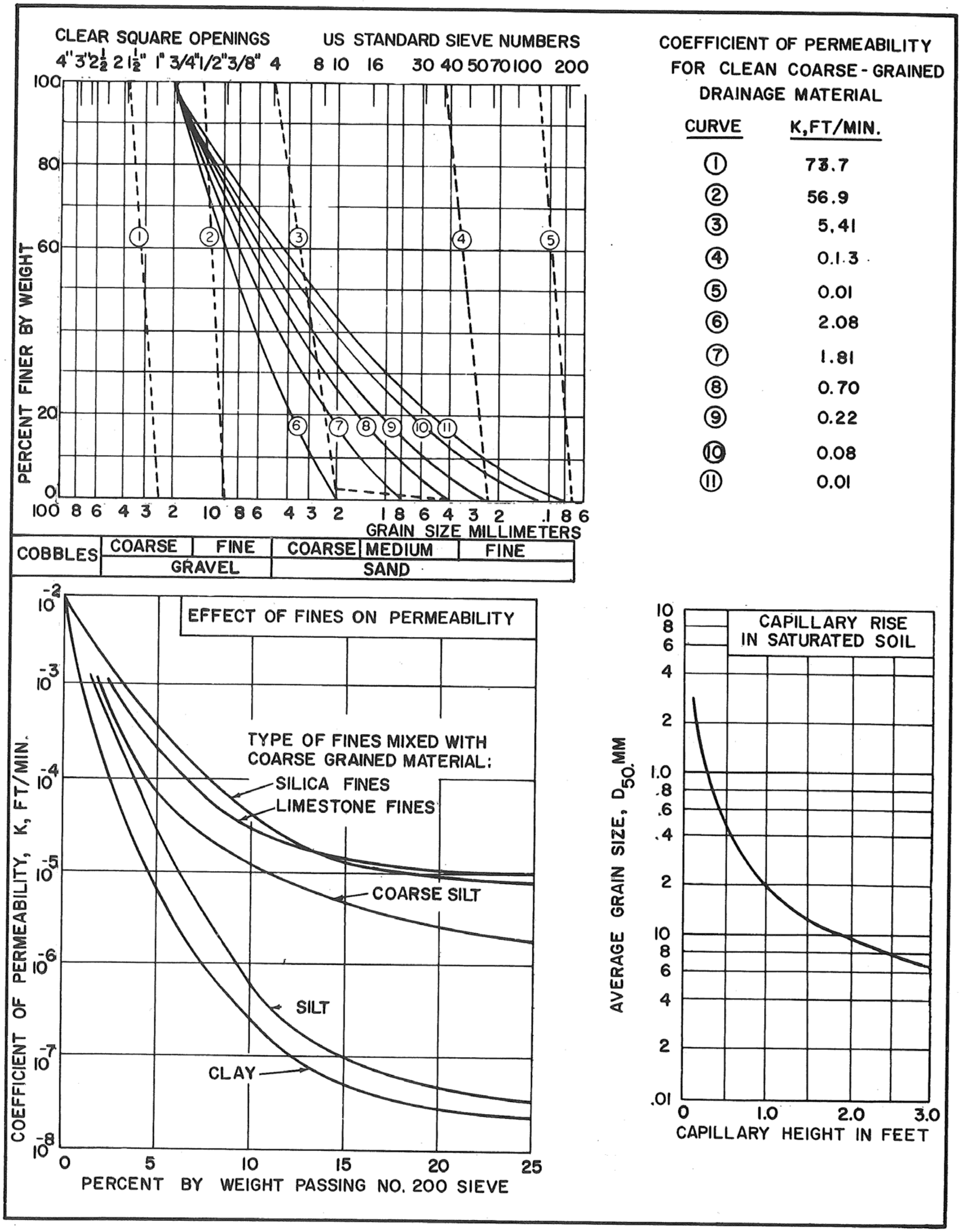
The ability of the wall backfill to drain water that infiltrates it from rain, snow melt, or ground water shall be considered in the design of the wall and its stability. Figure 15-2 illustrates the effect the percentage of fines can have on the permeability of the soil. In general, for a soil to be considered free draining, the fines content (i.e., particles passing the No. 200 sieve) should be less than 5 percent by weight. If the fines content is greater than this, the reinforced wall backfill cannot be fully depended upon to keep the reinforced wall backfill drained, and other drainage measures may be needed.

15-4 General Design Requirements

15-4.1 Design Methods

The AASHTO *LRFD Bridge Design Specifications* shall be used for all abutments and retaining walls addressed therein. The walls shall be designed to address all applicable limit states (strength, service, and extreme event). Rock walls, reinforced slopes, and soil nail walls are not specifically addressed in the AASHTO specifications, and shall be designed in accordance with this manual. Many of the FHWA manuals used as WSDOT design references were not developed for LRFD design. For those wall types (and including reinforced slopes) for which LRFD procedures are not available, allowable stress design procedures included in this manual, either in full or by reference, shall be used, again addressing all applicable limit states.

Figure 15-2 Permeability and Capillarity of Drainage Materials (Department of Defense 2005)



The load and resistance factors provided in the AASHTO LRFD Specifications have been developed in consideration of the inherent uncertainty and bias of the specified design methods and material properties, and the level of safety used to successfully construct thousands of walls over many years. These load and resistance factors shall only be applied to the design methods and material resistance estimation methods for which they are intended, if an option is provided in this manual or the AASHTO LRFD specifications to use methods other than those specified herein or in the AASHTO LRFD specifications. For estimation of soil reinforcement pullout in reinforced soil (MSE) walls, the resistance factors provided are to be used only for the default pullout methods provided in the AASHTO LRFD specifications. If wall system specific pullout resistance estimation methods are used, resistance factors shall be developed statistically using reliability theory to produce a probability of failure Pf of approximately 1 in 100 or smaller. Note that in some cases, Section 11 of the AASHTO *LRFD Bridge Design Specifications* refers to AASHTO LRFD Section 10 for wall foundation design and the resistance factors for foundation design. In such cases, the design methodology and resistance factors provided in the Chapter 8 shall be used instead of the resistance factors in AASHTO LRFD Section 10, where the GDM and the AASHTO Specifications differ.

For reinforced soil slopes, the FHWA manual entitled “Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines” by Berg, et al. (2009), or most current version of that manual, shall be used as the basis for design. The LRFD approach has not been developed as yet for reinforced soil slopes. Therefore, allowable stress design shall be used for design of reinforced soil slopes.

All walls shall meet the requirements in the *Design Manual* M 22-01 for layout and geometry. All walls shall be designed and constructed in accordance with the *Standard Specifications*, General Special Provisions, and *Standard Plans*. Specific design requirements for tiered walls, back-to-back walls, and MSE wall supported abutments are provided in the GDM as well as in the AASHTO *LRFD Bridge Design Specifications*, and by reference in those design specifications to FHWA manuals (Berg, et al. 2009).

15-4.2 Tiered Walls

Walls that retain other walls or have walls as surcharges require special design to account for the surcharge loads from the upper wall. Proprietary wall systems may be used for the lower wall, but proprietary walls shall not be considered preapproved in this case. Chapter 730 of the *Design Manual* M 22-01 discusses the requirements for utilizing non-preapproved proprietary walls on WSDOT projects. If the upper wall is proprietary, a preapproved system may be used provided it meets the requirements for preapproval and does not contain significant structures or surcharges within the wall reinforcing.

For tiered walls, the FHWA manual entitled “Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes” by Berg, et al. (2009), shall be used as the basis for design for those aspects of the design not covered in the AASHTO *LRFD Bridge Design Specifications* and the GDM.

15-4.3 Back-to-Back Walls

The face-to-face dimension for back-to-back sheetpile walls used as bulkheads for waterfront structures must exceed the maximum exposed height of the walls. Bulkhead walls may be cross braced or tied together provided the tie rods and connections are designed to carry twice the applied loads.

The face to face dimension for back to back Mechanically Stabilized Earth (MSE) walls should be 1.1 times the average height of the MSE walls or greater. Back-to- back MSE walls with a width/height ratio of less than 1.1 shall not be used unless approved by the State Geotechnical Engineer and the State Bridge Design Engineer. The maximum height for back-to-back MSE wall installations (i.e., average of the maximum heights of the two parallel walls) is 30 feet, again, unless a greater height is approved by the State Geotechnical Engineer and the State Bridge Design Engineer. Justification to be submitted to the State Geotechnical Engineer and the State Bridge Design Engineer for approval should include rigorous analyses such as would be conducted using a calibrated numerical model, addressing the force distribution in the walls for all limit states, and the potential deformations in the wall for service and extreme event limit states, including the potential for rocking of the back-to-back wall system.

The soil reinforcement for back-to-back MSE walls may be connected to both faces, i.e., continuous from one wall to the other, provided the reinforcing is designed for at least double the loading, if approved by the State Geotechnical Engineer. Reinforcement may overlap, provided the reinforcement from one wall does not contact the reinforcement from the other wall. Reinforcement overlaps of more than 3 feet are generally not desirable due to the increased cost of materials. Preapproved proprietary wall systems may be used for back-to-back MSE walls provided they meet the height, height/width ratio and overlap requirements specified herein. For seismic design of back-to-back walls in which the reinforcement layers overlap the walls may be considered able to slide to reduce the acceleration to be applied if both walls are free to slide. If the back-to-back walls are close enough together such that the active zones of the walls at least partially overlap, the inertial force of the walls shall be based on the total volume of both walls plus the retained soil between the walls.

For back-to-back walls, the FHWA manual entitled “Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes” by Berg, et al. (2009), shall be used as the basis for design for those aspects of the design not covered in the AASHTO *LRFD Bridge Design Specifications* and the GDM.

15-4.4 Walls on Slopes

Standard Plan walls founded on slopes shall meet the requirements in the *Standard Plans*. Additionally, all walls shall have a near horizontal bench at the wall face at least 4 feet wide to provide access for maintenance. Bearing resistance for footings in slopes and overall stability requirements in the AASHTO *LRFD Bridge Design Specifications* shall be met. Table C11.10.2.2-1 in the AASHTO *LRFD Bridge Design Specifications* should be used as a starting point for determining the minimum wall face embedment when the wall is located on a slope. Use of a smaller embedment must be justified based on slope geometry, potential for removal of soil in front of the wall due to erosion, future construction activity, etc., and external and global wall stability considerations.

15-4.5 Minimum Embedment

All walls and abutments should meet the minimum embedment criteria in AASHTO. The final embedment depth required shall be based on geotechnical bearing and stability requirements provided in the AASHTO LRFD specifications, as determined by the geotechnical designer (see also Section 15-4.4). Walls that have a sloping ground line at the face of wall may need to have a sloping or stepped foundation to optimize the wall embedment. Sloping foundations (i.e., not stepped) shall be 6H:1V or flatter. Stepped foundations shall be 1.5H:1V or flatter determined by a line through the corners of the steps. The maximum feasible slope of stepped foundations for walls is controlled by the maximum acceptable stable slope for the soil in which the wall footing is placed. Concrete leveling pads constructed for MSE walls shall be sloped at 6H:1V or flatter or stepped at 1.5H:1V or flatter determined by a line through the corners of the steps. As MSE wall facing units are typically rectangular shapes, stepped leveling pads are preferred.

In situations where scour (e.g., due to wave or stream erosion) can occur in front of the wall, the wall foundation (e.g., MSE walls, footing supported walls), the pile cap for pile supported walls, and for walls that include some form of lagging or panel supported between vertical wall elements (e.g., soldier pile walls, tieback walls), the bottom of the footing, pile cap, panel, or lagging shall meet the minimum embedment requirements relative to the scour elevation in front of the wall. A minimum embedment below scour of 2 feet, unless a greater depth is otherwise specified, shall be used.

15-4.6 Wall Height Limitations

Proprietary wall systems that are preapproved through the WSDOT Bridge and Structures Office are in general preapproved to 33 feet or less in total height. Greater wall heights may be used and for many wall systems are feasible, but a special design (i.e., not preapproved) may be required. The 33 feet preapproved maximum wall height can be extended for proprietary wall systems if approved by the State Geotechnical and Bridge Design Engineers.

Some types of walls may have more stringent height limitations. Walls that have more stringent height limitations include full height propped precast concrete panel MSE walls (Section 15-5.3.7), flexible faced MSE walls with a vegetated face (Section 15-5.3.8), MSE wall supported bridge abutments (Section 15-5.3.6), and modular dry cast concrete block faced systems (Section 15-5.3.9). Other specific wall systems may also have more stringent height limitations due to specific aspects of their design or the materials used in their construction.

15-4.7 Serviceability Requirements

Walls shall be designed to structurally withstand the effects of total and differential settlement estimated for the project site, both longitudinally and in cross-section, as prescribed in the AASHTO LRFD Specifications. In addition to the requirements for serviceability provided above, the following criteria (tables 15-2, 15-3, and 15-4) shall be used to establish acceptable settlement criteria (includes settlement that occurs during and after wall construction):

Table 15-2 Settlement Criteria for Reinforced Concrete Walls, Nongravity Cantilever Walls, Anchored/Braced Walls, and MSE Walls With Full Height Precast Concrete Panels (Soil is Place Directly Against Panel)

Total Settlement	Differential Settlement Over 100 Feet	Action
$\Delta H \leq 1$ in	$\Delta H_{100} \leq 0.75$ in	Design and Construct
1 in < $\Delta H \leq 2.5$ in	0.75 in < $\Delta H_{100} \leq 2$ in	Ensure structure can tolerate settlement
$\Delta H > 2.5$ in	$\Delta H_{100} > 2$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

Table 15-3 Settlement Criteria for MSE Walls With Modular (Segmental) Block Facings, Prefabricated Modular Walls, and Rock Walls

Total Settlement	Differential Settlement Over 100 Feet	Action
$\Delta H \leq 2$ in	$\Delta H_{100} \leq 1.5$ in	Design and Construct
2 in < $\Delta H \leq 4$ in	1.5 in < $\Delta H_{100} \leq 3$ in	Ensure structure can tolerate settlement
$\Delta H > 4$ in	$\Delta H_{100} > 3$ in	Obtain Approval[1] prior to proceeding with design and Construction
[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.		

Table 15-4 Settlement Criteria for MSE Walls With Flexible Facings and Reinforced Slopes, and Walls in Which the Structural Facing is Installed as a Second Construction Stage After the Wall Settlement is Complete

Total Settlement	Differential Settlement Over 50 Feet	Action
$\Delta H \leq 4$ in	$\Delta H_{50} \leq 3$ in	Design and Construct
4 in < $\Delta H \leq 12$ in	3 in < $\Delta H_{50} \leq 9$ in	Ensure structure can tolerate settlement
$\Delta H > 12$ in	$\Delta H_{50} > 9$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

For two-stage walls, Table 15-4 settlement limits apply to the first stage. In that case, the effect of that settlement on installation of the second stage facing shall be addressed. For the second stage facing, long-term settlement shall be limited to the values shown in tables 15-2 and 15-3.

For MSE walls with precast panel facings up to 75 feet^2 in area, limiting differential settlements shall be as defined in the AASHTO LRFD Specifications, Article C11.10.4.1, and total settlement shall be 4 inches or less unless approval by the WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer is obtained.

Note that more stringent tolerances than indicated in tables 15-2 to 15-4 may be necessary to meet aesthetic requirements for the walls.

15-4.8 Active, Passive, At-Rest Earth Pressures

The geotechnical designer shall assess soil conditions and shall develop earth pressure diagrams for all walls except standard plan walls in accordance with the AASHTO *LRFD Bridge Design Specifications*. Earth pressures may be based on either Coulomb or Rankine theories. The type of earth pressure used for design depends on the ability of the wall to yield in response to the earth loads. For walls that free to translate or rotate (i.e., flexible walls), active pressures shall be used in the retained soil. Flexible walls are further defined as being able to displace laterally at least 0.001H, where H is the height of the wall. Standard Plan reinforced concrete walls, Standard Plan Geosynthetic walls, MSE walls, soil nail walls, soldier pile walls and anchored walls are generally considered to be flexible retaining walls. Non-yielding walls shall use at-rest earth pressure parameters. Non-yielding walls include, for example, integral abutment walls, wall corners, cut and cover tunnel walls, and braced walls (i.e., walls that are cross-braced to another wall or structure). Where bridge wing and curtain walls join the bridge abutment, at rest earth pressures should be used. At distances away from the bridge abutment equal to or greater than the height of the abutment wall, active earth pressures may be used. This assumes that at such distances away from the bridge abutment, the wing or curtain wall can deflect enough to allow active conditions to develop.

If external bracing is used, active pressure may be used for design. For walls used to stabilize landslides, the applied earth pressure acting on the wall shall be estimated from limit equilibrium stability analysis of the slide and wall (external and global stability only). The earth pressure force shall be the force necessary to achieve stability in the slope, which may exceed at-rest or passive pressure.

Regarding the use of passive pressure for wall design and the establishment of its magnitude, the effect of wall deformation and soil creep should be considered, as described in the AASHTO *LRFD Bridge Design Specifications*, Article 3.11.1 and associated commentary. For passive pressure in front of the wall, the potential removal of soil due to scour, erosion, or future excavation in front of the wall shall be considered when estimating passive resistance.

15-4.9 Surcharge Loads

Article 3.11.6 in the AASHTO *LRFD Bridge Design Specifications* shall be used for surcharge loads acting on all retaining walls and abutments for walls in which the ground surface behind the wall is 4H:IV or flatter, the wall shall be designed for the possible presence of construction equipment loads immediately behind the wall. These construction loads shall be taken into account by applying a 250 psf live load surcharge to the ground surface immediately behind the wall. Since this is a temporary construction load, seismic loads should not be considered for this load case.

15-4.10 Seismic Earth Pressures

For seismic design of walls, the requirements in the AASHTO *LRFD Bridge Design Specifications* shall be met.

For free standing walls that are free to move during seismic loading, if it is desired to use a value of k_h that is less than 50 percent of A_s, such walls may be designed for a reduced seismic acceleration (i.e., yield acceleration) as specifically calculated in the AASHTO *LRFD Bridge Design Specifications*. The reduced (yield) acceleration should be determined using a wall displacement that is less than or equal to the following displacements:

- Structural gravity or semi-gravity walls – maximum horizontal displacement of 4 in.
- MSE walls – maximum horizontal displacement of 8 in.

These maximum allowed displacements do not apply to walls that support other structures, unless it is determined that the supported structures have the ability to tolerate the design displacement without compromising the required performance of the supported structure. These maximum allowed displacements also do not apply to walls that support utilities that cannot tolerate such movements and must function after the design seismic event or that support utilities that could pose a significant danger to the public of the utility ruptured. For walls that do support other structures, the maximum wall horizontal displacement allowed shall be no greater than the displacement that is acceptable for the structure supported by the wall.

These maximum allowed wall displacements also do not apply to non-gravity walls (e.g., soldier pile, anchored walls). A detailed structural analysis of non-gravity walls is required to assess how much they can deform laterally during the design seismic event, so that the appropriate value of k_h can be determined.

If fine grained soils are present behind the wall, the seismic earth pressure shall be determined accounting for the effect of earthquake shaking and displacement on the soil shear strength. For sensitive silts and clays (see also Section 6.4.3), the shear strength used to calculate the seismic earth pressure shall be reduced to account for the strength loss caused by the shaking. If over-consolidated cohesive soils (e.g., “Seattle Clays” as described in Section 5-13.3) are present behind the wall and the wall is designed to allow displacement, the residual drained friction angle rather than the peak friction angle in accordance with Chapter 5, should be used to determine the seismic lateral earth pressure. To justify a design shear strength greater than its residual value, a wall displacement analysis shall be conducted and shall demonstrate that the magnitude of the wall deflections allowed are too small to drop the shear strength to its residual value. See Chapter 5 for additional requirements regarding the shear strength issue, and Chapter 6 and the AASHTO *LRFD Bridge Design Specifications* for design methods and additional requirements to estimate the wall deflection.

Note that for the design methods typically used to estimate seismic earth pressure and which are specified in the GDM the slope of the active failure plane flattens as the earthquake acceleration increases. For anchored walls, the bonded zone of the anchors shall be located behind the active failure wedge. The methodology provided in FHWA Geotechnical Engineering Circular No. 4 (Sabatini et al., 1999) should be used to locate the active failure plane for the purpose of anchored zone location for anchored walls. If the anchors are needed to provide an acceptable level of safety for overall slope stability during seismic loading, the bonded zone of the anchors shall be located behind the critical slope stability failure surface and the active zone behind the wall for seismic loading.

For walls that support other structures that are located over the active zone of the wall, the inertial force due to the mass of the supported structure shall be considered in the design of the wall if that structure can displace laterally with the wall during the seismic event. For supported structures that are only partially supported by the active zone of the wall, numerical modeling of the wall and supported structure should be considered to assess the impact of the supported structure inertial force on the wall stability.

15-4.11 Liquefaction

Under extreme event loading, liquefaction and lateral spreading may occur. The geotechnical designer shall assess liquefaction and lateral spreading for the site and identify these geologic hazards. Design to assess and to mitigate these geologic hazards shall be conducted in accordance with the provisions in Chapter 6.

For walls that retain liquefiable soils, and for which ground improvement is not feasible or cost effective to mitigate the liquefiable soils, the Generalized Limit Equilibrium (GLE) Method should be used to estimate the seismic active earth pressure as specified in the AASHTO *LRFD Bridge Design Manual*, specifically Article 11.6.5.3. Two analyses are required when a wall retains soil layers that may liquefy. These two analyses include: (1) a pseudo-static wall design as specified in Section 15-4.10, and (2) an analysis in which the soil has liquefied. For sites where more than 20 percent of the hazard contributing to the peak ground acceleration is from an earthquake with a magnitude of 7.5 or more (i.e., a long duration earthquake where there is potential for strong motion to occur after liquefaction has occurred), it should be assumed that the additional earth pressure behind the wall due to liquefaction occurs simultaneously with the earthquake ground motion.

In this case, k_h shall be as specified in the previous section (i.e., Section 15-4.10). For earthquakes in which the magnitude is less than 7.5, it can be assumed that $k_h = 0$ when the soil is liquefied.

When using the GLE Method to determine seismic earth pressure when the soil is liquefied, the liquefied shear strength shall be determined as a function of vertical effective stress such as shown in Figures 6-1, 6-3, and 6-4. Furthermore, for soils that liquefy but which have relatively high SPT blowcounts, it is possible that the seismic lateral earth pressure generated could be higher than the earth pressure generated when the soil has not liquefied. In such cases, the earth pressure generated when using liquefied soil shear strength shall be limited to be no less than the non-liquefied earth pressure.

Numerical, two dimensional effective stress methods may also be used to assess the earth pressure on retaining walls due to retained soil that contains liquefiable layers. The geotechnical designer shall provide documentation that their numerical model has been validated and calibrated with field data, centrifuge data, and/or extensive sensitivity analyses. Due to the highly specialized nature of these more sophisticated liquefaction assessment approaches, approval by the State Geotechnical Engineer is required to use nonlinear effective stress methods for liquefaction evaluation, and independent peer review as described in Section 6-3.3 shall be conducted.

15-4.12 Overall Stability

All retaining walls and reinforced slopes shall be designed for overall stability using Strength Limit State load groups, using a load factor of 1.0 for non-structural loads and shall have a resistance factor for overall stability of 0.75 (i.e., a safety factor of 1.3 as calculated using a limit equilibrium slope stability method). This resistance factor is not to be applied directly to the soil properties used to assess this mode of failure. If structural foundation loads are to be applied to the slope being analyzed (e.g., such as a bridge footing or retaining wall), the structural foundation loads shall be factored as a Strength Limit State load, and the resistance factor shall be no greater than 0.75. If Extreme Event loading is a factor (e.g., for earthquake loading), the load and resistance factors specified in the AASHTO *LRFD Bridge Design Specifications* shall be used.

It is important to check overall stability for surfaces that include the wall mass, as well as surfaces that check for stability of the soil below the wall, if the wall is located well above the toe of the slope. If the slope below the wall is determined to be potentially unstable, the wall stability should be evaluated assuming that the unstable slope material has moved away from the toe of the wall, if the slope below the wall is not stabilized. The slope above the wall, if one is present, should also be checked for overall stability.

Stability shall be assessed using limiting equilibrium methods in accordance with Chapter 7.

15-4.13 Wall Drainage

Drainage shall be provided for all walls when it is possible for water to build up behind the wall due to groundwater, stormwater infiltration, flooding, or due to tidal influence. In instances where wall drainage cannot be provided, the hydrostatic pressure from the water shall be included in the design of the wall. In general, wall drainage shall be in accordance with the *Standard Plans*, General Special Provisions. Figure 730-11 in the *Design Manual* M 22-01 shall be used for drain details and drain placement for all walls not covered by Standard Plan D-4 except as follows:

- Gabion walls and rock walls are generally considered permeable and do not typically require wall drains, provided construction geotextile is placed against the native soil or fill.
- Soil nail walls shall use composite drainage material centered between each column of nails. The drainage material shall be connected to weep holes using a drain gate or shall be wrapped around an underdrain.
- Cantilever and Anchored wall systems using lagging shall have composite drainage material attached to the lagging face prior to casting the permanent facing. Walls without facing or walls using precast panels are not required to use composite drainage material provided the water can pass through the lagging unhindered.
- For walls subject to periodic inundation due to tides or frequent flooding, additional drainage features shall be included with the wall to prevent or at least minimize the potential for rapid draw-down conditions, such as additional weep holes, chimney drains, etc., plus rapidly draining backfill as described in Section 15-3.7 below the level of inundation, if wall backfill is needed.

15-4.14 Utilities

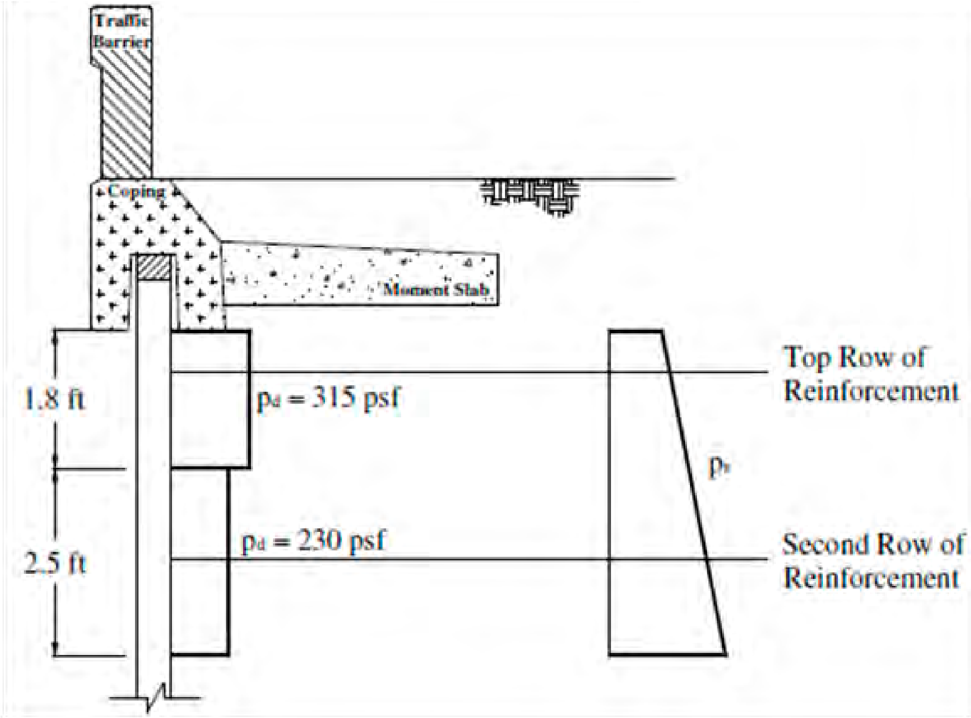
Walls that have or may have future utilities in the backfill should minimize the use of soil reinforcement. MSE, soil nail, and anchored walls commonly have conflicts with utilities and should not be used when utilities must remain in the reinforced soil zone unless there is no other wall option. Utilities that are encapsulated by wall reinforcement may not be accessible for replacement or maintenance. Utility agreements should specifically address future access if wall reinforcing will affect access.

15-4.15 Guardrail and Barrier

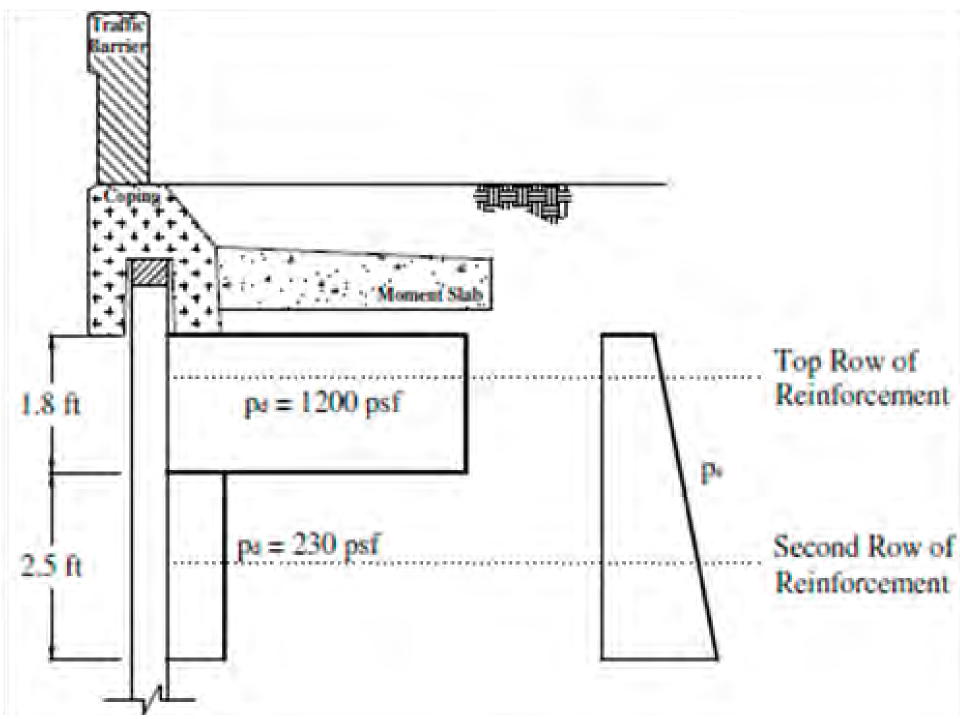
Guardrail and barrier shall meet the requirements of the *Design Manual* M 22-01, *Bridge Design Manual*, *Standard Plans*, and the AASHTO *LRFD Bridge Design Specifications*.
In no case shall guardrail posts be placed through MSE wall or reinforced slope soil reinforcement closer than 3 feet from the back of the wall facing elements. Furthermore, the guard rail posts shall be installed through the soil reinforcement in a manner that prevents ripping and distortion of the soil reinforcement, and the soil reinforcement shall be designed to account for the reduced cross-section resulting from the guardrail post holes.

For walls with a traffic barrier, the distribution of the applied impact load to the wall top shall be as described in the AASHTO *LRFD Bridge Design Specifications* Article 11.10.10.2 for LRFD designs unless otherwise specified in the *Bridge Design Manual*, except that for MSE walls, the impact load should be distributed into the soil reinforcement considering only the top two reinforcement layers below the traffic barrier to take the distributed impact load as described in NCHRP Report 663, Appendix I (Bligh, et al., 2010). See Figure 15-3 for an illustration of soil reinforcement load distributions for TL-3 and TL-4 loading. In that figure, p_d is the dynamic pressure distribution due to the traffic impact load that is to be resisted by the soil reinforcement, and p_s is the static earth pressure distribution, which is to be added to the dynamic pressure to determine the total soil reinforcement loading. For TL-5 loading, the soil reinforcement loads shown in the figure should be scaled up considering the magnitude of the impact load for TL-4 loading relative to the impact load for TL-5 loading.

Figure 15-3 MSE Wall Soil Reinforcement Design for Traffic Barrier Impact for TL-3 and TL-4 Loading (after Bligh, et al., 2010)



(a) Pressure distribution for reinforcement pullout



(b) Pressure distribution for reinforcement rupture.

15-5 Wall Type Specific Design Requirements

15-5.1 Abutments

Abutment foundations shall be designed in accordance with Chapter 8. Abutment walls, wingwalls, and curtain walls shall be designed in accordance with AASHTO *LRFD Bridge Design Specifications* and as specifically required in this GDM. Abutments that are backfilled prior to constructing the superstructure shall be designed using active earth pressures. Active earth pressures shall be used for abutments that are backfilled after construction of the superstructure, if the abutment can move sufficiently to develop active pressures. If the abutment is restrained, at-rest earth pressure shall be used. Abutments that are "U" shaped or that have curtain/wing walls should be designed to resist at-rest pressures in the corners, as the walls are constrained (see Section 15-4.8).

15-5.2 Nongravity Cantilever and Anchored Walls

WSDOT typically does not utilize sheet pile walls for permanent applications, except at Washington State Ferries (WSF) facilities. Sheet pile walls may be used at WSF facilities but shall not be used elsewhere without approval of the WSDOT Bridge Design Engineer. Sheet pile walls utilized for shoring or cofferdams shall be the responsibility of the Contractor and shall be approved on construction, unless the construction contract special provisions or plans state otherwise.

Permanent soldier piles for soldier pile and anchored walls should be installed in drilled holes. Impact or vibratory methods may be used to install temporary soldier piles, but installation in drilled holes is preferred.

Nongravity and Anchored walls shall be designed using the latest edition of the AASHTO *LRFD Bridge Design Specifications*. Key geotechnical design requirements for these types of walls are found in Sections 3 and 11 of the AASHTO LRFD specifications.

15-5.2.1 *Nongravity Cantilever Walls*

The exposed height of nongravity cantilever walls is generally controlled by acceptable deflections at the top of wall. In "good" soils, cantilever walls are generally 12 to 15 feet or less in height. Greater exposed heights can be achieved with increased section modulus or the use of secant/tangent piles. Nongravity cantilever walls using a single row of ground anchors or deadmen anchors shall be considered an anchored wall.

In general, the drilled hole for the soldier piles for nongravity cantilever walls will be filled with a relatively low strength flowable material such as controlled density fill (CDF), provided that water is not present in the drilled hole. Since CDF has a relatively low cement content, the cementitious material in the CDF has a tendency to wash out when placed through water. If the CDF becomes too weak because of this, the design assumption that the full width of the drilled hole, rather than the width of the soldier pile by itself, governs the development of the passive resistance in front of the wall will become invalid. The presence of groundwater will affect the choice of material specified by the structural designer to backfill the soldier pile holes, e.g., CDF if the hole is not wet, or higher strength concrete designed for tremie applications. Therefore, it is important that the geotechnical designer identify the potential for ground water in the drilled holes during design, as the geotechnical stability of a nongravity cantilever soldier pile wall is governed by the passive resistance available in front of the wall.

Typically, when discrete vertical elements are used to form the wall, it is assumed that due to soil arching, the passive resistance in front of the wall acts over three pile/shaft diameters. For typical site conditions, this assumption is reasonable. However, in very soft soils, that degree of soil arching may not occur, and a smaller number of pile diameters (e.g., 1 to 2 diameters) should be assumed for this passive resistance arching effect. For soldier piles placed in very dense soils, such as glacially consolidated till, when CDF is used, the strength of the CDF may be similar enough to the soil that the full shaft diameter may not be effective in mobilizing passive resistance. In that case, either full strength concrete should be used to fill the drilled hole, or only the width of the soldier pile should be considered effective in mobilizing passive resistance.

If the wall is being used to stabilize a deep seated landslide, in general, it should be assumed that full strength concrete will be used to backfill the soldier pile holes, as the shearing resistance of the concrete will be used to help resist the lateral forces caused by the landslide.

15-5.2.2 Anchored/Braced Walls

Anchored/braced walls generally consist of a vertical structural elements such as soldier piles or drilled shafts and lateral anchorage elements placed beside or through the vertical structural elements. Design of these walls shall be in accordance with the AASHTO *LRFD Bridge Design Specifications*.

In general, the drilled hole for the soldier piles for anchored/braced walls will be filled with a relatively low strength flowable material such as controlled density fill (CDF). For anchored walls, the passive resistance in front of the wall toe is not as critical for wall stability as is the case for nongravity cantilever walls. For anchored walls, resistance at the wall toe to prevent "kickout" is primarily a function of the structural bending resistance of the soldier pile itself. Therefore, it is not as critical that the CDF maintain its full shear strength during and after placement if the hole is wet. For anchored/braced walls, the only time full strength concrete would be used to fill the soldier pile holes in the buried portion of the wall is when the anchors are steeply dipping, resulting in relatively high vertical loads, or for the case when additional shear strength is needed to resist high lateral kickout loads resulting from deep seated landslides. In the case of walls used to stabilize deep seated landslides, the geotechnical designer must clearly indicate to the structural designer whether or not the shear resistance of the soldier pile and cementitious backfill material (i.e., full strength concrete) must be considered as part of the resistance needed to help stabilize the landslide.

15-5.2.3 Permanent Ground Anchors

The geotechnical designer shall define the no-load zone for anchors in accordance with the AASHTO *LRFD Bridge Design Specifications*. If the ground anchors are installed through landslide material or material that could potentially be unstable, the no load zone shall include the entire unstable zone as defined by the actual or potential failure surface plus 5 feet minimum. The contract documents should require the drill hole in the no load zone to be backfilled with a non-structural filler. Contractors may request to fill the drill hole in the no load zone with grout prior to testing and acceptance of the anchor. This is usually acceptable provided bond breakers are present on the strands, the anchor unbonded length is increased by 8 feet minimum, and the grout in the unbonded zone is not placed by pressure grouting methods.

The geotechnical designer shall determine the factored anchor pullout resistance that can be reasonably used in the structural design given the soil conditions. The ground anchors used on the projects shall be designed by the Contractor. Compression anchors (see Sabatini, et al., 1999) may be used, but conventional anchors are preferred by WSDOT.

The geotechnical designer shall estimate the nominal anchor bond stress (t_n) for the soil conditions and common anchor grouting methods. AASHTO *LRFD Bridge Design Specifications* and the FHWA publications listed at the beginning of this chapter provide guidance on acceptable values to use for various types of soil and rock. The geotechnical designer shall then apply a resistance factor to the nominal bond stress to determine a feasible factored pullout resistance (FPR) for anchors to be used in the wall. In general, a 5-in diameter low pressure grouted anchor with a bond length of 15 to 30 feet should be assumed when estimating the feasible anchor resistance. FHWA research has indicated that anchor bond lengths greater than 40 feet are not fully effective. Anchor bond lengths greater than 50 feet shall be approved by the State Geotechnical Engineer.

The structural designer shall use the factored pullout resistance to determine the number of anchors required to resist the factored loads. The structural designer shall also use this value in the contract documents as the required anchor resistance that Contractor needs to achieve. The Contractor will design the anchor bond zone to provide the specified resistance. The Contractor will be responsible for determining the actual length of the bond zone, hole diameter, drilling methods, and grouting method used for the anchors.

All ground anchors shall be proof tested, except for anchors that are subjected to performance tests. A minimum of 5 percent of the wall's anchors shall be performance tested. For ground anchors in clays, or other soils that are known to be potentially problematic, especially with regard to creep, at least one verification test shall be performed in each soil type within the anchor zone. Past WSDOT practice has been to perform verification tests at two times the design load with proof and performance tests loaded to 1.5 times the design load. National practice has been to test to 1.33 times the design load for proof and performance tests. Historically, WSDOT has utilized a higher safety factor in its anchored wall designs (FS=1.5) principally due to past performance with anchors constructed in Seattle Clay. For anchors that are installed in Seattle Clay, other similar formations, and clays in general, the level of safety obtained in past WSDOT practice shall continue to be used (i.e., FS = 1.5). Detailed testing and acceptance protocols, based on recommendations by Allen (2020), that shall be followed for tiebacks installed in clays are provided in Appendix 15-G. The recommended protocols for tiebacks in clay provided in Allen (2020) and in Appendix 15-G were primarily developed for straight-shafted, low pressure grouted tiebacks. Application of these criteria to pressure and post- grouted tiebacks may be considered, subject to approval by the State Geotechnical Engineer. For anchors in other soils (e.g., sands, gravels, glacial tills), the level of safety obtained when applying the national practice (i.e., FS = 1.33) should be used.

The AASHTO *LRFD Bridge Design Specifications* specifically addresses anchor testing. The AASHTO specifications recommend that the test loads used in past allowable stress design practice be reduced by the load factor applicable to the limit state that controls the maximum factored design load for the anchor. For the strength limit state, a load factor γ_{EH} of 1.35 is typically applied to the lateral earth pressure acting on the wall. If the seismic design (i.e., Extreme Event I) controls the factored load acting on the anchor, then the load factor is only 1.0. However, due to the extreme nature of the loading for this limit state, the extra margin of safety used to design in the strength limit state is not needed for the seismic load case, as past allowable stress design practice used a FS of 1.0.

To be consistent with previous WSDOT practice, for the Strength Limit State, verification tests, if conducted, shall be performed to 1.5 times the factored design load (FDL) for the anchor. Proof and performance tests shall be performed to 1.15 times the factored design load (FDL) for anchors installed in clays, and to 1.00 times the factored design load (FDL) for anchors in other soils and rock. The geotechnical designer should make the decision during design as to whether or not a higher test load is required for anchors in a portion of, or all of, the wall due to the presence of clays or other problematic soils. These proof, performance, and verification test loads assume that a load factor, γ_{EH}, of 1.35 is applied to the apparent earth pressure used to design the anchored wall. If the Extreme Event I limit state controls the design, the same loading sequence and magnitude as used for the strength limit state should be used for all anchor tests.

The following shall be used for verification tests:

Strength Limit State Controls	
Load	**Hold Time**
AL	1 Min.
0.25FDL	10 Min.
0.50FDL	10 Min.
0.75FDL	10 Min.
1.00FDL	10 Min.
1.15FDL	**60 Min.**
1.25FDL	10 Min.
1.50FDL	10 Min.
AL	1 Min.

AL is the alignment load. The test load shall be applied in increments of 25 percent of the factored design load. Each load increment shall be held for at least 10 minutes. Measurement of anchor movement shall be obtained at each load increment. The load-hold period shall start as soon as the test load is applied and the anchor movement, with respect to a fixed reference, shall be measured and recorded at 1 minute, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 45, and 60 minutes.

The following shall be used for proof tests, for anchors in clay or other creep susceptible or otherwise problematic soils or rock:

Strength Limit State Controls	
Load	**Hold Time**
AL	1 Min.
0.25FDL	1 Min.
0.50FDL	1 Min.
0.75FDL	1 Min.
1.00FDL	1 Min.
1.15FDL	**10 Min.**
AL	1 Min.

The following shall be used for proof tests, for anchors in sands, gravels, glacial tills, rock, or other materials where creep is not likely to be a significant issue:

Strength Limit State Controls	
Load	**Hold Time**
AL	1 Min.
0.25FDL	1 Min.
0.50FDL	1 Min.
0.75FDL	1 Min.
1.00FDL	**10 Min.**
AL	1 Min.

The maximum test load in a proof test shall be held for ten minutes, and shall be measured and recorded at 1 minute, 2, 3, 4, 5, 6, and 10 minutes. If the anchor movement between one minute and ten minutes exceeds 0.04 in, the maximum test load shall be held for an additional 50 minutes. If the load hold is extended, the anchor movements shall be recorded at 15, 20, 25, 30, 45, and 60 minutes.

Performance tests cycle the load applied to the anchor. Between load cycles, the anchor is returned to the alignment load (AL) before beginning the next load cycle. The following shall be used for performance tests:

Cycle 1	Cycle 2	Cycle 3	Cycle 4	Cycle 5*	Cycle 6
AL	AL	AL	AL	AL	AL
0.25FDL	0.25FDL	0.25FDL	0.25FDL	0.25FDL	Lock-off
	0.50FDL	0.50FDL	0.50FDL	0.50FDL	
		0.75FDL	0.75FDL	0.75FDL	
			1.00FDL	1.00FDL	
				1.15FDL	

*The fifth cycle shall be conducted if the anchor is installed in clay or other problematic soils. Otherwise, the load hold is conducted at 1.00FDL and the fifth cycle is eliminated.

The load shall be raised from one increment to another immediately after a deflection reading. The maximum test load in a performance test shall be held for 10 minutes. If the anchor movement between one minute and 10 minutes exceeds 0.04 inch, the maximum test load shall be held for an additional 50 minutes. If the load hold is extended, the anchor movements shall be recorded at 15, 20, 25, 30, 45, and 60 minutes. After the final load hold, the anchor shall be unstressed to the alignment load then jacked to the lock-off load.

The structural designer should specify the lock-off load in the contract. Past WSDOT practice has been to lock-off at 80 percent of the anchor design load. Because the factored design load for the anchor is higher than the “design load” used in past practice, locking off at 80 percent would result in higher tendon loads. To match previous practice, the lock-off load for all permanent ground anchors shall be 60 percent of the factored design load for the anchor. This applies to both the Strength and Extreme Event limit states.

Since the contractor designs and installs the anchor, the contract documents should require the following:

1. Lock off shall not exceed 70 percent of the specified minimum tensile strength for the anchor.
2. Test loads shall not exceed 80 percent of the specified minimum tensile strength for the anchor.
3. All anchors shall be double corrosion protected (encapsulated). Epoxy coated or bare strands shall not be used unless the wall is temporary.
4. Ground anchor installation angle should be 15 to 30 degrees from horizontal, but may be as steep as 45 degrees to install anchors in competent materials or below failure planes.

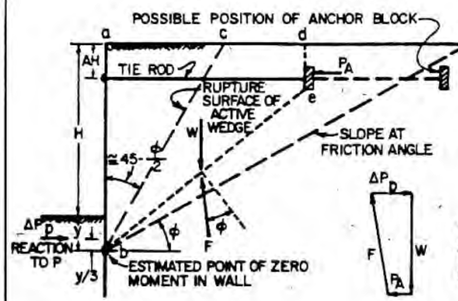
The geotechnical designer and the structural designer should develop the construction plans and special provisions to ensure that the contractor complies with these requirements.

15-5.2.4 *Deadmen*

The geotechnical designer shall develop earth pressures and passive resistance for deadmen in accordance with AASHTO *LRFD Bridge Design Specifications*. Deadmen shall be located in accordance with Figure 20 from NAVFAC DM-7.2, Foundations and Earth Structures, May 1982 (reproduced below for convenience in Figure 15-4).

Figure 15-4 Deadman Anchor Design (After NAVFAC, 1982)

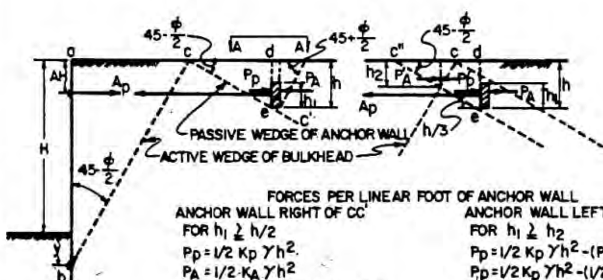
EFFECT OF ANCHOR LOCATION RELATIVE TO THE WALL



ANCHOR BLOCK LEFT OF bc PROVIDES NO RESISTANCE. ANCHOR BLOCK RIGHT OF bf PROVIDES FULL RESISTANCE WITH NO LOAD TRANSFERRED TO WALL. ANCHOR BLOCK BETWEEN bc AND bf PROVIDES PARTIAL RESISTANCE AND TRANSFERS LOAD ΔP_p TO BASE OF WALL.

VECTOR DIAGRAM FOR FREE BODY abed WHERE P_A = ACTIVE FORCE ON BACK OF de AT ANCHOR BLOCK.

CONTINUOUS ANCHOR WALL LOCATED BETWEEN RUPTURE SURFACE AND SLOPE AT FRICTION ANGLE



FORCES PER LINEAR FOOT OF ANCHOR WALL

ANCHOR WALL RIGHT OF CC'

FOR $h_1 \geq h/2$

$P_p = 1/2\, K_p \gamma h^2$.

$P_A = 1/2\, K_A \gamma h^2$

K_p OBTAINED FROM FIGURE 5 USING $-\delta/\phi = 0.5$

ANCHOR WALL LEFT OF CC'

FOR $h_1 \geq h_2$

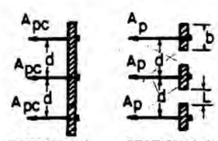
$P_p = 1/2\, K_p \gamma h^2 - (P_p' - P_A')$

$P_p = 1/2\, K_p \gamma h^2 - (1/2\, K_p \gamma h_2^2 - 1/2\, K_A \gamma h_2^2)$

$P_A = 1/2\, K_A \gamma h^2$

K_A IS OBTAINED FROM FIGURE 3

EFFECT OF DEPTH AND SPACING OF ANCHOR BLOCKS



SECTION A-A CONTINUOUS WALL

SECTION A-A INDIVIDUAL ANCHORS

ANCHOR RESISTANCE FOR $h_1 \geq \frac{h}{2}$

1. CONTINUOUS WALL:
   ULTIMATE $A_{pc}/d = P_p - P_A$, WHERE A_{pc}/d IS ANCHOR RESISTANCE AND P_p, P_A TAKEN PER LINEAL FOOT OF WALL.
2. INDIVIDUAL ANCHORS:
   IF $d > b+h$, ULTIMATE $A_p = b(P_p - P_A) + 2P_o \tan\phi$, WHERE P_o = RESULTANT FORCE OF SOIL AT REST ON VERTICAL AREA cde OR c"de.
   IF $d = h+b$, A_p/d IS 70% OF A_{pc}/d FOR CONTINUOUS WALL. L FOR THIS CONDITION IS L' AND $L' = h$.
   IF $d < h+b$, $A_p/d = A_{pc}/d - \frac{L}{L'}(.3\, A_{pc}/d)$, $L' = h$.

ANCHOR RESISTANCE FOR $h_1 < \frac{h}{2}$

ULTIMATE A_p/d OR A_{pc}/d EQUALS BEARING CAPACITY OF STRIP FOOTING OF WIDTH h_1 AND SURCHARGE LOAD $\gamma(h - \frac{h_1}{2})$, SEE FIGURE 1, CHAPTER 4

USE FRICTION ANGLE ϕ': WHERE $\tan\phi' = 0.6 \tan\phi$.

GENERAL REQUIREMENTS:

1. ALLOWABLE VALUE OF A_p AND A_{pc} = ULTIMATE VALUE/2, FACTOR OF SAFETY OF 2 AGAINST FAILURE.
2. VALUES OF K_A AND K_p ARE FOR COHESIONLESS MATERIALS. IF BACKFILL HAS BOTH ϕ AND C STRENGTHS, COMPUTE ACTIVE AND PASSIVE FORCES ACCORDING TO FIGURES 7 AND 9 FINE GRAINED SOILS OF MEDIUM TO HIGH PLASTICITY SHOULD NOT BE USED AT THE ANCHORAGE.
3. SOILS WITHIN PASSIVE WEDGE OF ANCHORAGE SHALL BE COMPACTED TO NO LESS THAN 90% OF MAX. UNIT WEIGHT (ASTM D698 TEST).
4. TIE ROD IS DESIGNED FOR ALLOWABLE A_p OR A_{pc}. TIE ROD CONNECTIONS TO WALL AND ANCHORAGE ARE DESIGNED FOR 1.2 (ALLOWABLE A_p OR A_{pc}).
5. TIE ROD CONNECTION TO ANCHORAGE IS MADE AT THE LOCATION OF THE RESULTANT EARTH PRESSURES ACTING ON THE VERTICAL FACE OF THE ANCHORAGE.

15-5.3 Mechanically Stabilized Earth Walls

Wall design shall be in accordance with the AASHTO *LRFD Bridge Design Specifications*, except as noted below.

With regard to internal stability design of MSE walls, three methods for estimating the design soil reinforcement loads (T_{max}) are available. They include the Simplified Method, the Coherent Gravity Method, and the Simplified Stiffness Method (hereinafter referred to as the Stiffness Method). The Simplified and Coherent Gravity methods have been in use for many years and are currently included in the AASHTO *LRFD Bridge Design Specifications*. The Stiffness Method, developed by Allen and Bathurst (2015, 2018), is newer than the other two methods. While each method started from different "theoretical" assumptions, all three methods have been empirically developed from measurements made during wall operational conditions. It is therefore important that these methods be applied to design situations that are within the range of the case history data used to develop them. For insights as to the range of the design situations applicable to the Coherent Gravity Method, see Schlosser (1978), Schlosser and Segrestin (1979), and Allen et al. (2001). Likewise, for the Simplified Method, see Allen et al. (2001). Finally, for the Stiffness Method, see Allen and Bathurst (2015, 2018). If any of these methods must be used for situations that are significantly beyond their empirical basis (e.g., for walls placed on soft compressible soil), additional evaluations should be conducted. Of the three methods, the Stiffness Method has the broadest empirical basis. However, the Stiffness Method has not been as widely used yet relative to the other two methods for new wall designs, especially for steel reinforced structures.

The Stiffness Method is in general less conservative, but more accurate, than the other two methods. For this reason, the load and resistance factors provided in the current AASHTO *LRFD Bridge Design Specifications* (2017), which are based on levels of safety used in previous long-term design practice, are not directly applicable to the Stiffness Method, requiring that the Stiffness Method be calibrated using reliability theory to achieve the target minimum reliability (see Allen et al. 2005). Therefore, the calibrated load and resistance factors provided in Section 15-5.3.10.2 for the Stiffness Method shall be used.

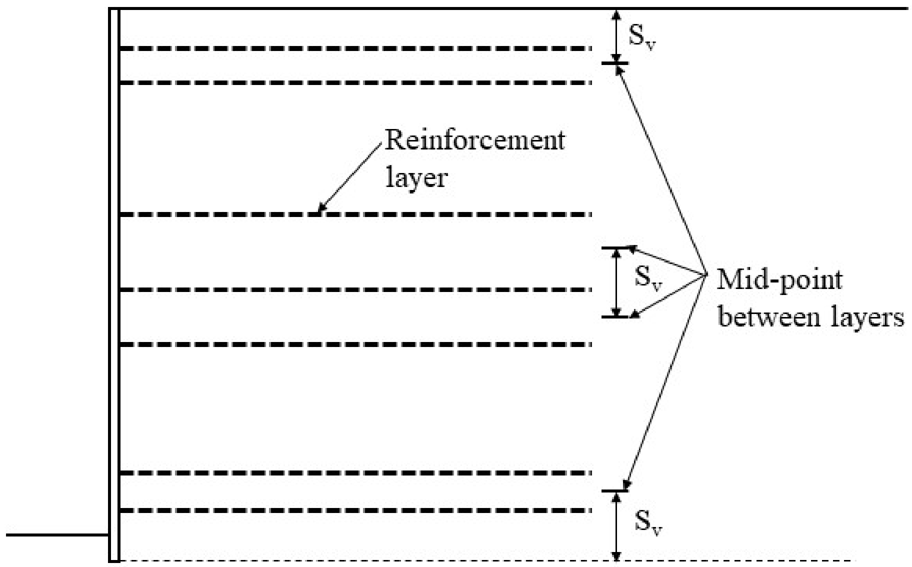
Note that load and resistance factors are not provided for the Stiffness Method in Section 15-5.3.10.2 for MSE walls with steel (i.e., inextensible) reinforcement. Calibration of the Stiffness Method load and resistance factors for steel reinforced systems are still in progress and therefore are not available at the time of this update. Until that calibration work is complete, the Stiffness Method is only approved for routine use for MSE walls with extensible reinforcement. This method may be used for steel reinforced MSE walls only if the reinforcement layers are instrumented such that the reinforcement loads are measured, subject to approval by the State Geotechnical Engineer. However, the Coherent Gravity and Simplified methods, using the load and resistance factors provided in the AASHTO *LRFD Bridge Design Manual*, should be used for inextensible steel reinforced MSE walls, considering long-term successful design practice.

These MSE wall design procedures assume that inextensible reinforcements are not mixed with extensible reinforcements within the same wall. Therefore, MSE walls shall not contain a mixture of inextensible and extensible reinforcements.

15-5.3.1 *Soil Reinforcement Spacing Considerations*

For uniform vertical spacing of soil reinforcement, S_v, the tributary layer thickness, is equal to the vertical spacing of the reinforcement. For nonuniform vertical spacing of soil reinforcement, S_v shall be taken as shown in Figure 15-5.

Figure 15-5 Determination of the tributary layer thickness, S_v



The design procedures provided herein assume that the wall facing combined with the reinforced backfill acts as a coherent unit to form a gravity retaining structure. The effect of relatively large vertical spacing of reinforcement on this assumption is not well known and a vertical spacing greater than 2.7 feet should not be used without full scale wall data (e.g., reinforcement loads and strains, and overall deflections) that support the acceptability of larger vertical spacing. However, for MSE wall systems with facing units equal to or greater than 2.7 ft high with a minimum facing unit width, W_u, equal to or greater than the facing unit height, the maximum spacing, S_v, shall not exceed the width of the facing unit, W_u, or 3.3 ft, whichever is less. See Allen and Bathurst (2003, 2018) for results from and analysis of case history data regarding this issue. It is also important to recognize that large vertical spacing of reinforcement can result in excessive facing deflection, both localized and global, which could in turn cause localized elevated stresses in the facing and its connection to the soil reinforcement, especially for walls with flexible facing. Center-to-center horizontal spacing of reinforcement elements should not exceed 3.3 ft for walls with rigid facing panels. For walls with flexible facing panels, horizontal gaps between soil reinforcement elements should not exceed 1.5 ft.

Horizontal spacings as large as 3.3 ft have been used in typical design and construction practice for MSE walls. Back-analysis of instrumented MSE walls indicates that reinforcement load prediction accuracy is not adversely compromised with horizontal spacing of this magnitude when the reinforcement elements are directly attached to rigid facings such as precast concrete panels. However, for flexible facings such as welded wire, large horizontal spacing of the reinforcement has be shown to cause poor wall performance and therefore should not be used for walls with flexible facing. For flexibly faced walls, even a gap of 1.5 ft between reinforcement elements can result in excessive deformation of the facing elements. Therefore, if horizontal gaps of this magnitude are used, the effect of the gaps on the facing panel deformation should be investigated.

15-5.3.2 Live Load Considerations for MSE Walls

The AASHTO design specifications allow traffic live load to not be specifically considered for pullout design (note that this does not apply to traffic barrier impact load design as discussed above). The concept behind this is that for the most common situations, it is unlikely that the traffic wheel paths will be wholly contained within the active zone of the wall, meaning that one of the wheel paths will be over the reinforcement resistant zone while the other wheel path is over the active zone. However, there are cases where traffic live load could be wholly contained within the active zone.

Therefore, include live load in calculation of T_{max}, where T_{max} is as defined in the AASHTO *LRFD Bridge Design Specifications* (i.e., the calculated maximum load in each reinforcement layer), for pullout design if it is possible for both wheels of a vehicle to drive over the wall active zone at the same time, or if a special live loading condition is likely (e.g., a very heavy vehicle could load up the active zone without having a wheel directly over the reinforcement in the resistant zone). Otherwise, live load does not need to be considered. For example, with a minimum 2 feet shoulder and a minimum vehicle width of 8 feet, the active zone for steel reinforced walls would be wide enough for this to happen only if the wall is over 30 feet high, and for geosynthetic walls over 22 feet high. For walls of greater height, live load would need to be considered for pullout for the typical traffic loading situation.

15-5.3.3 Backfill Considerations for MSE Walls

For steel reinforced MSE walls, the design soil friction angle for the backfill shall not be greater than 40° even if soil specific shear strength testing is conducted, as research conducted to date indicates that measured reinforcement loads do not continue to decrease as the soil shear strength increases (Bathurst, et al., 2009, Allen and Bathurst 2015 and 2018). For geosynthetic MSE walls, however, the load in the soil reinforcement does appear to be correlated to soil shear strength even for shear strength values greater than 40° (see Allen, et al., 2003 and Bathurst, et al., 2008). A maximum design friction angle of 40° should also be used for geosynthetic reinforced walls even with backfill specific shear strength testing, unless project specific approval is obtained from the WSDOT State Geotechnical Engineer to exceed 40°. If backfill shear strength testing is conducted, it shall be conducted in accordance with Section 15-3.7.

In general, low silt content backfill materials such as Gravel Borrow per the WSDOT *Standard Specifications* should be used for MSE walls. If higher silt content soils are used as wall backfill, the wall should be designed using only the frictional component of the backfill soil shear strength as discussed in Section 15-3.7. Other issues that shall be addressed if higher fines content soils are used are as follows:

- **Ability to place and compact the soil, especially during or after inclement weather** – In general, as the fines content increases and the soil becomes more well graded, water that gets into the wall backfill due to rain, surface water flow, or ground water flow can cause the backfill to "pump" during placement and compaction, preventing the wall backfill from being properly compacted. Even some gravel borrow gradations may be susceptible to pumping problems when wet, especially when the fines content is greater than 5 percent. Excessive wall face deformation during wall construction can also occur in this case. Because of this potential problem, higher silt content wall backfill should only be used during extended periods of dry weather, such as typically occurs in the summer and early fall months in Western Washington, and possibly most of the year in at least some parts of Eastern Washington.

- **For steel reinforced wall systems, the effect of the higher fines content on corrosion rate of the steel reinforcement** – General practice nationally is that use of backfill with up to 15 percent silt content is acceptable for steel reinforced systems (AASHTO, 2010; Berg, et al., 2009). If higher silt content soils are used, elevated corrosion rates for the steel reinforcement should be considered (see Elias, et al., 2009).
- **Prevention of water or moisture build-up in the wall reinforced backfill** – When the fines content is greater than 5 percent, the material should not be considered to be free draining (see Section 15-3.7). In such cases where the fines content is greater than that allowed in the WSDOT gravel borrow specification (i.e., greater than 7 percent), special measures to prevent water from entering the reinforced backfill shall be implemented. This includes placement of under-drains at the back of the reinforced soil zone, sheet drains to intercept possible ground and rainwater infiltration flow, and use of some type impermeable barrier over the top of the reinforced soil zone.
- **Potential for long-term lateral and vertical deformation of the wall due to soil creep, or in general as cohesive soil shear strength is lost over the life of the wall** – Strain and load increase with time in a steel reinforced soil wall was observed for a large wall in California, a likely consequence of using a backfill soil with a significant cohesion component (Allen, et al., 2001). The Stiffness Method (see Section 15-5.3.10.1, especially Table 15-E-2 in Appendix 15-E) may be used to estimate the reinforcement strain increase caused by loss of cohesive shear strength over time (i.e., estimate the reinforcement strain using the c-ϕ shear strength at end of construction, and subtract that from the reinforcement strain estimated using only the frictional component of that shear strength for design to get the long-term strain). This would give an indication of the long-term wall deformation that could occur.

15-5.3.4 *Compound Stability Assessment for MSE Walls*

If the MSE wall is located over a soft foundation soil, sloping ground above or below the wall, on or adjacent to unstable ground due to landslides, the wall is a combination of two or more tiers, or the wall supports foundation loads, compound stability of the wall shall be evaluated for the Strength Limit State and as applicable the Extreme Event Limit State in accordance with Section 15-4.12. It is recommended that this stability evaluation only be used to evaluate surfaces that intersect within the bottom 20 to 30 percent of the reinforcement layers. As discussed by Allen and Bathurst (2002) and Allen and Bathurst (2018), available limit equilibrium approaches such as the ones typically used to evaluate slope stability do not work well for internal stability of reinforced soil structures, resulting in excessively conservative designs, at least for geosynthetic or otherwise extensible reinforced systems, and resulting in unconservative designs for steel or otherwise inextensible reinforced systems.

Limit equilibrium analyses (LEA) shall be used to evaluate compound stability. The long-term strength of each backfill reinforcement layer intersected by the failure surface should be considered as resisting forces in the limit equilibrium slope stability analysis.

To perform a LEA for compound stability, three analysis steps are conducted, which are as follows:

- Estimate the nominal load in each reinforcement layer, T_{max}, targeting a load and resistance factor combination of 1.0.
- Adjust the reinforcement spacing and strength required to meet the limit states as specified in sections 15-5.3.10.3.2 and 15-5.3.10.3.3 for each reinforcement layer using factored load and resistance values. Load factors shall be as specified in the AASHTO *LRFD Bridge Design Specifications*, Table 3.4.1-1 and 3.4.1-2, and resistance factors as specified in AASHTO *LRFD Bridge Design Specifications* Table 11.5.7-1, except for the Stiffness Method, in which the load and resistance factors are as specified in GDM Section 15-5.3.10.1, Table 15-5.
- Check the factored design using LEA with factored load and resistance values.

When additional surcharge loads, such as a structure footing load or live load, are applied to the top of the reinforced zone of the MSE wall, for Step 3, they shall be factored as specified in the AASHTO *LRFD Bridge Design Manual*, Article 3.4.1 for the Strength I limit state.

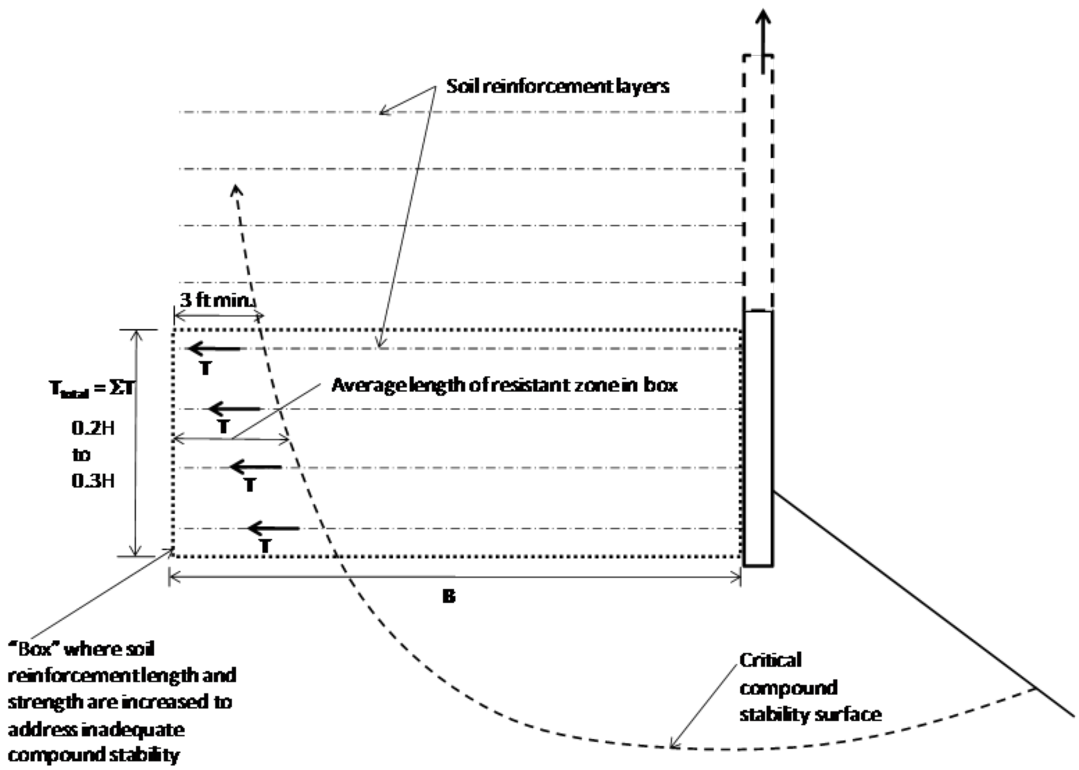
Development of LEA for MSE wall design is summarized in Leshchinsky et al. (2016, 2017). LEA, using either a log spiral or circular failure surface, is described by Vahedifard et al. (2014, 2016) and Leshchinsky et al. (2016, 2017). It is also possible to conduct the LEA using conventional slope stability computer software in which the tensile inclusions provide resistance to slope instability.The results of the compound stability analysis, if it controls the reinforcement needs near the base of the wall, should be expressed as minimum total reinforcement strength and total reinforcement pullout resistance for all layers within a “box” at the base of the wall to meet compound stability requirements. The location of the critical compound stability failure surface in the bottom portion of the wall should also be provided so that the resistant zone boundary location is identified.

Regarding pullout, the length of reinforcement needed behind the critical compound stability failure surface may vary significantly depending on the reinforcement coverage ratio anticipated and the frictional characteristics of the soil reinforcement. Therefore, several scenarios for these two key variables may need to be investigated to assure it is feasible to obtain the desired level of compound stability for all wall/ reinforcement types that are to be considered for the selected width “B” of the box. For convenience, to define the box width “B” required for the pullout length, an average active and resistant zone length should be defined for the box. This concept is illustrated in Figure 15-6. In this figure “H” is the total wall height, “T” is the load required in each reinforcement layer that must be resisted to achieve the desired level of safety in the wall for compound stability (Section 15-4.12 applies for compound stability with regard to the slope stability safety factor needed), and T_{total} is the total force increase needed in the compound stability analysis to achieve the desired level of safety with regard to compound stability. This total force should be less than or equal to the total long-term tensile strength, T_{al}, of the reinforcement layers within the defined “box” and the total pullout resistance available for the reinforcement contained within the box, considering factored loads and resistance values. The engineer needs to select the value of “B” that meets this pullout length requirement. However, the value of “B” selected should be minimized to keep the wall base width required to a minimum, to keep excavation needs as small as possible.

From the wall supplier's view, the contract would specify a specific value of "B" that is long enough such that the desired minimum pullout resistance can be obtained but that provides a consistent basis for bidding purposes with regard to the amount of excavation and shoring needed to build the wall.

Note that for taller walls, it may be desirable to define more than one box at the wall base to improve the accuracy of the pullout length for the intersected reinforcement layers. If the wall is tiered, a box may need to be provided at the base of each tier, depending on the horizontal separation between tiers.

Figure 15-6 Compound Stability Assessment Concept for MSE Wall Design



15-5.3.5 *Design of MSE Walls Placed in Front of Existing Permanent Walls or Rock*

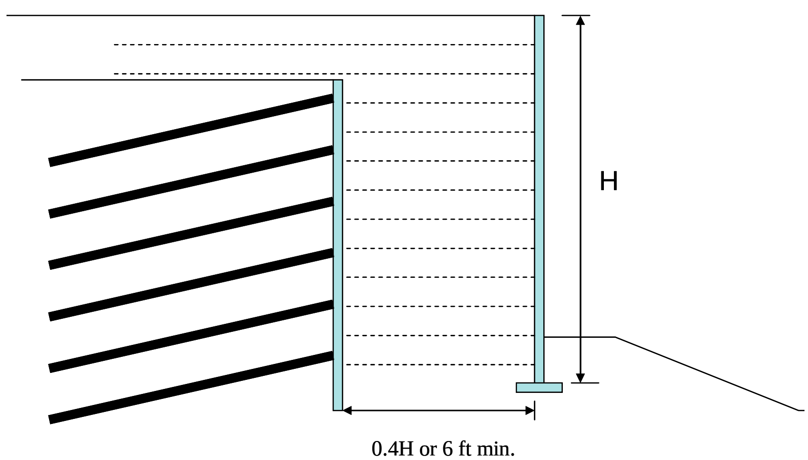
Widening existing facilities sometimes requires MSE walls to be built in front of those existing facilities with inadequate room to obtain the minimum 0.7H wall base width. To reduce excavation costs and shoring costs in side hill situations, the "existing facility" could in fact be a shoring wall or even a near vertical rock slope face. See Figure 15-7 for a conceptual illustration of this situation.

In such cases, assuming that the existing facility is designed as a permanent structure with adequate design life, or if the barrier to adequate reinforcement length is a rock slope, the following design requirements apply:

- The minimum base width is 0.4H or 6 feet, whichever is greater, where H is the total height of the new wall. Note that for soil reinforcement lengths that are less than 8 feet, the weight and size of construction equipment used to place and compact the soil backfill will need to be limited in accordance with the AASHTO *LRFD Bridge Design Specifications* Article C11.10.2.1.

- A minimum of two reinforcement layers, or whatever is necessary for stability, shall extend over the top of the existing structure or steep rock face an adequate distance to insure adequate pullout resistance. The minimum length of these upper two reinforcement layers should be 0.7H, 5 feet behind the face of the existing structure or rock face, or the minimum length required to resist the pullout forces applied to those layers, whichever results in the greatest reinforcement length. Note that to accomplish this, it may be necessary to remove some of the top of the existing structure or rock face if the existing structure is nearly the same height as the new wall. The minimum clearance between the top of the existing structure or rock face and the first reinforcement layer extended beyond the top of the existing structure should be 6 in to prevent stress concentrations.
- The MSE wall reinforcements that are truncated by the presence of the existing structure or rock face shall not be directly connected to that existing face, due to the risk of the development of downdrag forces at that interface and the potential to develop bin pressures and higher reinforcement forces (i.e., T_{max}).
- For internal stability design of MSE walls in this situation, see Morrison, et al. (2006). Global and compound stability, both for static (strength limit state) and seismic loading, shall be evaluated, especially to determine the strength and pullout resistance needed for the upper layers that extend over the top of the existing feature. At least one surface that is located at the face of the existing structure but that goes through the upper reinforcement layers shall be checked for both static and seismic loading conditions. That surface will likely be critical for sizing the upper reinforcement layers.
- For new walls with a height over 30 feet, a lateral deformation analysis should be conducted (e.g., using a properly calibrated numerical model). Approval from the State Geotechnical and Bridge Design Engineers is required in this case.
- This type of MSE wall design should not be used to support high volume mainline transportation facilities if the vertical junction between the existing wall or rock face and the back of the new wall is within the traffic lane, especially if there is potential for cracking in the pavement surface to occur due to differential vertical movement at that location unless approved by the State Geotechnical and State Pavement engineers.

Figure 15-7 Example of Steep Shored MSE Wall



15-5.3.6 *MSE Wall Supported Abutments*

The geotechnical design of MSE wall supported bridge abutments shall be in accordance with the requirements in the following documents, provided in hierarchal order:

1. This *Geotechnical Design Manual*
2. The *Bridge Design Manual* (Section 7.5).
3. AASHTO *LRFD Bridge Design Specifications*.
4. FHWA NHI-10-024 Volume I and NHI-10-025 Volume II, "Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes," (Berg et al., 2009)

See the WSDOT BDM, including Bridge Office Design Policy memoranda, for additional details regarding the design and geometric requirements for SE and geosynthetic wall supported bridge abutments.

The FHWA has developed a manual for a type of MSE wall supported bridge abutment, termed GRS-IBS, provided on the following FHWA website: http://www.fhwa.dot.gov/everydaycounts/technology/grs_ibs/

However, this GDM, and the referenced manuals and design memorandum provided at the beginning of this GDM section, shall be considered to supersede the FHWA GRS-IBS manual with regard to design and material requirements.

For MSE wall bridge abutments, two superstructure foundation support options are available:

- For single or multi-span bridges, subject to approval by the State Geotechnical and State Bridge Design engineer, use of a footing foundation placed directly above the MSE wall reinforced soil zone, or
- For flat slab single span bridges with a span length of up to 60 feet, the end of the flat slab itself bears directly on the surface of the MSE wall reinforced soil zone.

MSE walls directly supporting the bridge superstructure at the abutments shall be 30 feet or less in total height (i.e., height of exposed wall plus embedment depth of wall). Abutment spread footings, or the ends of the superstructure flat slab bearing directly on the surface of the MSE wall, should be designed for service loads not to exceed 3.0 TSF and factored strength limit state footing loads not to exceed 4.5 TSF. Because this is an increase relative to what is specified in the AASHTO *LRFD Bridge Design Specifications*, for bearing service loads greater than 2.0 TSF, a vertical settlement monitoring program with regard to footing or superstructure slab settlement shall be conducted. As a minimum, this settlement monitoring program should consist of monitoring settlement measurement points located at the front edge and back edge of the structure footing, or for slabs place directly on the SME wall top, two settlement measurement points located within the bearing area, and settlement monitoring points directly below the footing or slab bearing area at the base of the wall to measure settlement occurring below the wall. The monitoring program should be continued until movement has been determined to have stopped. If the measured footing settlement exceeds the vertical deformation and angular distortion requirements established for the structure, corrective action shall be taken.

For this MSE wall application, only the following MSE wall/facing types shall be used:

- Two stage geosynthetic wrapped face geosynthetic walls (i.e., similar to the Standard Plan D-3 wall) with cast-in-place (CIP) or precast concrete full height panels, or shotcrete depending on aesthetic needs,
- Single stage dry-cast concrete modular block faced walls using WSDOT preapproved concrete block – geosynthetic reinforcement combinations (see Appendix 15-D), and
- WSDOT preapproved proprietary MSE walls identified as such (see Appendix 15-D), but only those that are concrete faced. Welded wire faced preapproved MSE walls may be used for temporary bridge abutment applications. However, MSE walls identified in Appendix 15-D as preapproved proprietary walls shall not be considered preapproved for the MSE wall supported bridge abutment application (i.e., a special design is required).

Figures 15-8, 15-9, and 15-10 provide typical sections that should be used in the design of MSE wall bridge abutments. The base of the wall may be truncated to reduce excavation needs subject to the limitations provided in Section 11 of the AASHTO *LRFD Bridge Design Specifications*. Figure 15-9 is similar to the Standard Plan geosynthetic wall (Standard Plan D-3), except as modified in this figure for this application. This figure does not show all the details needed for the facing design. For the additional facing details needed, see Standard Plans D-3-10 and D-3-11. The minimum tensile strength of the geotextile or geogrid used as bridge approach soil reinforcement in figures 15-8 and 15-9 shall be 2.4 kips/ft in accordance with ASTM D4595 for geotextiles or ASTM D6637 for geogrids. The soil reinforcement and facing design is project specific and shall be completed in accordance with manuals and design policy documents cited at the beginning of this section.

Figure 15-8 Typical Section for MSE Wall Supported Abutment – Flat Slab Superstructure With no Footing and Dry-Cast Modular Block Wall Facing

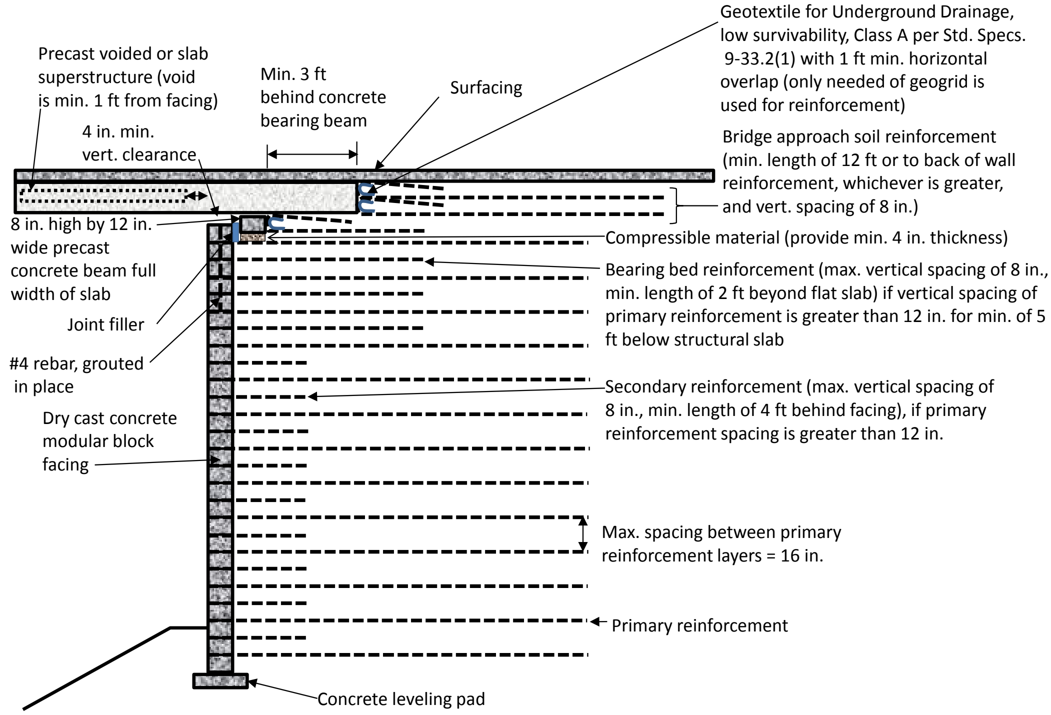


Figure 15-9 Typical Section for MSE Wall Supported Abutment – Flat Slab Superstructure With no Footing and Precast or CIP Concrete Wall Facing (Two Stage Wall Construction)

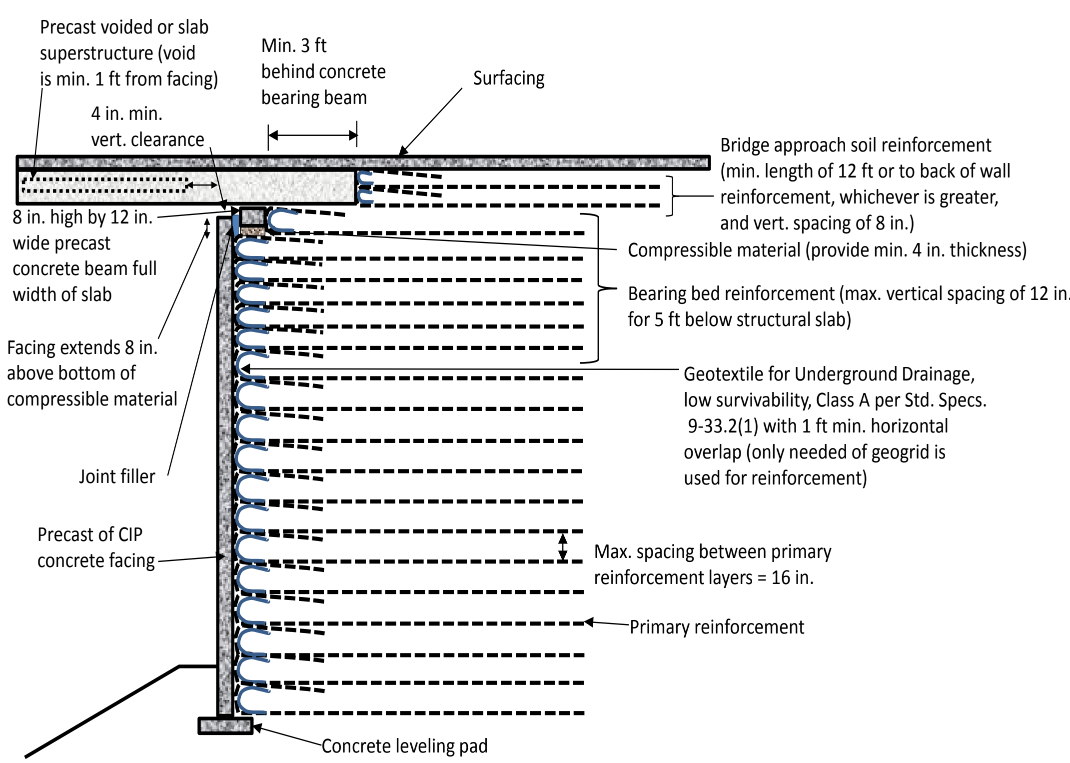
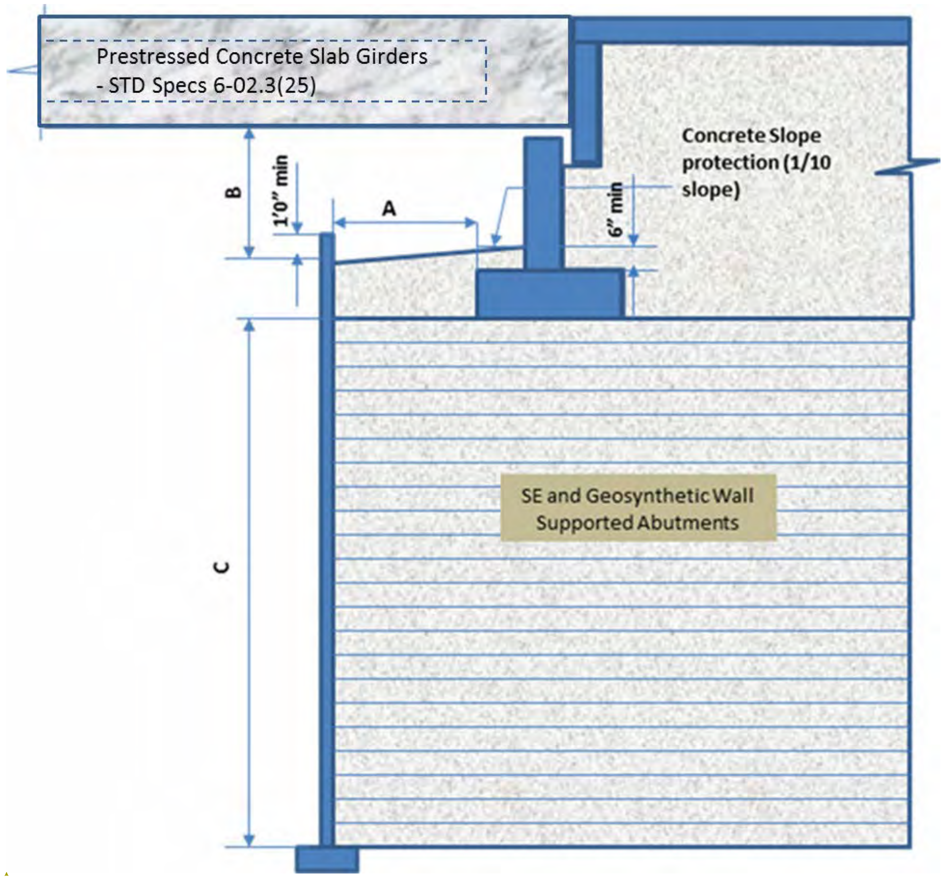


Figure 15-10 Typical Section Showing External Dimensions for Bridge With Spread Footing Supported Directly on an MSE Wall Semi-Integral Abutment (L-Abutment Similar; Wing/Curtain Wall Not Shown)



A = 4 feet min for SE Walls (precast concrete panel face or cast-in-place concrete face), 2 feet min for special designed geosynthetic retaining walls with wrapped face

B = 3 feet min for I-girder bridges, and 5 feet min for non-I-girder, slab, and box girder bridges

C = 30 feet max

For geosynthetic wrapped face two-stage walls with a precast or CIP concrete facing (e.g., similar to a Standard Plan geosynthetic wall) and walls faced with dry cast concrete blocks, a maximum reinforcement vertical spacing of 16 inches shall be used. However, for dry cast concrete block faced walls, secondary reinforcement layers with a minimum length of 4 feet behind the facing shall be placed between the primary reinforcement layers if the primary reinforcement layers are spaced at greater than 12 inches. This will result in a geosynthetic reinforcement layer being placed between every facing block. These spacing limitations apply to the portions of the MSE wall that directly support the bridge foundation (i.e., within the limits of stress increase due to the footing load per the AASHTO *LRFD Bridge Design Specifications*, Article 3.11.6.3). The secondary and bearing bed reinforcement layers, and the bridge approach reinforcement layers (see figures 15-8 and 15-9 for definition of these terms), shall be the same geosynthetic reinforcement product as the primary reinforcement layers directly above and below them. At transitions between primary reinforcement materials (if more than one geosynthetic product is used for the primary reinforcement), the secondary reinforcement materials shall be the stronger of the two primary reinforcement products above and below the secondary or bearing bed reinforcement layer.

For other MSE wall systems that can be used in this application as specified herein, the reinforcement spacing shall be as needed to meet the wall system requirements and the design requirements in the specified design manuals at the beginning of this section.

With regard to Figure 15-10, the minimum horizontal setbacks for the footing on the MSE wall are specified to minimize the potential for shear and excessive vertical deformation of the reinforced backfill too close to the connection of the reinforcement to the facing. The vertical clearance specified between the MSE facing units and the bottom of the superstructure is needed to provide access for bridge inspection. For flat slab single span bridges directly supported by MSE abutments, without a footing and bridge bearings (for span lengths up to 60 feet), these minimum setbacks and clearances do not apply.

The bearing resistance for the footing or flat slab supported by the MSE wall is a function of the soil reinforcement density in addition to the shear strength of the soil. If designing the wall using LRFD, two cases should be evaluated to size the footing for bearing resistance for the strength limit state, as two sets of load factors are applicable (see the AASHTO *LRFD Bridge Design Manual*, Section 3, for definitions of these terms):

- The load factors applicable to the structure loads applied to the footing, such as DC, DW, EH, LL, etc.
- The load factor applicable to the distribution of surcharge loads through the soil, ES.

When ES is used to factor the load applied to the soil to evaluate bearing, the structure loads and live load applied to the footing should be unfactored. When ES is not used to factor the load applied to the soil to evaluate bearing, the structure loads and live load applied to the footing should be factored using DC, DW, EH, LL, etc. The wall should be designed for both cases, and the case that results in the greatest amount of soil reinforcement should be used for the final strength limit state design. See the *Bridge Design Manual* for additional requirements on the application of load groups for design of MSE wall supported abutments, especially regarding how to handle live load, and for the structural detailing required.

The potential lateral and vertical deformation of the wall, considering the affect of the footing load on the wall, should be evaluated. Measures shall be taken to minimize potential deformation of the reinforced soil, such as use of high quality backfill such as Gravel Borrow compacted to 95 percent of maximum density. The settlement and lateral deformation of the soil below the wall shall also be included in this deformation analysis. If there is significant uncertainty in the amount of vertical deformation in or below the wall anticipated, the ability to jack the abutment to accommodate unanticipated abutment settlement should also be considered in the abutment design.

15-5.3.7 Full Height Propped Precast Concrete Panel MSE Walls

This wall system consists of a full height concrete facing panel directly connected to the soil reinforcement elements. The facing panel is braced externally during a significant percentage of the backfill placement. The amount the wall is backfilled before releasing the bracing is somewhat dependent on the specifics of the wall system and the amount of resistance needed to prevent the wall from moving excessively during placement of the remaining fill. Once the external bracing is released, the wall facing allowed to move in response to the release of the bracing.

A key issue regarding the performance of this type of wall is the differential settlement that is likely to occur between the rigid facing panel and the backfill soil as the backfill soil compresses due to the increase in overburden pressure as the fill is placed. Since the facing panel, for practical purposes, can be considered to be essentially rigid, all the downward deformation resulting from the backfill soil compression causes the reinforcing elements to be dragged down with the soil, causing a strain and load increase in the soil reinforcement at its connection with the facing panel. As the wall panel becomes taller, the additional reinforcement force caused by the backfill settlement relative to the facing panel becomes more significant.

WSDOT has successfully built walls of this nature up to 25 feet in height. For greater heights, the uncertainty in the prediction of the reinforcement loads at the facing connection for this type of MSE wall can become large. Specialized design procedures to estimate the magnitude of the excess force induced in the reinforcement at the connection may be needed, requiring approval by the WSDOT State Geotechnical Engineer.

15-5.3.8 Flexible Faced MSE Walls With Vegetation

If a vegetated face is to be used with an MSE wall, the exposed (i.e., above ground wall height shall be limited to 20 feet or less, and the wall face batter shall be no steeper than 1H:6V, unless the facing is battered at 1H:2V or flatter, in which case the maximum height could be extended to 30 feet). A flatter facing batter may be needed depending on the wall system – see appendices to this GDM chapter for specific requirements. For the vegetated facing, if the facing batter is steeper, or if the height is greater than specified here, the compressibility of the facing topsoil could create excessive stresses, settlement, and/or bulging in the facing, any of which could lead to facing stability or deformation problems.

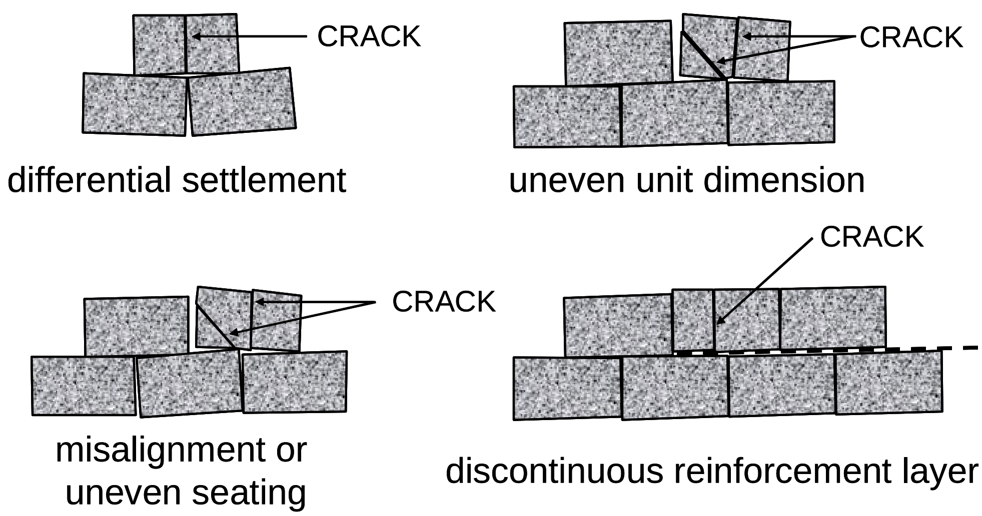
The topsoil placed in the wall face to encourage vegetative growth shall be minimized as much as possible, and should be compacted to minimize internal settlement of the facing. For welded wire facing systems, the effect of the topsoil on the potential corrosion of the steel shall be considered when sizing the steel members at the face and at the connection to the soil reinforcement.

In general, placement of drip irrigation piping within or above the reinforced soil volume to encourage the vegetative growth in the facing should be avoided. However, if a drip irrigation system must be used and placed within or above the reinforced soil volume, the wall shall be designed for the long-term presence of water in the backfill and at the face, regarding both increased design loads and increased degradation/ corrosion of the soil reinforcement, facing materials, and connections.

15-5.3.9 *Dry Cast Concrete Block Faced MSE Walls*

For modular dry cast block faced walls, WSDOT has observed block cracking in near vertical walls below a depth of 25 feet from the wall top in some block faced walls. Key contributing factors include tolerances in the vertical dimension of the blocks that are too great (maximum vertical dimension tolerance should be maintained at +1/16 in or less for walls built as part of WSDOT projects, even though the current ASTM requirements for these types of blocks have been relaxed to +1/8 in), poor block placement technique, soil reinforcement placed between the blocks that creates too much unevenness between the block surfaces, some forms of shimming to make facing batter adjustments, and inconsistencies in the block concrete properties. See Figure 15-11 for illustrations of potential causes of block cracking. Another tall block faced wall problem encountered by others includes shearing of the back portion of the blocks parallel to the wall, possibly face due to excessive buildup of downdrag forces immediately behind the blocks. This problem, if it occurs, has been observed in the bottom 5 to 7 feet of walls that have a hinge height of approximately 25 to 30 feet (total height of 35 feet or more) and may have been caused by excessive downdrag forces due to backfill soil compressibility immediately behind the facing.

Figure 15-11 Example Causes of Cracking in Modular Dry Cast Concrete Block Wall Facings



Considering these potential problems, for modular dry cast concrete block faced walls, the wall height should be limited to 30 feet if near vertical, or to a hinge height of 30 feet if battered. Block wall heights greater than this may be considered on a project specific basis, subject to the approval of the State Geotechnical and State Bridge Design Engineers, if the requirements identified below are met:

- Total settlement is limited to 2 in and differential settlement is limited to 1.5 inch as identified in Table 15-3. Since this is specified in Table 15-3, this also applies to shorter walls.
- A concrete leveling pad is placed below the first lift of blocks to provide a uniform flat surface for the blocks. Note that this should be done for all preapproved block faced walls regardless of height.

- A moderately compressible bearing material is placed between each course of blocks, such as a geosynthetic reinforcement layer. The layer must provide an even bearing surface (many polyester geogrids or multi-filament woven geotextiles provide an adequately even bearing surface with sufficient thickness and compressibility to distribute the bearing load between blocks evenly). The bearing material needs to extend from near the front edge of the blocks (without protruding beyond the face) to at least the back of the blocks or a little beyond. As a minimum, this should be done for all block lifts that are 25 feet or more below the wall top, but doing this for block lifts at depths of less than 25 feet as well is desirable.

If the wall face is tiered such that the front of the facing for the tier above is at least 3 feet behind the back of the facing elements in the tier below, then these height limitations only apply to each tier. The minimum setback between tiers is needed to reduce build-up of excessive down drag forces behind the lower tier wall facing.

Success in building such walls without these block cracking or shear failure problems will depend on the care with which these walls are constructed and the enforcement of good construction practices through proper construction inspection, especially with regard to the constructability issues identified previously. Success will also depend on the quality of the facing blocks. Therefore, making sure that the block properties and dimensional tolerances meet the requirements in the contract through testing and observation is also important and should be carried out for each project.

Modular block facings should not be used where periodic inundation due to tides or flooding can occur, unless a project specific assessment of the amount and frequency of inundation is conducted and approval by the WSDOT State Geotechnical Engineer to use the facing blocks below the inundation zone is obtained. Periodic inundation may affect the durability of dry cast concrete facing blocks and could locally elevate the pH at the connection between the soil reinforcement and the facing as unreacted lime leaches from the facing blocks. Elevated pH can affect the durability of polyester geosynthetics.

15-5.3.10 *Internal Stability Using the Stiffness Method*

The Stiffness Method, as described by Allen and Bathurst (2015, 2018), is provided in the AASHTO *LRFD Bridge Design Specifications* (Sections 3 and 11) to design the internal stability for MSE walls with extensible reinforcement that are not in high settlement areas (i.e., total settlement beneath the wall of more than 6 in.). See Allen and Bathurst (2018) for a definition of “extensible” for soil reinforcement. The AASHTO *LRFD Bridge Design Specifications* are applicable, as well as the traffic barrier design provisions in the WSDOT BDM, except as modified in the provisions that follow.

15-5.3.10.1 *Determination of T_{max} Using the Stiffness Method*

The AASHTO Simplified and Coherent Gravity methods rely on limit equilibrium and/or earth pressure theory concepts for their formulation but modified based on empirical data, whereas, the Stiffness Method, also empirically derived, relies on the difference in the stiffness of the various wall components to determine and distribute loads to the wall reinforcement layers and the facing.

Though all of these methods can be used to evaluate the potential for reinforcement rupture and pullout for the Strength and Extreme Event limit states, only the Stiffness Method can be used to directly evaluate the potential for soil backfill failure. These other methods used in historical practice indirectly account for soil failure based on the successful construction of thousands of structures (i.e., if the other limit states are met, soil failure will be prevented, and the wall will meet serviceability requirements for internal stability).

Detailed Stiffness Method procedures and design examples are provided in Allen and Bathurst (2018) in the Supplemental Data associated with that paper, and additional examples are provided in Appendix 15-E.

A key parameter for this method is the geosynthetic secant creep stiffness at 1,000 hours and 2% strain as determined using AASHTO R-69. Product specific creep stiffness test data can be obtained from NTPEP (2019) and Allen and Bathurst (2019).

For the Stiffness Method, T_{max} is calculated as follows:

$$T_{max} = S_v\,[H\gamma_r D_{tmax} + \gamma_r(H_{ref}/H)S]k_{avh}\,\Phi \tag{15-1}$$

where,

- S_v = tributary vertical thickness for reinforcement layer (ft)
- H = height of wall (ft)
- H_{ref} = reference wall height = 20 ft
- γ_r = unit weight of soil in wall reinforcement zone (lbs/ft^3)
- S = average soil surcharge thickness over reinforcement (ft)
- γ_f = unit weight of soil in wall in surcharge above wall (lbs/ft^3)
- D_{tmax} = T_{max} distribution factor (dim)
- k_{avh} = active earth pressure coefficient for a wall with a vertical face (dim.)
- Φ = empirically determined influence factor that captures the effect that the soil reinforcement properties, soil cohesion, and wall geometry have on T_{max} (dim)

D_{tmax} shall be determined as follows:

For $z < z_b$:

$$D_{tmax} = D_{tmax0} + (z/z_b)(1 - D_{tmax0}) \tag{15-2}$$

For $z \geq z_b$: $D_{tmax} = 1.0$

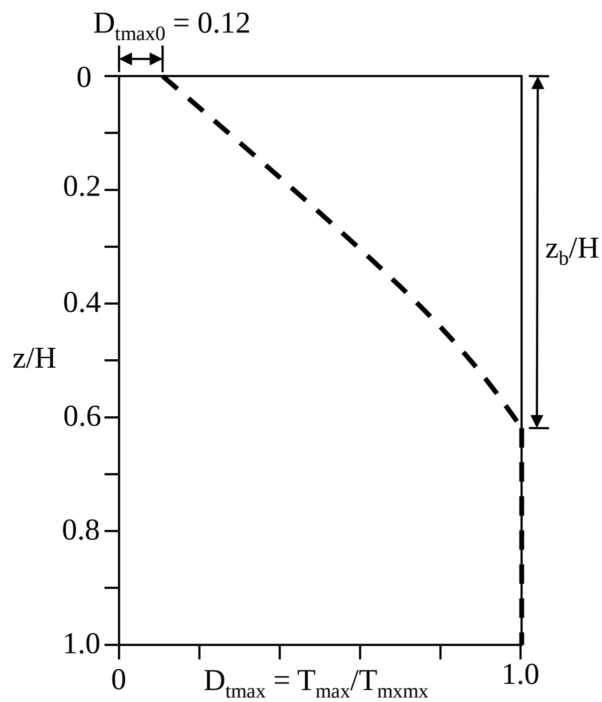
$$z_b = C_h\,(H)^{1.2} \tag{15-3}$$

where,

- z = depth of reinforcement layer below top of wall at wall face (ft)
- z_b = depth below top of wall at wall face where D_{tmax} becomes equal to 1.0 (and below which D_{tmax} equals 1.0) (ft)
- D_{tmax0} = T_{max} distribution factor magnitude at top of wall at wall face, equal to 0.12 (dim)
- C_h = coefficient equal to 0.32 when H is in ft and 0.40 when H is in meters

Determination of the T_{max} distribution factor D_{tmax} is illustrated in Figure 15-12. In the figure, depths below the wall top have been normalized by the wall height, H. T_{mxmx} is the maximum value of T_{max} in the wall section where the soil backfill failure surface crosses the reinforcement layers.

Figure 15-12 Illustration of D_{tmax} factor for the Stiffness Method



For vertical or near-vertical walls (i.e., a facing batter of 10° or less from the vertical) with a single reinforcement strength and stiffness, and cohesionless backfill soil (defined as having a plasticity index of 6 or less), Φ may be determined as follows:

$$\Phi = \Phi_g \Phi_{fs} \tag{15-4}$$

where,

Φ_g = global stiffness factor (dim)
Φ_{fs} = facing stiffness factor (dim)

The global stiffness factor Φ_g shall be determined as follows:

$$\Phi_g = \alpha \left(\frac{S_{global}}{p_a} \right)^{\beta} \tag{15-5}$$

where,

α = empirical coefficient = 0.16
β = empirical exponent = 0.26
S_{global} = global reinforcement stiffness (ksf)
P_a = atmospheric pressure at sea level (equals 2.11 ksf if S_{global} is in ksf, or 101 kPa if S_{global} is in kPa)

and,

$$S_{\text{global}} = \frac{J_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} R_c J_i}{H} \tag{15-6}$$

where,

J_{ave} = average secant tensile creep stiffness corrected for the coverage ratio, i.e., R_cJ_i, of all "n" reinforcement layers (kips/ft)

J_i = secant tensile creep stiffness of reinforcement layer i per unit of reinforcement width (kips/ft)

R_c = reinforcement coverage ratio (dim)

n = number of reinforcement layers in wall section (dim)

S_{global} and Φ_g shall be determined per unit of wall width rather than per reinforcement width, as T_{max} represents a force per unit per unit of wall width. Hence, R_c is included in Equation 15-6.

For geogrids and geotextiles, the reinforcement stiffness J_i should be based on the laboratory secant creep stiffness at 2% strain and 1,000 hours as specified in AASHTO R-69. For polymer strap walls, working strains tend to be lower than for other geosynthetics based on strain measurements observed in full scale polymer strap walls (Miyata et al., 2018), and J_i determined at a strain level of 1% may be more appropriate.

The facing stiffness factor Φ_{fs} shall be determined as follows:

$$\Phi_{fs} = \eta\left(\left(\frac{S_{global}}{p_a}\right)F_f\right)^{\kappa} \tag{15-7}$$

where,

η = empirical coefficient = 0.57

κ = empirical exponent = 0.15

F_f = facing stiffness parameter as calculated using Equation 15-8 (dim)

$$F_f = \frac{1.5H^3 p_a}{Eb^3(h_{eff}/H)} \tag{15-8}$$

where,

E = elastic modulus of the "equivalent elastic beam" representing the wall face (ksf)

b = thickness of the facing column (ft)

h_{eff} = equivalent height of an un-jointed facing column that is approximately 100% efficient in transmitting moment through the height of the facing column (ft)

All other variables are as defined previously.

For a flexible faced wall with extensible reinforcement (e.g., geosynthetics), and for all inextensible reinforced (e.g., steel) walls, set Φ_{fs} = 1. For full height and incremental panel walls, h_{eff} = H and panel height, respectively. Since the facing stiffness factor Φ_{fs} is intended to be a single value for the wall, a single representative value of h_{eff} must be selected. Typically, h_{eff} is set equal to the reinforcement vertical spacing in modular block-type structures since the reinforcement is located at the horizontal joints between facing units. For blocks that do not have a reinforcement layer at the horizontal joints, these facings will have better interlock and moment transfer from block to block. If the reinforcement spacing is non-uniform, the smallest predominate spacing (e.g., involving 3 or more reinforcement layers in the wall), defined as a spacing that involves three or more reinforcement layers, should be used for this calculation. Smaller h_{eff} values will lead to more conservative (safer) design because the facing stiffness factor will be larger. For two-stage walls in which the outer facing is built after the wall is built to full height, the facing stiffness factor shall be based on the facing stiffness of the first stage wall (typically the first stage wall face is flexible, and Φ_{fs} = 1.0 in that case). The facing stiffness factor Φ_{fs} could also be conservatively set to 1.0 for tall geosynthetic walls (i.e., H > 30 ft) and for typical "thin" panel-face systems, such as incremental concrete panels.

To calculate F_f, an elastic modulus of the facing column is needed. For wet cast concrete (e.g., in incremental concrete panels), the modulus typically is typically 300,000 to 600,000 ksf. For dry cast concrete, the elastic modulus is typically less, on the order of 200,000 to 250,000 ksf. In addition, for dry cast concrete facing blocks, if the blocks are not solid or have an irregular geometry, this modulus should be further reduced based on the plan view cross-sectional area of the block.

For discontinuous reinforcement, the reinforcement coverage ratio shall be determined as specified in Article 11.10.6.4.1 of the AASHTO *LRFD Bridge Design Specifications*.

If the wall is tall enough such that layers with different strength and stiffness properties are needed to match the layer strengths to the layer specific T_{max} values, the complete Stiffness Method equation should be used, though the complete equation can be used any time if a more accurate estimate of T_{max} is desired. For the complete Stiffness Method, Φ in Equation 15-4 is expanded as follows:

$$\Phi = \Phi_g \Phi_{fs} \Phi_{fb} \Phi_{local} \Phi_c \qquad (15\text{-}9)$$

where,

Φ_g = global stiffness factor (dim)
Φ_{fs} = facing stiffness factor (dim)
Φ_{fb} = facing batter factor (dim)
Φ_{local} = local stiffness factor (dim)
Φ_c = soil cohesion factor (dim)

Φ_g and Φ_{fs} are determined as shown in equations 15-5 and 15-7. Φ_{fb} shall be determined as follows:

$$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}}\right)^d \qquad (15\text{-}10)$$

where,

d = empirical exponent = 0.40
K_{abh} = coefficient of active lateral earth pressure considering wall face batter (dim)
K_{avh} = coefficient of active lateral earth pressure not considering wall face batter (i.e., assuming wall face is vertical) (dim)

For both determinations of the coefficient of active lateral earth pressure, wall friction is assumed to be zero.

The local stiffness factor, Φ_{local}, shall be determined as follows:

$$\Phi_{local} = \left(\frac{S_{local}}{S_{localave}}\right)^{a} \tag{15-11}$$

where,

a = empirical exponent = 0.50 for extensible reinforcement (e.g., geotextiles, geogrids, polymer straps)

S_{local} = local reinforcement stiffness determined as follows:

$$S_{local} = R_C J / S_v \tag{15-12}$$

where,

R_C, J_i and S_v are as defined previously

$S_{localave}$ shall be determined as follows:

$$S_{localave} = \frac{\sum_{i=1}^{n}(R_c J_i / S_V)}{n}$$

where,

all variables are as defined previously.

As is true for S_{global}, S_{local} , $S_{localave}$, and Φ_{local} shall be determined per unit of wall width rather than per reinforcement width, as T_{max} represents a force per unit per unit of wall width. Hence, R_c is included in equations 15-11 and 15-12.

The soil cohesion factor, Φ_c, shall be determined as follows:

$$\Phi_c = e^{\lambda(c/(\gamma_r H))} \tag{15-13}$$

where,

e = base for the natural logarithm, equal to approximately 2.718…

λ = empirical coefficient within exponent = -16

c = cohesion of MSE wall backfill (psf)

All other variables are as defined previously.

Note that this cohesion term does not apply to apparent cohesion resulting from matric suction or nonlinearity of Mohr's envelope (Allen and Bathurst 2018). See Table 15-E-2 for selecting soil parameters for design and how soil cohesion should be handled. Soil backfill cohesion shall be assumed to be zero for design. Furthermore, for WSDOT projects, cohesive backfill shall not be used for the MSE wall. However, if soil cohesion (i.e., "true cohesion" as identified in Table 15-E-2) is present, Φ_c may be used to assess the potential for post-construction deformation and reinforcement load increase. See Appendix 15-E for additional information on this subject.

Conceptually, the Stiffness Method was developed by starting with the Simplified Method, but modifying that method empirically to improve its accuracy, considering the stiffness of the wall components, and improving the distribution of T_{max} as a function of depth in the wall to more accurately reflect full scale wall measurements. Figure 15-13 illustrates the relationship between the Simplified Method and the Stiffness Method.

Figure 15-13 Comparison of AASHTO Simplified and Stiffness Method equations (Allen and Bathurst 2015)

Simplified Method

$$T_{max} = S_v \sigma_v K = S_v [\gamma_f Z + \gamma_f S] K_a (K_r/K_a)$$

$$T_{max} = S_v [\gamma_f H D_{tmax} + \gamma_f (H_{ref}/H) S] K_{avh} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c$$

Stiffness Method

15-5.3.10.2 *Load and Resistance Factors for the Stiffness Method*

Table 15-5 provides a summary of the load and resistance factors needed for MSE wall internal stability design using the Stiffness method to estimate T_{max}. Reliability theory, using the Monte Carlo method as described in Allen et al. (2005), was used to determine the load and resistance factors provided in the table. For additional information regarding calibration of these load and resistance factors, see Allen and Bathurst (2018) and the Supplemental Materials associated with that paper. Note that the resistance factors were adjusted relative to Allen and Bathurst (2018) to reflect the load factor (i.e., 1.35 for vertical earth pressure, EV) currently in the AASHTO *LRFD Bridge Design Manual* for the Strength Limit State.

Table 15-5 Load and Resistance Factors for the Stiffness Method (Service and Strength Limit States)

Limit State[1]	Reinforcement Type	Load Factor, $\gamma_{p\text{-}EV}$, $\gamma_{p\text{-}EVc}$, and $\gamma_{p\text{-}EVsf}$	Live Load,[2] γ_{LL}	Resistance Factor ϕ_{rr}, ϕ_{cr}, ϕ_{po} and ϕ_{sf}
Reinforcement rupture, $\gamma_{p\text{-}EV}$, and connection failure, $\gamma_{p\text{-}con}$ (strength limit)	Geogrids and geotextiles	1.35	1.75	0.80
	[4]Polymer straps	1.35	1.75	0.55
Soil failure, $\gamma_{p\text{-}EVsf}$ (service limit)	All geosynthetics	1.20	1.0	1.0
Pullout, $\gamma_{p\text{-}EV}$ (strength limit - default model in AASHTO 2020)[3]	All geosynthetics	1.35	N/A	0.70

Notes:

[1] Based on probability of failure = 1% (target reliability index $\beta = 2.3$) to determine resistance factor for strength limit states. Probability of failure = 15% ($\beta = 1.0$) for service limit state. See Allen and Bathurst (2018) and Bathurst et al. (2019) for additional background on these calibrations.

[2] AASHTO (2020); Berg et al. (2009) use $\gamma_{ES} = 1.5$ for traffic loads on MSE walls.

[3] The pullout resistance factor was developed assuming that the default pullout models provided in AASHTO 2020 are used. See Bathurst et al. (2019) for reliability theory calibrations using available empirical data. See Miyata et al. (2019) for pullout model calibration for polymer strap reinforcement.

[4] Also termed geostrips.

15-5.3.10.3 *Design for Internal Stability Limit States Using the Stiffness Method*

Limit states considered here include the soil failure limit state in Service I, and pullout, reinforcement strength, and connection strength in Strength I and Extreme Event I (seismic) and II (scour).

15-5.3.10.3.1 *Soil Failure Limit State (Service I)*

The soil failure limit state is considered a service limit state for design because it is a deformation criterion. Furthermore, if this criterion is substantially exceeded, the structure will not collapse but will more likely develop progressive increases in facing deformation.

The soil failure limit state often controls the amount of geosynthetic reinforcement required. See Allen and Bathurst (2019) for proof of this and to determine the relationship between creep stiffness and tensile strength. Therefore, it is recommended that this limit state be checked first to establish the minimum reinforcement stiffness required and to use this as input for determining T_{max} for reinforcement and connection rupture, and pullout. For wall systems that have relatively low facing-reinforcement connection strength, it is possible that connection strength may control the amount of reinforcement needed instead. If this is the case, be sure to check whether or not the increased tensile strength will require a stiffer reinforcement, in which case, the increased stiffness value(s) will need to be used to recalculate T_{max} (i.e., it is important to make sure that the tensile strength and stiffness specified for final design are well matched).

Reinforced fill soil failure is defined to occur when the working strain in the reinforcement exceeds a value sufficient to allow the soil to reach or exceed its peak shear strength and a contiguous shear failure zone within the reinforced wall backfill develops. For the stiffness Method as described in GDM Section 15-5.3.10.1, the wall shall be designed to prevent failure of the soil within the reinforced soil mass, thus preserving working stress conditions. To prevent exceedance of the soil failure limit state, the reinforcement strain ε_{rein} in individual layers shall be determined as follows for extensible reinforcement:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} \leq \varepsilon_{mxmx} \tag{15-14}$$

where,

- ε_{rein} = the reinforcement strain in any individual reinforcement layer corresponding to T_{max} (%)
- $\gamma_{p\text{-}EVsf}$ = load factor for prediction of T_{max} for the soil failure limit state in Table 15-5 (dim)
- T_{max} = the maximum load in the reinforcement at each reinforcement level, as specified in Section 15-5.3.10.1 (kips/ft)
- ϕ_{sf} = resistance factor that accounts for uncertainty in the measurement of the reinforcement stiffness at the specified strain, as specified in Table 15-5 (dim)
- R_c = reinforcement coverage ratio (dim)
- J_i = secant tensile stiffness of reinforcement layer i per unit of reinforcement width (kips/ft)
- ε_{mxmx} = maximum acceptable strain in the wall cross-section corresponding to T_{max} in any reinforcement layer (%)

J_i should be determined at a strain of 2% for geogrids and geotextiles. J_i should be determined at 1,000 hrs or the estimated time to complete the wall, as specified in AASHTO R-69. ϕ_{sf} is as specified in Table 15-5.

If multiple load sources are acting on the reinforced soil backfill, they shall be added to T_{max} as determined using Equation 15-1 by using superposition.

The maximum acceptable strain in each reinforcement layer ε_{mxmx} corresponding to T_{max} should be set at 2.0% strain for stiff faced walls and 2.5% strain for flexible faced walls. These criteria have the objective of preventing the development of a contiguous shear surface though the reinforced soil zone. If it is decided to treat the wall as having a flexible face (i.e., a facing stiffness factor of 1.0) even though the facing is classified as a stiff face, such as a modular block facing, or if the calculated facing stiffness factor is 1.0, such as typically occurs for taller walls, the maximum acceptable strain for a flexible faced wall should be used.

For polymer strap walls, working strains tend to be lower than for other geosynthetics based on strain measurements observed in full scale polymer strap walls (Miyata et al., 2018), and J_i determined at a strain level of 1% may be more appropriate.

Note that to account for reinforcement coverage ratios less than one, R_c must be included in Equation 15-14 as shown, where J_i is the reinforcement stiffness from laboratory testing.

15-5.3.10.3.2 *Pullout Limit State (Strength I)*

The requirements in the AASHTO *LRFD Bridge Design Manual* apply, except that T_{max} is calculated using the Stiffness Method, and T_{max} is considered to be unfactored. Therefore, the pullout limit state equation in the current AASHTO LRFD specifications is modified to be as follows:

$$L_e \geq \frac{\gamma_{p-EV} T_{max}}{\phi_{po} F^* \alpha \sigma_v C R_c} \tag{15-15}$$

where,

- L_e = length of reinforcement in resisting zone (ft)
- T_{max} = applied load in the reinforcement as specified in Section 15-5.3.10.1 (kips/ft)
- $\gamma_{p\text{-EV}}$ = load factor for vertical earth pressure specified in Table 15-5 (dim.)
- ϕ_{po} = resistance factor for reinforcement pullout from Table 15-5 (dim.)
- F^* = pullout friction factor (dim.)
- α = scale effect correction factor (dim.)
- σ_v = unfactored vertical stress at the reinforcement level in the resistant zone (ksf)
- C = overall reinforcement surface area geometry factor based on the gross perimeter of the reinforcement and is equal to 2 for strip, grid and sheet-type reinforcements, i.e., two sides (dim.)
- R_c = reinforcement coverage ratio determined as shown in the AASHTO *LRFD Bridge Design Manual*, Article 11.10.6.4.1 (dim.)

If T_{max} includes multiple load sources with different load factors, $\gamma_{p\text{-}EV}T_{max}$ should be replaced with T_{totalf}, calculated using superposition, as follows:

$$T_{totalf} = \gamma_{p\text{-}EV}T_{max} + \gamma_{p\text{-}ES}S_V(k_a\Delta\sigma_V + \Delta\sigma_H) \tag{15-16}$$

where,

- $\gamma_{p\text{-}EV}$ = load factor for vertical earth pressure specified in Table 15-5 (dim.)
- $\gamma_{p\text{-}ES}$ = load factor for earth surcharge (ES) in the AASHTO *LRFD Bridge Design Manual*, Table 3.4.1-2
- $\Delta\sigma_v$ = vertical soil stress due to concentrated load such as a footing load (ksf)
- $\Delta\sigma_H$ = horizontal stress at reinforcement level resulting from a concentrated horizontal surcharge load (ksf)
- S_v = tributary layer vertical thickness for reinforcement (ft)
- K_a = active lateral earth pressure coefficient (dim)

Note that Equation 15-16 does not include traffic live load nor seismic load.

For polymer strap reinforcement, the default pullout F* envelope and α value in the AASHTO *LRFD Bridge Design Manual* (Figure 11.10.6.3.2-2 and Table 11.10.6.3.2-1, respectively) for geogrids shall be used.

15-5.3.10.3.3 *Reinforcement Tensile and Connection Strength Limit States (Strength I)*

The requirements in the AASHTO *LRFD Bridge Design Manual* apply, except that T_{max} is calculated using the Stiffness Method, and T_{max} is considered to be unfactored. Therefore, the reinforcement strength limit state equation in the current AASHTO LRFD specifications is modified to be as follows:

$$\gamma_{p\text{-}EV}\ T_{max} \leq \phi T_{al} R_c \tag{15-17}$$

where,

- T_{max} = applied load in the reinforcement as specified in Section 15-5.3.10.1 (kips/ft)
- $\gamma_{p\text{-}EV}$ = load factor for vertical earth pressure specified in Table 3.4.1-2 (dim.)
- ϕ = resistance factor for reinforcement tension, specified in Table 15-5 (dim.)
- $T_{a\ell}$ = nominal long-term reinforcement strength (kips/ft)
- R_c = reinforcement coverage ratio determined as shown in the AASHTO *LRFD Bridge Design Manual*, Article 11.10.6.4.1 (dim.)

If traffic live load is present replace $\gamma_{p\text{-}EV}T_{max}$ with T_{totalf} calculated as shown below:

$$T_{totalf} = \gamma_{p\text{-}EV}T_{max} + (\gamma_{LS})\gamma_f h_{eq} < \phi T_{al} R_c \tag{15-18}$$

where,

- T_{totalf} = total factored load for each reinforcement layer (lbs/ft)
- γ_{LS} = load factor for live load surcharge, LS, as specified in the AASHTO *LRFD Bridge Design Manual*, Table 3.4.1-1 (dim.)
- γ_f = unit weight of soil used to calculate live load surcharge, LS (lbs/ft^3)
- h_{eq} = equivalent height of soil for live load surcharge (ft)

If multiple load sources other than traffic live load are present, use Equation 15-16 to determine T_{totalf}. It follows that if these additional load sources are added by superposition for the Strength limit state design, that these additional load sources should also be added by superposition to the Service limit state value of T_{max} in Equation 15-14. However, doing so is likely to be excessively conservative, especially for typical loads used for bridge footings. If such foundation loads are present above the reinforced soil portion of the wall, it may be best to design the geosynthetic wall using the Simplified Method or using limit equilibrium as included in the AASHTO LRFD Bridge Design Specifications.

The long-term geosynthetic strength away from the connection of the reinforcement to the wall facing shall be determined in accordance with the AASHTO *LRFD Bridge Design Manual*, Article 11.10.6.4, and AASHTO R-69. Values of T_{al} for specific geosynthetic products shall be as provided in the WSDOT QPL, Appendix D.

For the reinforcement connection strength, the AASHTO *LRFD Bridge Design Manual* requirements shall apply. Connection tests shall be conducted in accordance with ASTM D6638 to obtain the short-term connection strength $T_{ultconn}$ for modular block facings or ASTM D4884 for seam connections. The connection strength requirements provided for the specific wall systems identified in the appendices to Chapter 15 shall be used.

15-5.3.10.3.4 *Seismic Internal Stability Design Using the Stiffness Method*

The requirements in the AASHTO *LRFD Bridge Design Manual*, Article 11.10.7.2, apply, except that T_{max} is calculated using the Stiffness Method, and the additional seismically induced reinforcement load is added to T_{max} using superposition. The load and resistance factors for the Extreme Event I Limit State provided in the AASHTO *LRFD Bridge Design Manual* shall be used, except that the resistance factors for reinforcement tensile resistance and pullout resistance shall be reduced to 1.0. See Appendix 15-E for additional details on requirements for conducting seismic design for internal stability using the Stiffness Method.

15-5.4 Prefabricated Modular Walls

Modular block walls without soil reinforcement, gabion, bin, and crib walls shall be considered prefabricated modular walls.

In general, modular block walls without soil reinforcement (referred to as Gravity Block Walls in the *Standard Specifications* Section 8-24 shall have heights no greater than 2.5 times the depth of the block into the soil perpendicular to the wall face, and shall be stable for all modes of internal and external stability failure mechanisms. In no case, shall their height be greater than 15 feet. Gabion walls shall be 15 feet or less in total height. Gabion baskets shall be arranged such that vertical seams are not aligned, i.e., baskets shall be overlapped.

15-5.5 Rock Walls

Rock walls shall be designed in accordance with the *Standard Specifications*, and the wall-slope combination shall be stable regarding overall stability as determined per Chapter 7.

Rock walls shall not be used unless the retained material would be at least minimally stable without the rock wall (a minimum slope stability factor of safety of 1.25). Rock walls are considered to act principally as erosion protection and they are not considered to provide strength to the slope unless designed as a buttress using limit equilibrium slope stability methods. Rock walls shall have a batter of 6V:1H or flatter. The rocks shall increase in size from the top of the wall to the bottom at a uniform rate. The minimum rock sizes shall be:

Table 15-6 Minimum Rock Sizes for Rock Walls

Depth from Top of Wall (feet)	Minimum Rock Size	Typical Rock Weight (lbs)	Average Dimension (inches)
0	Two Man	200-700	18-28
6	Three Man	700-2000	28-36
9	Four Man	2000-4000	36-48
12	Five Man	4000-6000	48-54

Rock walls shall be 12 feet or less in total height. Rock walls used to retain fill shall be 6 feet or less in total height. Fills constructed for this purpose shall be compacted to 95 percent maximum density, per WSDOT *Standard Specifications* Section 2-03.3(14)D.

Rock walls should be designed in accordance with FHWA Manual No. FHWA- CFL/TD-06-006 (Mack, et al., 2006), but subject to the limitations and requirements specified in this GDM.

15-5.6 Reinforced Slopes

Reinforced slopes do not have a height limit but they do have a face slope steepness limit. Reinforced slopes steeper than 0.5H:1V shall be considered to be a wall and designed as such. Reinforced slopes with a face slope steeper than 1.2H:1V shall have a wrapped face or a welded wire slope face, but should be designed as a reinforced slope. Slopes flatter than or equal to 1.2H:1V shall be designed as a reinforced slope, and may use turf reinforcement to prevent face slope erosion except as noted below. Reinforcing shall have a minimum length of 6 feet. Turf reinforcement of the slope face shall only be used at sites where the average annual precipitation is 20 in or more. Sites with less precipitation shall have wrapped faces regardless of the face angle. The primary reinforcing layers for reinforced slopes shall be vertically spaced at 3 feet or less. Primary reinforcement shall be steel grid, geogrid, or geotextile. The primary reinforcement shall be designed in accordance with Berg, et al. (2009), using allowable stress design procedures, since LRFD procedures are not available. Secondary reinforcement centered between the primary reinforcement at a maximum vertical spacing of 1 foot shall be used, but it shall not be considered to contribute to the internal stability. Secondary reinforcement aids in compaction near the face and contributes to surficial stability of the slope face. Design of the secondary reinforcement should be done in accordance with Berg, et al. (2009). The secondary reinforcement ultimate tensile strength measured per ASTM D6637 or ASTM D4595 should not be less than 1,300 lb/ft in the direction of tensile loading to meet survivability requirements. Higher strengths may be needed depending on the design requirements. Gravel borrow shall be used for reinforced slope construction as modified by the *General Special Provisions* in Division 2 (see GDM Appendix 15-A for details). The design and construction shall be in accordance with the General Special Provisions.

15-5.7 Soil Nail Walls

Soil nail walls shall be designed in accordance with the most current edition of AASHTO *LRFD Bridge Design Manual*. The following manual should be consulted for additional information on soil nail wall design; however, the AASHTO *LRFD Bridge Design Manual* shall govern if there are any conflicts.

Lazarte, C. A., Robinson, H., Gomez, J. E., Baxter, A., Cadden, A., and Berg, R., 2015. *Geotechnical Engineering Circular No. 7*, *Soil Nail Walls – Reference Manual*, U.S. Department of Transportation, Federal Highway Administration, FHWA-NHI-14-007, 425 pp.

For external stability and compound stability analysis, as described in Section 15-5.3.4 and the AASHTO *LRFD Bridge Design Specifications*, limit equilibrium slope stability analysis as described in Chapter 7 should be used.

The geotechnical designer shall design the wall at critical wall sections. Each critical wall section shall be evaluated during construction of each nail lift. To accomplish this, the wall shall be analyzed for the case where excavation has occurred for that lift, but the nails have not been installed. The minimum construction safety factor shall be 1.2 for noncritical walls and 1.35 for critical walls (e.g., when underpinning bridge abutments or other structures that are sensitive to settlement). However, temporary and permanent underpinning of bridge, wall, or other moderately to heavily loaded structure foundations with soil nail walls, or other cut wall types that use non-tensioned drilled in place lateral elements, shall not be done without approval by the WSDOT State Geotechnical Engineer and State Bridge Engineer.

Permanent soil nails shall be installed in predrilled holes. Soil nails that are installed concurrently with drilling shall not be used for permanent applications, but may be used in temporary walls.

Soil nail tendons shall be number 6 bar or larger and a minimum of 12 feet in length or 60 percent of the total wall height, whichever is greater. Nail testing shall be in accordance with the WSDOT *Standard Specifications* and *General Special Provisions*.

The nail spacing should be no less than 3 feet vertical and 3 feet horizontal. In very dense glacially over consolidated soils, horizontal nail spacing should be no greater than 8 feet and vertical nail spacing should be no greater than 6 feet. In all other soils, horizontal and vertical nail spacing should be 6 feet or less.

Nails may be arranged in a square row and column pattern or an offset diamond pattern. Horizontal nail rows are preferred, but sloping rows may be used to optimize the nail pattern. As much as possible, rows should be linear so that each individual nail elevation can be easily interpolated from the station and elevation of the beginning and ending nails in that row. Nails that cannot be placed in a row must have station and elevation individually identified on the plans. Nails in the top row of the wall shall have at least 1 foot of soil cover over the top of the drill hole during nail installation. Horizontal nails shall not be used. Nails should be inclined at least 10 degrees downward from horizontal. Inclination should not exceed 30 degrees.

Walls underpinning structures such as bridges and retaining walls shall have double corrosion protected (encapsulated) nails within the zone of influence of the structure being retained or supported.

Furthermore, nails installed in soils with strong corrosion potential, defined as:

- pH < 4.5 or > 10 (AASHTO T289),
- Resistivity < 2000 ohm-cm (AASHTO T-288),
- Sulphates > 200 ppm (AASHTO T290), or
- Chlorides > 100 ppm (AASHTO T291)

shall also have double corrosion protection. All other nails shall be epoxy, coated unless the wall is temporary and in soils not defined as having strong corrosion potential.

For inspection of soil nail wall installation and testing, the guidance in the following manual should be used:

Porterfield, J. A., Cotton, D. A., Byrne, R. J., 1994, *Soil Nail Walls-Demonstration Project 103*, *Soil Nailing Field Inspectors Manual*, U.S. Department of Transportation, Federal Highway Administration, FHWA-SA-93-068, 86 pp.

15-6 Standard Plan Walls

Currently, two Standard Plan walls are available for use on WSDOT projects. These include standard cast-in-place reinforced concrete walls (Standard Plans D-10.10 through D-10.45), and standard geosynthetic walls (Standard Plans D-3, 3a, 3b, and 3c). For Standard Plan walls, the internal stability design and the external stability design for overturning and sliding stability have already been completed, and the maximum soil bearing stress below the wall calculated, for a range of loading conditions. The geotechnical designer shall identify the appropriate loading condition to use (assistance from the Bridge and Structures Office and/or the project office may be needed), and shall assess overall slope stability, compound stability for geosynthetic walls as applicable, soil bearing resistance, and settlement for each standard plan wall. If it is not clear which loading condition to use, both external and internal stability may need to be evaluated to see if one of the provided loading conditions is applicable to the wall under consideration. The geotechnical designer shall assess whether or not a Standard Plan wall is geotechnically applicable and stable given the specific site conditions and constraints.

The Standard Plan walls have been designed using LRFD methodology in accordance with the AASHTO *LRFD Bridge Design Specifications*. Standard Plan reinforced concrete walls are designed for internal and external stability using the following parameters:

- A_s = 0.51g for Wall Types 1 through 4, and 0.20g for Wall Types 5 through 8. For sliding stability, the wall is allowed to slide 4 in to calculate k_h from A_s using a Newmark deformation analysis, or a simplified version of it.
- For the wall Backfill, φ = 36° and γ = 130 pcf.
- For the foundation soil, for sliding stability analysis, φ = 32°.
- Wall settlement criteria are as specified in Table 15-2.

Standard Plan geosynthetic walls are designed for internal and external stability using the following parameters:

- A_s = 0.51g for Wall Types 1 through 4, and 0.20g for Wall Types 5 through 8. For sliding stability, the wall is allowed to slide 8 in to calculate k_h from A_s using a Newmark deformation analysis, or a simplified version of it.

- For the wall Backfill, φ = 38° and γ = 130 pcf.
- For the foundation soil, for sliding stability analysis, φ = 36°, and interface friction angle of 0.7×36° = 25°.
- For the retained soil behind the soil reinforcement, for external stability analysis, φ = 36° and γ = 130 pcf.
- Wall settlement criteria are as specified in Table 15-2, unless the settlement of the first stage wall (i.e., the geosynthetic wall without the final concrete fascia) is complete before the final concrete fascia is installed, in which case the settlement criteria in Table 15-4 may be used).

Regarding the seismic sliding analysis, the geotechnical and structural designers should determine if the amount of deformation allowed (4 in for reinforced concrete walls and 8 in for geosynthetic walls) is acceptable for the wall and anything above the wall that the wall supports. Note that for both static and seismic loading conditions, no passive resistance in front of the geosynthetic wall is assumed to be present for design.

15-7 Temporary Cut Slopes and Shoring

This section addresses the design requirements for temporary cut slopes and shoring, both separately and in combination. For temporary cuts and shoring, construction submittals are required in accordance with the *Standard Specifications* M 41-10 or other contract documents. This section also addresses submittal review requirements for these temporary facilities. The design and submittal requirements for temporary fills for haul roads, construction equipment access, and other temporary construction activities are as specified in Section 9.5.5.

15-7.1 Overview

Temporary shoring, cofferdams, and cut slopes are frequently used during construction of transportation facilities. Examples of instances where temporary shoring may be necessary include:

- Support of an excavation until permanent structure is in-place such as to construct structure foundations or retaining walls.
- Control groundwater.
- Limit the extent of fill needed for preloads or temporary access roads/ramps.

Examples of instances where temporary slopes may be necessary include:

- Situations where there is adequate room to construct a stable temporary slope in lieu of shoring.
- Excavations behind temporary or permanent retaining walls.
- Situations where a combination of shoring and temporary excavation slopes can be used.
- Removal of unsuitable soil adjacent to an existing roadway or structure;
- Shear key construction for slide stabilization.
- Culvert, drainage trench, and utility construction, including those where trench boxes are used.

The primary difference between temporary shoring/cut slopes/cofferdams, hereinafter referred to as temporary shoring, and their permanent counterparts is their design life. Typically, the design life of temporary shoring is the length of time that the shoring or cut slope are required to construct the adjacent, permanent facility. Because of the short design life, temporary shoring is typically not designed for seismic loading, and corrosion protection is generally not necessary. Additionally, more options for temporary shoring are available due to limited requirements for aesthetics. Temporary shoring is typically designed by the contractor unless the contract plans include a detailed shoring design. For contractor designed shoring, the contractor is responsible for internal and external stability, as well as global slope stability, soil bearing capacity, and settlement of temporary shoring walls.

Exceptions to this, in which WSDOT provides the detailed shoring design, include shoring in unusual soil deposits or in unusual loading situations in which the State has superior knowledge and for which there are few acceptable options or situations where the shoring is supporting a critical structure or facility. One other important exception is for temporary shoring adjacent to railroads. Shoring within railroad right of way typically requires railroad review. Due to the long review time associated with their review, often 9 months or more, WSDOT has been designing the shoring adjacent to railroads and obtaining the railroad's review and concurrence prior to advertisement of the contract. Designers involved in alternative contract projects may want to consider such an approach to avoid construction delays.

Temporary shoring is used most often when excavation must occur adjacent to a structure or roadway and the structure or traffic flow cannot be disturbed. For estimating purposes during project design, to determine if temporary shoring might be required for a project, a hypothetical 1H:1V temporary excavation slope can be utilized to estimate likely limits of excavation for construction, unless the geotechnical designer recommends a different slope for estimating purposes. If the hypothetical 1H:1V slope intersects roadway or adjacent structures, temporary shoring may be required for construction. The actual temporary slope used by the contractor for construction will likely be different than the hypothetical 1H:1V slope used during design to evaluate shoring needs, since temporary slope stability is the responsibility of the contractor unless specifically designated otherwise by the contract documents.

15-7.2 Geotechnical Data Needed for Design

The geotechnical data needed for design of temporary shoring is essentially the same as needed for the design of permanent cuts and retaining structures. Chapter 10 provides requirements for field exploration and testing for cut slope design, and Section 15-3 discusses field exploration and laboratory testing needs for permanent retaining structures. Ideally, the explorations and laboratory testing completed for the design of the permanent infrastructure will be sufficient for design of temporary shoring systems by the Contractor. This is not always the case, however, and additional explorations and laboratory testing may be needed to complete the shoring design.

For example, if the selected temporary shoring system is very sensitive to groundwater flow velocities (e.g., frozen ground shoring) or if dewatering is anticipated during construction, as the Contractor is also typically responsible for design and implementation of temporary dewatering systems, more exploration and testing may be needed. In these instances, there may need to be more emphasis on groundwater conditions at a site; and multiple piezometers for water level measurements and a large number of grain size distribution tests on soil samples should be obtained. Downhole pump tests should be conducted if significant dewatering is anticipated, so the contractor has sufficient data to develop a bid and to design the system. It is also possible that shoring or excavation slopes may be needed in areas far enough away from the available subsurface explorations that additional subsurface exploration may be needed. Whatever the case, the exploration and testing requirements for permanent walls and cuts in the GDM shall also be applied to temporary shoring and excavation design.

15-7.3 General Design Requirements

Temporary shoring shall be designed such that the risk to health and safety of workers and the public is kept to an acceptable level and that adjacent improvements are not damaged.

15-7.3.1 *Design Procedures*

For geotechnical design of retaining walls used in shoring systems, the shoring designer shall use the AASHTO *LRFD Bridge Design Specifications* and the additional design requirements provided in the GDM. For those wall systems that do not yet have a developed LRFD methodology available, for example, soil nail walls, the FHWA design manuals identified herein that utilize allowable stress methodology shall be used, in combination with the additional design requirements in the GDM. The design methodology, input parameters, and assumptions used must be clearly stated on the required submittals (see Section 15-7.2).

Regardless of the methods used, the temporary shoring wall design must address both internal and external stability. Internal stability includes assessing the components that comprise the shoring system, such as the reinforcing layers for MSE walls, the bars or tendons for ground anchors, and the structural steel members for sheet pile walls and soldier piles. External stability includes an assessment of overturning, sliding, bearing resistance, settlement and global stability.

For geotechnical design of cut slopes, the design requirements provided in chapters 7 and 10 shall be used and met, in addition to meeting the applicable WACs (see Section 15-7.5).

For shoring systems that include a combination of soil or rock slopes above and/or below the shoring wall, the stability of the slope(s) above and below the wall shall be addressed in addition to the global stability of the wall/slope combination.

For shoring and excavation conducted below the water table elevation, the potential for piping below the wall or within the excavation slope shall be assessed, and the effect of differential water elevations behind and in front of the shoring wall, or see page in the soil cut face, shall be assessed regarding its effect on wall and slope stability, and the shoring system stabilized for that condition.

If temporary excavation slopes are required to install the shoring system, the stability of the temporary excavation slope shall be assessed and stabilized.

15-7.3.2 *Safety Factors/Resistance Factors*

For temporary structures, the load and resistance factors provided in the AASHTO *LRFD Bridge Design Specifications* are applicable. Global stability shall be evaluated for the Strength Limit State. Therefore, any structure loads present shall be factored using the Strength Limit State load factors. The resistance factor for global stability of the shoring system should be 0.75 (slope stability factor of safety of 1.3 for wall types in which LRFD procedures are not available). For soil nail walls, the load and resistance factors provided in the AASHTO *LRFD Bridge Design Manual* shall be used.

For design of cut slopes that are part of a temporary excavation, a factor of safety of 1.25 or more as specified in chapters 7 and 10, shall be used. If the soil properties are well defined and shown to have low variability, a lower factor of safety may be justified through the use of the Monte Carlo simulation feature available in slope stability analysis computer programs. In this case, a probability of failure of 0.01 or smaller shall be targeted (Santamarina, et al., 1992). However, even with this additional analysis, in no case shall a slope stability safety factor less than 1.2 be used for design of the temporary cut slope.

15-7.3.3 *Design Loads*

The active, passive, and at-rest earth pressures used to design temporary shoring shall be determined in accordance with the procedures outlined in Article 3.11.5 of the AASHTO *LRFD Bridge Design Specifications* or Section 5 of the AASHTO *Standard Specifications for Highway Bridges* (2002) for wall types in which LRFD procedures are not available. Surcharge loads on temporary shoring shall be estimated in accordance with the procedures presented in Article 3.11.6 of the AASHTO LRFD Specifications, or Section 5 of the AASHTO *Standard Specifications for Highway Bridges* (2002) for wall types in which LRFD procedures are not available. It is important to note that temporary shoring systems often are subject to surcharge loads from stockpiles and construction equipment, and these surcharges loads can be significantly larger than typical vehicle surcharge loads often used for design of permanent structures. The design of temporary shoring must consider the actual construction-related loads that could be imposed on the shoring system. As a minimum, the shoring systems shall be designed for a live load surcharge of 250 psf to address routine construction equipment traffic above the shoring system. For unusual temporary loadings resulting from large cranes or other large equipment placed above the shoring system, the loading imposed by the equipment shall be specifically assessed and taken into account in the design of the shoring system. For the case where large or unusual construction equipment loads will be applied to the shoring system, the construction equipment loads shall still be considered to be a live load, unless the dynamic and transient forces caused by use of the construction equipment can be separated from the construction equipment weight as a dead load, in which case, only the dynamic or transient loads carried or created by the use of the construction equipment need to be considered live load.

As described previously, temporary structures are typically not designed for seismic loads, provided the design life of the shoring system is 3 years or less. Similarly, geologic hazards, such as liquefaction, are not mitigated for temporary shoring systems.

The design of temporary shoring must also take into account the loading and destabilizing effect caused by excavation dewatering.

15-7.3.4 *Design Property Selection*

The procedures provided in Chapter 5 shall be used to establish the soil and rock properties used for design of the shoring system.

Due to the temporary nature of the structures and cut slopes in shoring design, long-term degradation of material properties, other than the minimal degradation that could occur during the life of the shoring, need not be considered. Therefore, corrosion for steel members, and creep for geosynthetic reinforcement, need to only be taken into account for the shoring design life.

Regarding soil properties, it is customary to ignore any cohesion present for permanent structure and slope design (i.e., fully drained conditions). However, for temporary shoring/cutslope design, especially if the shoring/cutslope design life is approximately six months or less, a minimal amount of cohesion may be considered for design based on previous experience with the geologic deposit and/or lab test results. This does not apply to glacially overconsolidated clays and clayey silts (e.g., Seattle clay), unless it can be demonstrated that deformation in the clayey soil resulting from release of locked in stresses during and after the excavation process can be fully prevented. If the deformation cannot be fully prevented, the shoring/cutslope shall be designed using the residual shear strength of the soil (see Chapter 5). If the glacially overconsolidated clay is already in a disturbed state due to previous excavations at the site or due to geologic processes such as landsliding, glacial shoving, or shearing due to fault activity, resulting in significant fracturing and slickensides, residual strength parameters should be used even if the shoring system can fully prevent further deformation (see Section 5.13.3 for additional requirements on this issue).

If it is planned to conduct soil modification activities that could temporarily or permanently disturb or otherwise loosen the soil in front of or behind the shoring (e.g., stone column installation, excavation), the shoring shall be designed using the disturbed or loosened soil properties.

15-7.4 Special Requirements for Temporary Cut Slopes

Temporary cuts slopes are used extensively in construction due to the ease of construction and low costs. Since the contractor has control of the construction operations, the contractor is responsible for the stability of cut slopes, as well as the safety of the excavations, unless otherwise specifically stated in the contact documents. Because excavations are recognized as one of the most hazardous construction operations, temporary cut slopes must be designed to meet Federal and State regulations in addition to the requirements stated in the GDM. Federal regulations regarding temporary cut slopes are presented in Code of Federal Regulations (CFR) Part 29, Sections 1926. The State of Washington regulations regarding temporary cut slopes are presented in Part N of WAC 296-155. Key aspects of the WAC with regard to temporary slopes are summarized below for convenience. To assure obtaining the most up to date requirements regarding temporary slopes, the WAC should be reviewed.

WAC 296-155 presents maximum allowable temporary cut slope inclinations based on soil or rock type, as shown in Table 15-7. WAC 296-155 also presents typical sections for compound slopes and slopes combined with trench boxes. The allowable slopes presented in the WAC are applicable to cuts 20 feet or less in height. The WAC requires that slope inclinations steeper than those specified by the WAC or for slope heights greater than 20 feet, as well as slopes in soils or rock not meeting the requirements to be classified as stable rock, or Type A, B, or C soil, shall be designed by a registered professional engineer. As a minimum, the design by or under the supervision of the registered professional engineer shall include a geotechnical slope stability analysis (i.e., Chapter 7) that is based on a knowledge of the subsurface conditions present, including soil and rock stratigraphy, engineering data that can be used to estimate soil and rock properties, and ground water conditions, and with consideration to the loading conditions on or above the slope that could affect its stability. The design shall be conducted in accordance with the requirements in this GDM and referenced documents. Engineering recommendations based upon field observations alone shall not be considered to be an engineering design as defined in the WAC and this GDM.

Table 15-7 WAC 296-155 Allowable Temporary Cut Slopes

Soil or Rock Type	Maximum Allowable Temporary Cut Slopes (20 Feet Maximum Height)
Stable Rock	Vertical
Type A Soil	¾H:1V
Type B Soil	1H:1V
Type C Soil	1½H:1V

Type A Soil – Type A soils include cohesive soils with an unconfined compressive strength of 3,000 psf or greater. Examples include clay and plastic silts with minor amounts of sand and gravel. Cemented soils such as caliche and glacial till (hard pan) are also considered Type A Soil. No soil is Type A if:

- It is fissured.
- It is subject to vibrations from heavy traffic, pile driving or similar effects.
- It has been previously disturbed.
- The soil is part of a sloped, layered system where the layers dip into the excavation at 4H:1V or greater.
- The material is subject to other factors that would require it to be classified as a less stable material.

Type B Soil – Type B soils generally include cohesive soils with an unconfined compressive strength greater than 1000 psf but less than 3000 psf and granular cohesionless soils with a high internal angle of friction, such as angular gravel or glacially overridden sand and gravel soils. Some silty or clayey sand and gravel soils that exhibit an apparent cohesion may sometimes classify as Type B soils. Type B soils may also include Type A soils that have previously been disturbed, are fissured, or subject to vibrations. Soils with layers dipping into the excavation at inclinations steeper than 4H:1V cannot be classified as Type B soil.

Type C Soil – Type C soils include most non-cemented granular soils (e.g., gravel, sand, and silty sand) and soils that do not otherwise meet Types A or B.

The allowable slopes described above apply to dewatered conditions. Flatter slopes may be necessary if seepage is present on the cut face or if localized sloughing occurs. All temporary cut slopes greater than 20 feet in height shall be designed by a registered civil engineer (geotechnical engineer). All temporary cut slopes supporting a structure or wall, regardless of height, shall also be designed by a registered civil engineer (geotechnical engineer) in accordance with the GDM. If for a specific project, as specifically identified in the contract documents, the location of a proposed temporary excavation could undermine marginally stable ground, such as would occur if the excavation will result in material being removed from the toe of an inactive or active landslide, the cut for the excavation shall be designed by a registered civil engineer (geotechnical engineer) in accordance with the GDM.

For open temporary cuts, the following requirements shall be met:

- No traffic, stockpiles or building supplies shall be allowed at the top of the cut slopes within a distance of at least 5 feet from the top of the cut.
- Exposed soil along the slope shall be protected from surface erosion,
- Construction activities shall be scheduled so that the length of time the temporary cut is left open is reduced to the extent practical.
- Surface water shall be diverted away from the excavation.
- The general condition of the slopes should be observed periodically by the Geotechnical Engineer or his representative to confirm adequate stability.

15-7.5 Performance Requirements for Temporary Shoring and Cut Slopes

Temporary shoring, shoring/slope combinations, and slopes shall be designed to prevent excessive deformation that could result in damage to adjacent facilities, both during shoring/cut slope construction and during the life of the shoring system. An estimate of expected displacements or vibrations, threshold limits that would trigger remedial actions, and a list of potential remedial actions if thresholds are exceeded should be developed. Thresholds shall be established to prevent damage to adjacent facilities, as well as degradation of the soil properties due to deformation.

Typically, the allowance of up to 1 to 2 inches of lateral movement will prevent unacceptable settlement and damage of most structures and transportation facilities. A little more lateral movement could be allowed if the facility or structure to be protected is far enough away from the shoring/slope system.

Guidance regarding the estimation of wall deformation and tolerable deformations for structures is provided in the AASHTO *LRFD Bridge Design Specifications*. Additional guidance on acceptable deformations for walls and bridge foundations is provided in Chapter 8 and Section 15-4.7.

In the case of cantilever walls, the resistance factor of 0.75 applied to the passive resistance accounts for variability in properties and other sources of variability, as well as the prevention of excess deformation to fully mobilize the passive resistance. The amount of deformation required to mobilize the full passive resistance typically varies from 2 to 6 percent of the exposed wall height, depending on soil type in the passive zone (AASHTO 2017).

15-7.6 Special Design Requirements for Temporary Retaining Systems

The design requirements that follow for temporary retaining wall systems are in addition, or are a modification, to the design requirements for permanent walls provided in Chapter 15 and its referenced design specifications and manuals. Detailed descriptions of various types of shoring systems and general considerations regarding their application are provided in Appendix 15-F.

15-7.6.1 *Fill Applications*

Primary design considerations for temporary fill walls include external stability to resist lateral earth pressure, ground water, and any temporary or permanent surcharge pressures above or behind the wall. The wall design shall also account for any destabilizing effects caused by removal or modification of the soil in front of the wall due to construction activities. The wall materials used shall be designed to provide the required resistance for the design life of the wall. Backfill and drainage behind the wall shall be designed to keep the wall backfill well drained with regard to ground see page and rainfall runoff.

If the temporary wall is to be buried and therefore incorporated in the finished work, it shall be designed and constructed in a manner that it does not inhibit drainage in the finished work, so that:

- It does not provide a plane or surface of weakness with regard to slope stability.
- It does not interfere with planned installation of foundations or utilities.
- It does not create the potential for excessive differential settlement of any structures placed above the wall.

Provided the wall design life prior to burial is three years or less, the wall does not need to be designed for seismic loading.

15-7.6.1.1 *MSE Walls*

MSE walls shall be designed for internal and external stability in accordance with Section 15-5.3 and related AASHTO Design Specifications. Because the walls will only be in service a short time (typically a few weeks to a couple years), the reduction factors (e.g., creep, durability, installation damage) used to assess the allowable tensile strength of the reinforcing elements are typically much less than for permanent wall applications. The T_{al} values (i.e., long-term tensile strength) of geosynthetics, accounting for creep, durability, and installation damage in Appendix D of the WSDOT *Qualified Products List* (QPL) may be used for temporary wall design purposes.

However, those values will be quite conservative, since the QPL values are intended for permanent reinforced structures.

Alternatively, for geosynthetic reinforcement, a default combined reduction factor for creep, durability, and installation damage in accordance with the AASHTO specifications (LRFD or *Standard Specifications*) may be used, ranging from a combined reduction factor RF of 4.0 for walls with a life of up to three years, to 3.0 for walls with a one-year life, to 2.5 for walls with a six month life. If steel reinforcement is used for temporary MSE walls, the reinforcement is not required to be galvanized, and the loss of steel due to corrosion is estimated in consideration of the anticipated wall design life.

15-7.6.1.2 *Prefabricated Modular Block Walls*

Prefabricated modular block walls without soil reinforcement are discussed in Section 15-5.4 and should be designed as gravity retaining structures. The blocks shall meet the requirements in the WSDOT *Standard Specifications*. Implementation of this specification will reduce the difficulties associated with placing blocks in a tightly fitted manner. Large concrete blocks should not be placed along a curve. Curves should be accomplished by staggering the wall in one-half to one full block widths.

15-7.6.2 *Cut Applications*

Primary design considerations for temporary cut walls include external stability to resist lateral earth pressure, ground water, and any temporary or permanent surcharge pressures above or behind the wall. The wall design shall also account for any destabilizing effects caused by removal or modification of the soil in front of the wall due to construction activities. The wall materials used shall be designed to provide the required resistance for the design life of the wall. Backfill and drainage behind the wall should be designed to keep the retained soil well drained with regard to ground water see page and rainfall runoff. If this is not possible, then the shoring wall should be designed for the full hydrostatic head.

If the temporary wall is to be buried and therefore incorporated in the finished work, it shall be designed and constructed in a manner that it does not inhibit drainage in the finished work, so that:

- It does not provide a plane or surface of weakness with regard to slope stability.
- It does not interfere with planned installation of foundations or utilities.
- It does not create the potential for excessive differential settlement of any structures placed above the wall.

Provided the wall design life prior to burial is three years or less, the wall does not need to be designed for seismic loading.

15-7.6.2.1 *Trench Boxes*

In accordance with the WSDOT *Standard Specifications*, trench boxes are not considered to be structural shoring, as they generally do not provide full lateral support to the excavation sides. Trench boxes are not appropriate for excavations that are deeper than the trench box. Generally, detailed analysis is not required for design of the system; however, the contractor should be aware of the trench box's maximum loading conditions for situations where surcharge loading may be present, and should demonstrate that the maximum anticipated lateral earth pressures will not exceed the structural capacity of the trench box. Geotechnical information required to determine whether trench boxes are appropriate for an excavation include the soil type, density, and groundwater conditions. Also, where existing improvements are located near the excavation, the soil should exhibit adequate standup time to minimize the risk of damage as a result of caving soil conditions against the outside of the trench box. In accordance with sections 15-7.3 and 15-7.4, the excavation slopes outside of the trench box shall be designed to be stable.

15-7.6.2.2 *Sheet Piling, with or without Ground Anchors*

The design of sheet piling requires a detailed geotechnical investigation to characterize the retained soils and the soil located below the base of excavation/dredge line. The geotechnical information required for design includes soil stratigraphy, unit weight, shear strength, and groundwater conditions. In situations where lower permeability soils are present at depth, sheet piles are particularly effective at cutting off groundwater flow. Where sheet piling is to be used to cutoff groundwater flow, characterization of the soil hydraulic conductivity is necessary for design.

The sheet piling shall be designed to resist lateral stresses due to soil and groundwater, both for temporary (i.e., due to dewatering) and permanent ground water levels, as well as any temporary and permanent surcharges located above the wall. If there is the potential for a difference in ground water head between the back and front of the wall, the depth of the wall, or amount of dewatering behind the wall, shall be established to prevent piping and boiling of the soil in front of the wall.

The steel section used shall be designed for the anticipated corrosion loss during the design life of the wall. The ground anchors for temporary walls do not need special corrosion protection if the wall design life is three years or less, though the anchor bar or steel strand section shall be designed for the anticipated corrosion loss that could occur during the wall design life. Easements may be required if the ground anchors, if used, extend outside the right of way/property boundary.

Sheet piling should not be used in cobbly, bouldery soil or dense soil. They also should not be used in soils or near adjacent structures that are sensitive to vibration.

15-7.6.2.3 *Soldier Piles With or Without Ground Anchors*

Design of soldier pile walls requires a detailed geotechnical investigation to characterize the retained soils and the soil located below the base of excavation. The geotechnical information required for design includes soil stratigraphy, unit weight, shear strength, surcharge loading, foreslope and backslope inclinations, and groundwater conditions. The required information presented in sections 15-3 and 15-5.3 is pertinent to the design of temporary soldier pile walls.

The wall shall be designed to resist lateral stresses due to soil and groundwater, both for temporary (i.e., due to dewatering) and permanent ground water levels, as well as any temporary and permanent surcharges located above the wall. If there is the potential for a difference in ground water head between the back and front of the wall, the depth of the wall, or amount of dewatering behind the wall, shall be established to prevent boiling of the soil in front of the wall. The temporary lagging shall be designed and installed in a way that prevents running/caving of soil below or through the lagging.

The ground anchors for temporary walls do not need special corrosion protection if the wall design life is three years or less. However, the anchor bar or steel strand section shall be designed for the anticipated corrosion loss that could occur during the wall design life. Easements may be required if the ground anchors, if used, extend outside the right of way/property boundary.

15-7.6.2.4 *Prefabricated Modular Block Walls*

Modular block walls for cut applications shall only be used in soil deposits that have adequate standup time such that the excavation can be made and the blocks placed without excessive caving or slope failure. The temporary excavation slope required to construct the modular block wall shall be designed in accordance with sections 15-7.3 and 15-7.4. See Section 15-7.6.1.2 for additional special requirements for the design of this type of wall.

15-7.6.2.5 *Braced Cuts*

The special design considerations for soldier pile and sheet pile walls described above shall be considered applicable to braced cuts.

15-7.6.2.6 *Soil Nail Walls*

Design of soil nail walls requires a detailed geotechnical investigation to characterize the reinforced soils and the soil located below the base of excavation. The geotechnical information required for design includes soil stratigraphy, unit weight, shear strength, surcharge loading, foreslope and backslope inclinations, and groundwater conditions. The required information presented in sections 15-3 and 15-5.7 is pertinent to the design of temporary soil nail walls. Easements may be required if the soil nails extend outside the right of way/property boundary.

15-7.6.3 *Uncommon Shoring Systems for Cut Applications*

The following shoring systems require special, very detailed, expert implementation, and will only be allowed either as a special design by the State, or with special approval by the State Geotechnical Engineer and State Bridge Engineer.

- Diaphragm/slurry walls
- Secant pile walls
- Cellular cofferdamsGround freezing
- Deep soil mixing
- Permeation grouting
- Jet grouting

More detailed descriptions of each of these methods and special considerations for their implementation are provided in Appendix 15-F.

15-7.7 Shoring and Excavation Design Submittal Review Guidelines

When performing a geotechnical review of a contractor shoring and excavation submittal, the following items should be specifically evaluated:

1. Shoring System Geometry

   a. Has the shoring geometry been correctly developed, and all pertinent dimensions shown?

   b. Are the slope angle and height above and below the shoring wall shown?

   c. Is the correct location of adjacent structures, utilities, etc., if any are present, shown?

2. Performance Objectives for the Shoring System

   a. Is the anticipated design life of the shoring system identified?

   b. Are objectives regarding what the shoring system is to protect, and how to protect it, clearly identified?

   c. Does the shoring system stay within the constraints at the site, such as the right of way limits, boundaries for temporary easements, etc?

3. Subsurface conditions

   a. Is the soil/rock stratigraphy consistent with the subsurface geotechnical data provided in the contract boring logs?

   b. Did the contractor/shoring designer obtain the additional subsurface data needed to meet the geotechnical exploration requirements for slopes and walls as identified in chapters 10 and 15, respectively, and Appendix 15-F for unusual shoring systems?

   c. Was justification for the soil, rock, and other material properties used for the design of the shoring system provided, and is that justification, and the final values selected, consistent with Chapter 5 and the subsurface field and lab data obtained at the shoring site?

   d. Were ground water conditions adequately assessed through field measurements combined with the site stratigraphy to identify zones of ground water, aquitards and aquicludes, artesian conditions, and perched zones of ground water?

4. Shoring system loading

   a. Have the anticipated loads on the shoring system been correctly identified, considering all applicable limit states?

   b. If construction or public traffic is near or directly above the shoring system, has a minimum traffic live load surcharge of 250 psf been applied?

   c. If larger construction equipment such as cranes will be placed above the shoring system, have the loads from that equipment been correctly determined and included in the shoring system design?

   d. If the shoring system is to be in place longer than three years, have seismic and other extreme event loads been included in the shoring system design?

5. Shoring system design

   a. Have the correct design procedures been used (i.e., the GDM and referenced design specifications and manuals)?

   b. Have all appropriate limit states been considered (e.g., global stability of slopes above and below wall, global stability of wall/slope combination, internal wall stability, external wall stability, bearing capacity, settlement, lateral deformation, piping or heaving due to differential water head)?

6. Are all safety factors, or load and resistance factors for LRFD shoring design, identified, properly justified in a manner that is consistent with the GDM, and meet or exceed the minimum requirements of the GDM?

7. Have the effects of any construction activities adjacent to the shoring system on the stability/performance of the shoring system been addressed in the shoring design (e.g., excavation or soil disturbance in front of the wall or slope, excavation dewatering, vibrations and soil loosening due to soil modification/improvement activities)?

8. Shoring System Monitoring/Testing

   a. Is a monitoring/testing plan provided to verify that the performance of the shoring system is acceptable throughout the design life of the system?

   b. Have appropriate displacement or other performance triggers been provided that are consistent with the performance objectives of the shoring system?

9. Shoring System Removal

   a. Have any elements of the shoring system to be left in place after construction of the permanent structure is complete been identified?

   b. Has a plan been provided regarding how to prevent the remaining elements of the shoring system from interfering with future construction and performance of the finished work (e.g., will the shoring system impede flow of ground water, create a hard spot, create a surface of weakness regarding slope stability)?

15-8 References

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15-9 Appendices

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Appendix 15-A Preapproved Proprietary Wall and Reinforced Slope General Design Requirements and Responsibilities

15-A-1 Design Requirements

Wall design shall be in accordance with the Geotechnical *Design Manual* (GDM), the LRFD *Bridge Design Manual* (BDM), and the AASHTO LRFD Specifications. Where there are differences between the requirements in the GDM and the AASHTO LRFD Specifications, this manual shall be considered to have the highest priority. Note that since a LRFD design method for reinforced slopes is currently not available, the allowable stress design method provided in Berg, et al. (2009) shall be used for reinforced slopes, except that geosynthetic reinforcement long-term nominal strength shall be determined in accordance with AASHTO R 69.

The wall/reinforced slope shall be designed for a minimum life of 75 years, unless otherwise specified by the State. All wall/reinforced slope components shall be designed to provide the required design life.

15-A-2 Design Responsibilities

The geotechnical designer shall determine if a preapproved proprietary wall system is suitable for the wall site. The geotechnical designer shall be responsible for design of the wall for external stability (sliding, overturning, and bearing), compound stability, and overall (global) stability of the wall. The wall/reinforced slope supplier shall be responsible to design the wall for internal stability (structural failure of wall/reinforced slope components including the soil reinforcement, facing, and facing connectors to the reinforcement, and pullout), for all applicable limit states (as a minimum, serviceability, strength and extreme event). The wall supplier shall also be responsible to design the traffic barrier (all walls) and the distribution of the impact load into the soil reinforcement (MSE walls) in accordance with the AASHTO LRFD *Bridge Design Manual* and as specified in the GDM and BDM. The wall or reinforced slope supplier, or the supplier's consultant, performing the geotechnical design of the structure shall be performed by, or under the direct supervision of, a civil engineer licensed to perform such work in the state of Washington, who is qualified by education or experience in the technical specialty of geotechnical engineering per WAC 196-27A-20. Final designs and plan sheets produced by the wall supplier shall be certified (stamped) in accordance with the applicable RCWs and WACs and as further specified in this manual (see chapters 1 and 23).

The design calculation and working drawing submittal shall be as described in *Standard Specifications* Section 6.13.3(2). All computer output submitted shall be accompanied by supporting hand calculations detailing the calculation process, unless the computer program MSEW 3.0 supplied by ADAMA Engineering, Inc., is used to perform the calculations, in which case supporting hand calculations are not required.

Overall stability and compound stability as defined in the AASHTO LRFD Specifications is the responsibility of the geotechnical designer of record for the project. The geotechnical designer of record shall also provide the settlement estimate for the wall and the estimated bearing resistance available for all applicable limit states. If settlement is too great for the wall/reinforced slope supplier to provide an acceptable design, the geotechnical designer of record is responsible to develop a mitigation design in accordance with this manual during contract preparation to provide adequate bearing resistance, overall stability, and acceptable settlement magnitude to enable final design of the structure. The geotechnical designer of record shall also be responsible to provide the design properties for the wall/reinforced slope backfill, retained fill, and any other properties necessary to complete the design for the structure, and the peak ground acceleration for seismic design. Design properties shall be determined in accordance with Chapter 5. The geotechnical designer of record is responsible to address geologic hazards resulting from earthquakes, landslides, and other geologic hazards as appropriate. Mitigation for seismic hazards such as liquefaction and the resulting instability shall be done in accordance with Chapter 6. The geotechnical designer of record shall also provide a design to make sure that the wall/reinforced slope is adequately drained, considering ground water, infiltration from rainfall and surface runoff, and potential flooding if near a body of surface water, and considering the ability of the structure backfill material to drain.

15-A-3 Limits of Preapproved Wall/Reinforced Slope Designs

Preapproved wall design is intended for routine design situations where the design specifications (e.g., AASHTO, GDM, and BDM) can be readily applied. Whether or not a particular design situation is within the limits of what is preapproved also depends specifically on what plan details the proprietary wall supplier has submitted to WSDOT for approval. See the GDM preapproved wall appendices for details. In general, all the wall systems are preapproved up to the wall heights indicated in Appendix 15-D, and are also preapproved for use with traffic barriers, guardrail, hand rails, fencing, and catch basins placed on top of the wall. Preapproval regarding culvert penetration through the wall face and obstruction avoidance details varies with the specific wall system, as described in the GDM preapproved wall appendices.

In general, design situations that are not considered routine nor preapproved are as follows:

- Very tall walls, as defined for each wall system in Appendix 15-D.
- Vertically stacked or stepped walls, unless the step is less than or equal to 5 percent of the combined wall height, or unless the upper wall is completely behind the back of the lower wall, i.e., (for MSE walls, the back of the soil reinforcement) by a distance equal to the height of the lower wall.
- Back-to-back MSE walls, unless the distance between the backs of the walls (i.e., the back of the soil reinforcement layers) is 50 percent of the wall height or more.
- In the case of MSE walls and reinforced slopes, any culvert or other conduit that has a diameter which is greater than the vertical spacing between soil reinforcement layers, and which does not come through the wall at an angle perpendicular to the wall face and parallel to the soil reinforcement layers, unless otherwise specified in the GDM preapproved wall appendix for a specific wall system.
- If the wall or reinforced slope is supporting structure foundations, other walls, noise walls, signs or sign bridges, or other types of surcharge loads. The wall or reinforced slope is considered to support the load if the surcharge load is located within a 1H:1V slope projected from the bottom of the back of the wall, or reinforced soil zone in the case of reinforced soil structures.

- Walls in which bridge or other structure deep foundations (e.g., piles, shafts, micropiles) must go through or immediately behind the wall.
- Any wall design that uses a wall detail that has not been reviewed and preapproved by WSDOT.

Backfill Selection and Effect on Soil Reinforcement Design – Backfill selection shall be based on the ability of the material to drain and the drainage design developed for the wall/reinforced slope, and the ability to work with and properly compact the soil in the anticipated weather conditions during backfill construction. Additionally, for MSE walls and reinforced slopes, the susceptibility of the backfill reinforcement to damage due to placement and compaction of backfill on the soil reinforcement shall be taken into account with regard to backfill selection.

Minimum requirements for backfill used in the reinforced zone of MSE walls and reinforced slopes are provided in the WSDOT *Standard Specifications* Section 9-03.14(4). If the wall backfill is exposed to tidal influence or other water conditions that result in significant water level changes within the reinforced soil backfill, a free draining backfill shall be used as described in Section 15.3.7.

For reinforced soil slopes, the gradation requirements in WSDOT *Standard Specifications* Section 9-03.14(4) shall be used, but modified to require the percent passing a No. 200 sieve of between 7 and 12 percent, and the minimum SE reduced to 15. Based on experience, for typical reinforced slopes, it is difficult to compact slopes with cleaner soils as well as to prevent erosion of the slope face while the slope vegetation is becoming established. However, due to the greater fines content, the reinforced soil is likely to drain more slowly than the MSE wall backfill, which should be considered in the reinforced slope design, depending on the anticipated seepage into the reinforced backfill.

All material within the reinforced zone of MSE walls, and also within the bins of prefabricated bin walls, shall be substantially free of shale or other soft, poor durability particles, and shall not contain recycled materials, such as glass, shredded tires, portland cement concrete rubble, or asphaltic concrete rubble, nor shall it contain chemically active or contaminated soil such as slag, mining tailings, or similar material.

The corrosion criteria provided in the AASHTO LRFD Specifications for steel reinforcement in soil are applicable to soils that meet the following criteria:

- pH = 5 to 10 (AASHTO T289)
- Resistivity ≥ 3000 ohm-cm (AASHTO T288)
- Chlorides ≤ 100 ppm (AASHTO T291)
- Sulfates ≤ 200 ppm (AASHTO T290)
- Organic Content ≤ 1 percent (AASHTO T267)

If the resistivity is greater than or equal to 5000 ohm-cm, the chlorides and sulfates requirements may be waived.

For geosynthetic reinforced structures, the approved products and values of T_{al} in the Qualified Products List (QPL) are applicable to soils meeting the following requirements, unless otherwise noted in the QPL or special provisions:

- Soil pH (determined by AASHTO T289) = 4.5 to 9 for permanent applications and 3 to 10 for temporary applications.

- Maximum soil particle size ≤ 1.25 inches, unless full scale installation damage tests are conducted in accordance with AASHTO R 69 so that the design can take into account the potential greater degree of damage.

Soils used for MSE walls and reinforced slopes shall meet the requirements provided above.

15-A-4 MSE Wall Facing Tolerances

The design of the MSE wall (precast panel faced, and welded wire faced, with or without a precast concrete, cast-in-place concrete, or shotcrete facia placed after wall construction) shall result in a constructed wall that meets the following tolerances:

1. Deviation from the design batter and horizontal alignment, when measured along a 10 feet straight edge, shall not exceed the following:
   a. Welded wire faced structural earth wall: 2 inches
   b. Precast concrete panel and concrete block faced structural earth wall: ¾ inch
2. Deviation from the overall design batter of the wall shall not exceed the following per 10 feet of wall height:
   a. Welded wire faced structural earth wall: 1.5 inches
   b. Precast concrete panel and concrete block faced structural earth wall: ½ inch
3. The maximum outward bulge of the face between welded wire faced structural earth wall reinforcement layers shall not exceed 2 inches. The maximum allowable offset in any precast concrete facing panel joint shall be ¾ inch. The maximum allowable offset in any concrete block joint shall be ⅜ inch.

The design of the MSE wall (geosynthetic wrapped face, with or without a precast concrete, cast-in-place concrete, or shotcrete facia placed after wall construction) shall result in a constructed wall that meets the following tolerances:

Description of Criteria	Permanent Wall	Temporary Wall
Deviation from the design batter and horizontal alignment for the face when measured along a 10 feet straight edge at the midpoint of each wall layer shall not exceed:	3 inches	5 inches
Deviation from the overall design batter per 10 feet of wall height shall not exceed:	2 inches	3 inches
Maximum outward bulge of the face between backfill reinforcement layers shall not exceed:	4 inches	6 inches

15-A-5 References

AASHTO. 2015. R-69, Standard Practice for Determination of Long-Term Strength for Geosynthetic Reinforcement. American Association of State Highway and Transportation Officials, Washington, D.C., USA.

Berg, R. R., Christopher, B. R., and Samtani, N. C., 2009, *Design of Mechanically Stabilized Earth Walls and Reinforced Slopes*, No. FHWA-NHI-10-024, Federal Highway Administration, 306 pp.

Appendix 15-B Preapproved Proprietary Wall/Reinforced Slope Design and Construction Review Checklist

The review tasks provided herein have been divided up relative to the various aspects of wall and reinforced slope design and construction. These review tasks have not been specifically divided up between those tasks typically performed by the geotechnical reviewer and those tasks typically performed by the structural reviewer. However, to better define the roles and responsibilities of each office, following each task listed below, either GT (geotechnical designer), ST (structural designer), or both are identified beside each task as an indicator of which office is primarily responsible for the review of that item.

Review contract plans, special provisions, applicable *Standard Specifications*, any contract addendums, the appendix to Chapter 15 for the specific wall system proposed in the shop drawings, and Appendix 15A as preparation for reviewing the shop drawings and supporting documentation. Also review the applicable AASHTO design specifications and Chapter 15 as needed to be fully familiar with the design requirements. If a HITEC report is available for the wall system, it should be reviewed as well.

The shop drawings and supporting documentation should be quickly reviewed to determine whether or not the submittal package is complete. Identify any deficiencies in terms of the completeness of the submittal package. The shop drawings should contain wall plans for the specific wall system, elevations, and component details that address all of the specific requirements for the wall as described in the contract. The supporting documentation should include calculations supporting the design of each element of the wall (i.e., soil reinforcement density, corrosion design, connection design, facing structural design, external wall stability, special design around obstructions in the reinforced backfill, etc., and example hand calculations demonstrating the method used by any computer printouts provided and that verify the accuracy of the computer output. The contract will describe specifically what is to be included in the submittal package.

The following geotechnical design and construction issues should be reviewed by the geotechnical designer (GT) and/or structural designer (ST) when reviewing proprietary wall/reinforced slope designs:

1. External stability design

   a. Are the structure dimensions, and design cross-sections, in the wall/reinforced slope supplier's plan consistent with the contract requirements and geotechnical design? As a minimum, check wall/slope base width, embedment depth, and face batter in comparison to the geotechnical external stability design. (GT, ST).

   b. Have the design documents and plan details been certified in accordance with this manual? (GT, ST)

2. Internal stability design

    a. Has the correct, and agreed upon, design procedure been used (i.e., as specified in the GDM, BDM, and AASHTO LRFD Specifications), including the correct earth pressures and earth pressure coefficients? (GT)

    b. Has appropriate load group for each limit state been selected? (GT, ST)

        i. In general, with the exception of the Stiffness Method described in Section 15.5.3.10.3.1 the service limit state is not specifically checked for internal stability.

        ii. Strength I should be used for the strength limit state, unless an owner specified vehicle is to be used, in which case Strength II should also be checked.

        iii. Extreme Event I should be used for seismic design.

        iv. Extreme Event II should be used for scour design.

    c. Have the correct load factors been selected (see GDM, BDM and the AASHTO LRFD Specifications)? Note that for reinforced slopes, since LRFD procedures are currently not available, load factors are not applicable to reinforced slope design. (GT, ST)

    d. Has live load been treated correctly regarding magnitude (in general, approximated as 2 feet of soil surcharge load) and location (over reinforced zone for bearing, behind reinforced zone for sliding and overturning)? (GT, ST)

    e. Have the effects of any external surcharge loads, including traffic barrier impact loads, been taken into account in the calculation of load applied internally to the wall reinforcement and other elements? (GT, ST)

    f. Has the correct PGA been used for seismic design for internal stability? (GT)

    g. Have the correct resistance factors been selected for design for each limit state? For reinforced slopes, since LRFD design procedures are currently not available, check to make sure that the correct safety factors have been selected. (GT)

    h. Have the correct reinforcement and connector properties been used?

        i. For steel reinforcement, have the steel reinforcement dimensions and spacing been identified? (GT, ST)

        ii. For steel reinforcement, has it been designed for corrosion using the correct corrosion rates, correct design life (75 years, unless specified otherwise in the contract documents)? (GT, ST)

        iii. Have the steel reinforcement connections to the facing been designed for corrosion, and has appropriate separation between the soil reinforcement and the facing concrete reinforcement been done so that a corrosion cell cannot occur, per the AASHTO LRFD Specifications? (GT, ST)

iv. For geosynthetic reinforcement products selected, are the long-term design nominal strengths, T_{al}, used for design consistent with the values of T_{al} provided in the Qualified Products List (QPL) and consistent with the products approved for the particular wall system in this GDM. (GT)

v. Are the soil reinforcement - facing connection design parameters used consistent with the connection plan details provided? For steel reinforced systems, such details include the shear resistance of the connection pins or bolts, bolt hole sizes, etc. For geosynthetic reinforced systems, such details include the type of connection, and since the connection strength is specific to the reinforcement product (i.e., product material, strength, and type) – facing unit (i.e., material type and strength, and detailed facing unit geometry) combination, and the specific type of connector used, including material type and connector geometry, as well as how it fits with the facing unit. Check to make sure that the reinforcement – facing connection has been previously approved and that the approved design properties have been used. (GT, ST)

vi. If a coverage ratio, Rc, of less than 1.0 is used for the reinforcement, and its connection to the facing, has the facing been checked to see that it is structurally adequate to carry the earth load between reinforcement connection points without bulging of facing units, facing unit distress, or overstressing of the connection between the facing and the soil reinforcement? (GT, ST)

vii. Are the facing material properties used by the wall supplier consistent with what is required to produce a facing system that has the required design life and that is durable in light of the environmental conditions anticipated? Have these properties been backed up with appropriate supporting test data? Is the facing used by the supplier consistent with the aesthetic requirements for the project? (GT, ST)

i. Check to make sure that the following limit states have been evaluated, and that the wall/reinforced slope internal stability meets the design requirements:

i. Reinforcement resistance in reinforced backfill (strength and extreme event) (GT)

ii. Reinforcement resistance at connection with facing (strength and extreme event) (GT, ST)

iii. Reinforcement pullout (strength and extreme event) (GT)

iv. If the Stiffness Method is used, soil failure at the strength limit state (GT)

j. If obstructions such as small structure foundations, culverts, utilities, etc., must be placed within the reinforced backfill zone (primarily applies to MSE walls and reinforced slopes), has the design of the reinforcement placement, density and strength, and the facing configuration and details, to accommodate the obstruction been accomplished in accordance with the GDM, BDM, and AASHTO LRFD Specifications? (GT, ST)

k. Has the computer output for internal stability been hand checked to verify the accuracy of the computer program calculations (compare hand calculations to the computer output; also, a spot check calculation by the reviewer may also be needed if the calculations do not look correct for some reason)? (GT)

l. Have the specific requirements, material properties, and plan details relating to internal stability specified in the sections that follow in this Appendix for the specific wall/reinforced slope system been used? (GT, ST)

m. Note that for structural wall facings for MSE walls, design of prefabricated modular walls, and design of other structural wall systems, a structural design and detail review must be conducted by the structural reviewer (for WSDOT, the Bridge and Structures Office conducts this review in accordance with the BDM and the AASHTO LRFD Specifications). (ST)

    i. Compare preapproved wall details to the shop drawing regarding the concrete facing panel dimensions, concrete cover, rebar size, orientation and location. This also applies to any other structural elements of the wall (e.g., steel stiffeners for welded wire facings, concrete components of modular walls whether reinforced or not, etc.). (ST)

    ii. Is a quantity summary of components listed for each wall? (ST)

    iii. Do the geometry and dimensions of any traffic barriers or coping shown on shop drawings match with what is required by contract drawings (may need to check other portions of contract plans for verification (i.e. paving plans)? Has the structural design and sizing of the barrier/reaction slab been done consistently with the AASHTO specifications and BDM? Are the barrier details constructable? (ST)

    iv. Do notes in the shop drawings state the date of manufacture, production lot number, and piece mark be marked clearly on the rear face of each panel (if required by special the contract provisions)? (ST)

3. Wall/slope construction sequence and requirements provided in shop drawings

    a. Make sure construction sequence and notes provided in the shop drawings do not conflict with the contract specifications (e.g., minimum lift thickness, compaction requirements, construction sequence and details, etc.). Any conflicts should be pointed out in the shop drawing review comments, and such conflicts should be discussed during the precon meeting with the wall supplier, wall constructor, and prime contractor for the wall/slope construction. (GT, ST)

    b. Make sure any wall/slope corner or angle point details are consistent with the preapproved details and the contract requirements, both regarding the facing and the soil reinforcement. This also applies to overlap of reinforcement for back-to-back walls (GT, ST)

4. Wall and reinforced slope construction quality assurance

   a. Discuss all aspects of the wall/slope construction and quality assurance activities at the wall/reinforced preconstruction meeting. The preconstruction meeting should include representatives from the wall supplier and related materials suppliers, the earthwork contractor, the wall constructor, the prime contractor, the project inspection and construction administration staff, and the geotechnical and structural reviewers/designers. (GT, ST, and region project office)

   b. Check to make sure that the correct wall or reinforced slope elements, including specific soil reinforcement products, connectors, facing blocks, etc., are being used to construct the wall (visually check identification on the wall elements). For steel systems, make sure that reinforcement dimensions are correct, and that they have been properly galvanized. (region project office)

   c. Make sure that all wall elements are not damaged or otherwise defective. (region project office)

   d. Make sure that all materials certifications reflect what has been shipped to the project and that the certified properties meet the contract/design requirements. Also make sure that the identification on the wall elements shipped to the site match the certifications. Determine if the date of manufacture, production lot number, and piece mark on the rear face of each panel match the identification of the panels shown on the shop drawings (if req. by special prov.) (region project office)

   e. Obtain samples of materials to be tested, and compare test results to project minimum requirements. Also check dimensional tolerances of each wall element. (region project office)

   f. Make sure that the wall backfill meets the design/contract requirements regarding gradation, ability to compact, and aggregate durability. (region project office)

   g. Check the bearing pad elevation, thickness, and material to make sure that it meets the specifications, and that its location relative to the ground line is as assumed in the design. Also check to make sure that the base of the wall excavation is properly located, and that the wall base is firm. (region project office)

   h. As the wall is being constructed, make sure that the right product is being used in the right place. For soil reinforcement, make sure that the product is the right length, spaced vertically and horizontally correctly per the plans, and that it is placed and pulled tight to remove any slack or distortion, both in the backfill and at the facing connection. Make sure that the facing connections are properly and uniformly engaged so that uneven loading of the soil reinforcement at the facing connection is prevented. (region project office)

   i. Make sure that facing panels or blocks are properly seated on one another as shown in the wall details. (region project office)

j. Check to make sure that the correct soil lift thickness is used, and that backfill compaction is meeting the contract requirements. (region project office)

k. Check to make sure that small hand compactors are being used within 3 feet of the face. Reduced lift thickness should be used at the face to account for the reduced compaction energy available from the small hand compactor. The combination of a certain number of passes and reduced lift thickness to produce the required level of compaction without causing movement or distortion to the facing elements should be verified at the beginning of wall construction. For MSE walls, compaction at the face is critical to keeping connection stresses and facing performance problems to a minimum. Check to make sure that the reinforcement is not connected to the facing until the soil immediately behind the facing elements is up to the level of the reinforcement after compaction. Also make sure that soil particles do not spill over on to the top of the facing elements. (region project office)

l. Make sure that drainage elements are placed properly and connected to the outlet structures, and at the proper grade to promote drainage. (region project office)

m. Check that the wall face embedment is equal to or greater than the specified embedment. (region project office)

n. Frequently check to determine if wall face alignment, batter, and uniformity are within tolerances. Also make sure that acceptable techniques to adjust the wall face batter and alignment are used. Techniques that could cause stress to the reinforcement/facing connections or to the facing elements themselves, including shimming methods that create point loads on the facing elements, should not be used. (region project office)

o. For reinforced slopes, in addition to what is listed above as applicable, check to make sure that the slope facing material is properly connected to the soil reinforcement. Also check that secondary reinforcement is properly placed, and that compaction out to the slope surface is accomplished. (region project office)

Appendix 15-C Wall/Reinforced Slope Systems Evaluation: Submittal Requirements

15-C-1 Instructions

The submittal requirements outlined below are intended to cover multiple wall types. Some items may not apply to certain wall types. If a wall system has special material or design requirement not covered in the list below, the WSDOT Bridge Design Office and the WSDOT Geotechnical Office should be contacted prior to submittal to discuss specific requirements.

To help WSDOT understand the functioning and performance of the technology and thereby facilitate the Technical Audit, Applicants are urged to spend the time necessary to provide clear, complete and detailed responses. A response on all items that could possibly apply to the system or its components, even those where evaluation protocol has not been fully established, would be of interest to WSDOT. Any omissions should be noted and explained.

The submittal should be provided electronically to facilitate distribution within WSDOT for review purposes (e.g., as a PDF). Responses should be organized in the order shown and referenced to the given numbering system. Additionally, duplication of information is not needed or wanted. A simple statement referencing another section is adequate.

If the wall system has been reviewed and a report produced through the IDEA program or HITEC (if the HITEC report is still relevant to the submitted wall system), please indicate so and provide an electronic copy of the report(s). It is likely that much of what is contained in those reports will meet the submittal requirements provided below. If that is the case, please indicate that is the case, and indicate where in the IDEA or HITEC report the requested submittal information can be found.

15-C-2 Part One: Wall System Overview

Provide an overview of the wall system. Product brochures will usually fulfill the requirements of this section.

15-C-3 Part Two: Plan Details

As a minimum, provide the following plan sheet details:

1. All system component details.
2. Typical plan, profile, and section views.
3. Details that show the facing batter(s) that can be obtained with the wall system (example details that illustrate the permissible range are acceptable).
4. Corner details
    - Acute inside corner
    - Obtuse inside corner
    - Orthogonal inside corner
    - Obtuse outside corner
    - Orthogonal outside corner

5. Radius Details (inside and outside radii, include system limitations).
   - Inside radii
   - Outside radii
   - System limitations for inside and outside radii

6. Traffic barrier systems
   - Guardrail
   - Moment slab barrier

7. Horizontal obstruction details for obstructions
   - Horizontal obstructions up to 24 inches oriented parallel to the wall face
   - Horizontal obstructions up to 48 inches oriented perpendicular to the wall face

8. Vertical obstruction details for obstructions up to 48 inches.

9. Culvert Penetration
   - Up to 48 inch culverts oriented perpendicular to the wall face.
   - Up to 24 inch culverts oriented up to a 45 degree skew angle as measured from perpendicular to the wall face.

10. Leveling pad details in accordance with Section 6-13 of the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction*.
    - Minimum dimensions
    - Steps
    - Corners

11. Coping and gutter details.

All plan sheet details should be provided as 11×17 size, hard or electronic copies. All dimensions shall be given in English Units (inches and feet). The plan sheet shall as a minimum identify the wall system, an applicable sheet title, the date the plan sheet was prepared, and the name of the engineer and company responsible for its preparation.

15-C-4 Part Three: Materials and Material Properties

WSDOT has established material requirements for certain non-proprietary wall components. These requirements are described in the *Standard Specifications for Road, Bridge, and Municipal Construction*, and General Special Provisions (GSP) available at www.wsdot.wa.gov/design/projectdev/gspamendments.htm. Specifically, GSP 130201. GB6 covers welded wire faced structural earth wall materials, GSP 130202.GB covers precast concrete panel faced structural earth wall materials, and GSP 130203. GB6 covers concrete block faced structural earth wall materials. All wall components falling into the categories currently defined by WSDOT should meet the WSDOT material requirements.

For materials not currently covered by WSDOT specifications, provide material specifications describing the material type, quality, certifications, lab and field testing, acceptance and rejection criteria along with support information for each material items. Include representative test results (lab and/or field) clearly referencing the date, source and method of test, and, where required, the method of interpretation and/or extrapolation. Along with the source of the supplied information, include a listing of facilities normally used for testing (i.e., in-house and independent).

All geosynthetic reinforced wall systems shall use a soil reinforcement product listed in the WSDOT Qualified Product List (QPL). Inclusion of geosynthetic reinforcement products on the QPL will be a necessary prerequisite to wall system approval.

1. For facing units, provide the following information:
   - Standard dimensions and tolerances
   - Joint sizes and details
   - Facing unit to facing unit shear resistance
   - Bearing pads (joints)
   - Spacers
   - Connectors (pins, etc.)
   - Joint filler requirements: geotextile or graded granular
   - Other facing materials, such as for reinforced slopes, or other materials not specifically identified above

2. For the soil reinforcement (applies to structural earth walls and reinforced slopes), provide the following information:
   - Manufacturing sizes, tolerances, lengths
   - Ultimate and yield strength for metallic reinforcement
   - Corrosion resistance test data for metallic reinforcement (for metallic materials other than those listed in the GSP's)
   - Pullout interaction coefficients for WSDOT Gravel Borrow (*Standard Specification* Section 9-03.14(4)), or similar gradation, if default pullout requirements in the AASHTO *LRFD Bridge Design Specifications* are not used or are not applicable.

3. For the connection between the facing units and the soil reinforcements (applies to structural earth walls and reinforced slopes), provide the following information:
   - Photographs/drawings that illustrate the connection
   - Ultimate connection strength, $T_{ultconn}$, at various confining pressures up to the anticipated preapproved wall height (typically 33 ft or less) for each reinforcement product, connection type, and facing unit, and connection test specific reinforcement strength, T_{lot}, for all connection tests.
   - Provide connection data in an editable format using the table below:

Facing Unit	Geogrid Product	Wall Height, H (ft)	Normal Load, N (lbs/ft)	$T_{ultconn}$ (lbs/ft)	T_{lot} (lbs/ft)
		Provide range of H for which each $T_{ultconn}$ equation applies	Provide range of N for which each $T_{ultconn}$ equation applies	Provide regression equation(s) here	

4. For the coping, provide the following information:
   - Dimensions and tolerances
   - Material used (including any reinforcement)
   - Method/details to attach coping to wall top

5. For the traffic railing/barrier, provide the following information:
   - Dimensions of precast and cast-in-place barriers and reaction slabs
   - How barrier/railing is placed on/in and/or attached to wall top
   - How guard railing is placed on/in and/or attached to wall top

6. Regarding the quality control/quality assurance of the wall system material suppliers, provide the following information:
   - QC/QA for metallic or polymeric reinforcement
   - QC/QA for facing materials and connections
   - QC/QA for other wall components
   - Backfill (unit core fill, facing backfill, etc.)

15-C-5 Part Four: Design

Walls shall be designed in conformance with the WSDOT *Geotechnical Design Manual* (GDM), LRFD *Bridge Design Manual* (BDM), and the AASHTO *LRFD Bridge Design Specifications*. Provide design assumptions and procedures with specific references (e.g., design code section) for each of the design requirements listed below. Clearly show any deviations from the GDM, LRFD BDM and the AASHTO *LRFD Bridge Design Specifications*, along with theoretical or empirical information which support such deviations. In general, proprietary wall suppliers will only be responsible for internal stability of their wall system. However, if there are any special external stability considerations for the wall system, those special considerations should be identified and explained in the wall system submittal.

Provide detailed design calculations for a 25 feet high wall with a 2H:1V sloping soil surcharge (extending from the back face of the wall to an infinite distance behind the wall). The calculations should address the technical review items listed below. The calculations shall include detailed explanations of any symbols, design input, materials property values, and computer programs used in the design of the walls. The example designs shall be completed with seismic forces (assume a PGA of 0.50g). In addition, a 25 feet high example wall shall be performed with no soil surcharge and a traffic barrier placed on top of the wall at the wall face. The barrier is to be of the “F shape” and “single slope” configuration and capable of resisting a TL-4 loading in accordance with LRFD BDM Section 10.2.1 for barrier height and test level requirement. With regard to the special plan details required in Section 2, provide an explanation of how the requirements in the GDM, LRFD BDM, and the AASHTO *LRFD Bridge Design Specifications* will be applied to the design of these details, including any deviations from those design standards, and any additional design procedures not specifically covered in those standards, necessary to complete the design of those details. This can be provided as a narrative, or as example calculations in addition to those described earlier in this section.

For internal stability design, provide design procedures, assumptions, and any deviations from the design standards identified above required to design the wall or reinforced system for each of the design issues: listed below. Note that some of these design issues are specific to structural earth wall or reinforced slope design and may not be applicable to other wall types.

1. Assumed failure surface used for design
2. Distribution of horizontal stress
3. How surcharge loads are handled in design
   - Concentrated dead load
   - Sloped surcharge
   - Broken-back surcharge
   - Live load
   - Traffic impact
4. Determination of the long-term tensile strength of reinforcement
5. Pullout design of soil reinforcement or facing components that protrude into wall backfill
6. Determination of vertical and horizontal spacing of soil reinforcements (including traffic impact requirements)
7. Facing design
   - Connections between facing units and components
   - Facing unit strength requirements
   - Interface shear between facing units
   - Connections between facing and soil reinforcement/reinforced soil mass
   - How facing batter is taken into account for the range of facing batters available for the system
   - Facing compressibility/deformation, if a flexible facing is used
8. Seismic design considerations
9. Design assumptions/parameters for assessing mobilization of backfill weight internal to wall system (primarily applies to prefabricated modular walls as defined in the AASHTO *LRFD Bridge Design Specifications*)

List all wall/slope system design limitations, including:

- Seismic loading
- Environmental constraints
- Wall height
- External loading
- Horizontal and vertical deflection limits
- Tolerance to total and differential settlement
- Facing batter
- Other

Computer Support:

If a computer program is used for design or distributed to customers, provide representative computer printouts of design calculations for the above typical applications demonstrating the reasonableness of computer results. All computer output submitted shall be accompanied by supporting hand calculations detailing the calculation process. If MSEW 3.0, or later version, is used for the wall design, hand calculations supporting MSEW are not required.

Quality Control/Quality Assurance for design of the wall/slope systems:

Include the system designer's Quality Assurance program for evaluation of conformance to the wall supplier's quality program.

15-C-6 Part Five: Construction

Provide the following information related to the construction of the system:

1. Provide a documented field construction manual describing in detail and with illustrations as necessary the step-by-step construction sequence, including requirements for:
   - Foundation preparation
   - Special tools required
   - Leveling pad
   - Facing erection
   - Facing batter for alignment
   - Steps to maintain horizontal and vertical alignment
   - Retained and backfill placement/compaction
   - Erosion mitigation
   - All equipment requirements

2. Include sample construction specifications, showing field sampling, testing and acceptance/rejection requirements. Provide sample specifications for:
   - Materials
   - Installation
   - Construction

3. Quality Control/Quality Assurance of Construction:

   Describe the quality control and quality assurance measurements required during construction to assure consistency in meeting performance requirements.

15-C-7 Part Six: Performance

Provide the following information related to the performance of the system:

1. Provide a copy of any system warranties.
2. Identify the designated Responsible Party for:
   - System performance
   - Material performance
   - Project-specific design (in-house, consultant)
3. List insurance coverage types (e.g., professional liability, product liability, performance) limits, basis (i.e., per occurrence, claims made) provided by each responsible party
4. Provide a well documented history of performance (with photos, where available), including:
   - Oldest
   - Highest
   - Projects experiencing maximum measure settlement (total and differential)
   - Measurements of lateral movement/tilt
   - Demonstrated aesthetics
   - Project photos
   - Maintenance history
5. Provide the following types of field test results, if available:
   - Case histories of instrumented structures
   - Construction testing
   - Pullout testing
6. Regarding construction/in-service structure problems, provide case histories of structures where problems have been encountered, including an explanation of the problems and methods of repair.
7. Provide a list of state DOT's that have used this wall system, including contact persons, addresses and telephone numbers.

Appendix 15-D Preapproved Proprietary Wall Systems

The following wall systems are preapproved for use in WSDOT projects:

Table 15-D-1 Preapproved Proprietary Walls

Wall Supplier	System Name and *Appendix Location*	System Description and Appendix Location	ASD/ LFD or LRFD? [1]	Height, or Other Limitations	Year Initially Approved	Last Approved Update
Hilfiker Retaining Walls 1902 Hilfiker Lane Eureka, CA 95503-5711 707-443-5093	Welded Wire Retaining Wall Appendix 15-H	Welded wire facing that is continuous with welded wire soil reinforcement	ASD/LFD	33 feet	Unknown	Approved 11/9/04 (submitted 9/15/03)
Hilfiker Retaining Walls 1902 Hilfiker Lane Eureka, CA 95503-5711 707-443-5093	Eureka Reinforced Soil Wall Appendix 15-I	Precast concrete 5'×5' facing panels and welded wire mat soil reinforcement	ASD/LFD	33 feet	Unknown	Approved 11/9/04 (submitted 10/5/04)
The Reinforced Earth Co. 9025 East Kenyon Ave. Suite 200 Denver, CO 80237 303-790-1481	Reinforced Earth Wall Appendix 15-J	Precast concrete 5'×5' facing panels and steel strip soil reinforcement	LRFD	33 feet	1987	Approved 11/9/04 (submitted 3/29/04)
Tensar Earth Technologies, Inc. 2500 Northwinds Parkway Suite 500 Alpharetta, GA 30009 770-344-2090	ARES Wall Appendix 15-K	Precast concrete 5'×5' facing panels and Tensar geogrid soil reinforcement	ASD/LFD	33 feet	1998	Approved 11/9/04 (submitted 8/6/04)
Tensar Earth Technologies, Inc. 2500 Northwinds Parkway Suite 500 Alpharetta, GA 30009 770-344-2090	MESA Wall Appendix 15-L	Modular dry cast concrete block facing with Tensar geogrid soil reinforcement	ASD/LFD	33 feet	2000	Approved 11/9/04 (submitted 4/19/04 and 9/22/04)
Tensar Earth Technologies, Inc. 2500 Northwinds Parkway Suite 500 Alpharetta, GA 30009 770-344-2090	Welded Wire Form Wall Appendix 15-M	Tensar geogrid wrapped face wall with welded wire facing form	ASD/LFD	33 feet*	2006	Approved 3/3/06 (submitted 11/26/05)
SSL, LLC 4740 Scotts Valley Dr., Suite E Scotts Valley, CA 95066 831-430-9300	MSEPlus Wall Appendix 15-N	Precast concrete 5'x5' facing panels and steel welded wire strip soil reinforcement	LRFD	33 feet	1999	Approved 8/5/13 (submitted 5/28/13)

*If the vegetated face option is used for the Hilfiker Welded Wire Retaining Wall or the Tensar Welded Wire Form Wall, the maximum wall height shall be limited to 20 feet. Greater wall heights for the vegetated face option for these walls may be used on a case by case basis as a special design if approved by the State Geotechnical Engineer and the State Bridge Engineer.

[1] For those systems still identified as ASD/LFD, use of the current AASHTO LRFD Bridge Design Specifications is preferred.

Table 15-D-1 Preapproved Proprietary Walls

Wall Supplier	System Name and *Appendix Location*	System Description and Appendix Location	ASD/ LFD or LRFD? [1]	Height, or Other Limitations	Year Initially Approved	Last Approved Update
Anchor Wall Systems, Inc. 5959 Baker Rd, Suite 390 Minnetonka, MN 55345-5996 952-933-8855	Landmark Appendix 15-O	Modular dry cast concrete block facing with Miragrid geogrid soil reinforcement	LRFD	33 feet	2012	Approved 4/2/12 (submitted 10/21/11)
Allan Block Corporation 7424 W. 78th St. Bloomington, MN 55439 952-835-5309	Allan Block Wall (battered face) Appendix 15-P	Modular dry cast concrete block facing with Miragrid or Stratagrid geogrid soil reinforcement	LRFD	33 feet	2009	Approved 7/15/09 (submitted 1/15/08)
Redi-Rock International LLC 05481 US 31 South Charlevoix, MI 49720 866-222-8400	Redi-Rock PC (Positive Connection) Wall Appendix 15-Q	Precast concrete block facing with Miragrid strip soil reinforcement	LRFD	33 feet	2015	Approved 8/3/15 (submitted Aug. 2012)
Lock and Load Retaining Walls LTD 1681 Chestnut St., Suite 400 Vancouver, BC V6J 4M6 Canada 604-732-9990	Lock and Load Wall Appendix 15-R	Precast concrete panel facing attached to wrapped face geogrid wall	LRFD	33 feet	2013	Approved 7/10/13 (submitted 5/3/13)
Keystone Retaining Wall Systems, LLC 4444 West 78th St. Minneapolis, MN 55435 800-747-8971	Keystone Keygrid (Compac II and III Units) Appendix 15-S	Modular dry cast concrete block facing with Miragrid geogrid soil reinforcement	LRFD	33 feet	2015	Approved 8/3/15 (submitted 4/17/15)
Basalite Concrete Products, LLC 3299 International Place Dupont, WA 98327-7707 253-964-5000	GEOWALL Structural Earth Retaining Wall Appendix 15-T	Modular dry cast concrete block facing with Miragrid or Stratagrid geogrid soil reinforcement	LRFD	33 feet	2018	Approved 1/2/18 (submitted 3/4/17)

*If the vegetated face option is used for the Hilfiker Welded Wire Retaining Wall or the Tensar Welded Wire Form Wall, the maximum wall height shall be limited to 20 feet. Greater wall heights for the vegetated face option for these walls may be used on a case by case basis as a special design if approved by the State Geotechnical Engineer and the State Bridge Engineer.

[1] For those systems still identified as ASD/LFD, use of the current AASHTO LRFD Bridge Design Specifications is preferred.

Appendix 15-E MSE Wall Design Using the Stiffness Method

15-E-1 Summary of the Stiffness Method and Notations

Table 15-E-1 provides a summary of how to calculate each of the parameters in the Stiffness Method, including coefficient values, based on the method details provided by Allen and Bathurst (2015, 2018). The Stiffness Method equation is repeated below for convenience:

$$T_{max} = S_v \left[H \gamma_r D_{tmax} + \left(\frac{H_{ref}}{H} \right) S \gamma_f \right] K_{avh} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c \tag{15-E-1}$$

where,

T_{max}	=	maximum load in the soil reinforcement away from the facing connection (kips/ft)
K_{avh}	=	active earth pressure coefficient
S_v	=	tributary area (equivalent to the vertical spacing of the reinforcement in the vicinity of each layer when analyses are carried out per unit length of wall) (ft)
H	=	total wall height (ft)
H_{ref}	=	reference height = 20 ft
S	=	average surcharge height above wall within 0.7H of the wall face (ft)
γ_r	=	unit weight of wall backfill soil (kcf)
γ_f	=	unit weight of surcharge soil (kcf)
D_{tmax}	=	T_{max} distribution factor
Φ_g	=	global stiffness factor
Φ_{fs}	=	facing stiffness factor
Φ_{fb}	=	facing batter factor
Φ_{local}	=	local stiffness factor
Φ_c	=	soil cohesion factor

Table 15-E-2 provides a recommended approach to address any soil cohesion that may be present in the wall backfill, as well as what to do if soil shear strength data for the backfill to be used is not available. Note that in WSDOT experience, if Gravel Borrow that meets the requirements in Section 9-03.14(4) of the *Standard Specifications for Road, Bridge, and Municipal Construction* M 41-10 is used as the wall backfill, backfill friction angles are usually at or above 38°, and 38° may be used without backfill specific shear strength tests on WSDOT projects in this case (see Table 5-2 in GDM Chapter 5).

Cohesive shear strength of the MSE wall backfill shall not be used for final design (other than as illustrated in Example 5 at the end of this appendix), and MSE wall backfill that has significant soil cohesion should be avoided, as soil cohesion can be lost over time after wall construction and can also significantly reduce the ability of the wall backfill to drain as water percolates into it. This potential post-construction loss of cohesion over time as well as increase in the amount of water stored in the backfill can cause post-construction reinforcement load and deformation increases. The Stiffness Method can be used to estimate the reinforcement load and deformation increases that could occur post-construction as soil cohesion is lost. See Example 5 at the end of this appendix for an illustration of the effect of lost cohesion after wall construction on reinforcement strains.

Table 15-E-1 Summary of equations, parameters, and coefficients for the Stiffness Method (Allen and Bathurst 2018)

Parameter	Name	Equation	Coefficient	Value
D_{tmax}	T_{max} distribution factor	$z_b = C_h \times (H)^y \times \Phi_{fb}$ For $z < z_b$: $D_{tmax} = D_{tmax0} + (z/z_b) \times (1 - D_{tmax0})$ For $z \geq z_b$: $D_{tmax} = 1.0$	C_h (for H in m) C_h (for H in ft) y D_{tmax0}	0.40 0.32 1.2 0.12
Φ_g	Global stiffness factor	$\Phi_g = \alpha\left(\frac{S_{global}}{p_a}\right)^{\beta}$	α β	0.16 0.26
S_{global}	Global reinforcement stiffness	$S_{global} = \frac{J_{ave}}{(H/n)} = \frac{\sum_{i=1}^{n} R_c J_i}{H}$		
Φ_{local}	Local stiffness factor	$\Phi_{local} = \left(\frac{S_{local}}{S_{localave}}\right)^{a}$	"a" for steel "a" for geosynthetic and extensible steel grids [b]	0 0.5
S_{local}	Local reinforcement stiffness	$S_{local} = R_C J_i / S_v$		
$S_{localave}$	Average local reinforcement stiffness	$S_{localave} = \frac{\sum_{i=1}^{n} (R_c J_i / S_v)}{n}$		
Φ_{fb}	Facing batter factor	$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}}\right)^{d}$	d	0.4
Coefficient of active earth pressure		$K_{abh} = \frac{cos^2(\phi_r + \omega)}{cos^3 \omega \left(1 + \frac{sin\phi_r}{cos\omega}\right)^2}$		
Φ_{fs}	Facing stiffness factor	$\Phi_{fs} = \eta\left(\left(\frac{S_{global}}{P_a}\right) F_f\right)^{\kappa}$	η κ	0.57 0.15
F_f	Facing stiffness parameter	$F_f = \frac{1.5 H^3 P_a}{E b^3 (h_{eff}/H)}$		
Φ_c	Soil cohesion factor	$\Phi_c = e^{\lambda(c/(\gamma_r H))}$	λ	-16

Notes:

[a] see Allen and Bathurst (2015)

[b] e.g., crimped longitudinal steel wire

Other Notation in Table 15-E-1:

T_{max}	=	the maximum load in the reinforcement (force/unit running length of wall – e.g. (lbs/ft))
n	=	number of reinforcement layers
H	=	height of wall (ft)
H_{ref}	=	reference wall height = 20 ft
S_v	=	tributary vertical spacing of the reinforcement layer (ft)
b	=	thickness of the facing column (ft)
E	=	elastic modulus of the “equivalent elastic beam” representing the wall face (ksf)
p_a	=	atmospheric pressure (101 kPa or 2.11 ksf)
h_{eff}	=	equivalent height of an un-jointed facing column that is approximately 100% efficient in transmitting moment through the height of the facing column (ft)
K_{abh}	=	horizontal component of active earth pressure coefficient accounting for wall face batter
K_{avh}	=	horizontal component of active earth pressure coefficient assuming the wall is vertical (ω = 0)
J_i	=	secant tensile creep stiffness of geosynthetic reinforcement layer i at 2% strain and 1000 hours on a per unit of reinforcement width basis from laboratory testing (kips/ft)
J_{ave}	=	average secant tensile creep stiffness corrected for the coverage ratio, i.e., R_cJ_i, of all “n” geosynthetic reinforcement layers (kips/ft)
R_c	=	reinforcement coverage ratio
σ_v	=	vertical pressure due to gravity forces from self-weight of the reinforced soil and soil above the reinforced wall backfill (ksf)
c	=	soil cohesion (ksf)
γ_r	=	unit weight of the reinforced soil (kcf)
γ_f	=	unit weight of the soil surcharge (kcf)
q	=	$S\gamma_f$ = average vertical pressure due to soil surcharge on the top of the reinforced soil mass up to a maximum width of 70% of the wall height H (ksf)
z	=	depth below wall top measured at the back of the facing (ft)
K_a	=	active earth pressure coefficient
S	=	average soil surcharge depth above the wall top using a soil surcharge unit weight γ_f (ft)
ϕ_r	=	friction angle of the reinforced soil backfill (degrees)
ω	=	wall face batter in clockwise direction from the vertical (degrees). In AASHTO (2020) the face batter θ is taken clockwise from the horizontal, hence $\omega = \theta - 90°$

Table 15-E-2 Soil shear strength parameters recommended for design using the Stiffness Method (after Allen and Bathurst 2018)

Cohesive strength component deduced from failure envelope	Plasticity Index PI	Used to calculate K_{avh} and K_{ahb}		Value of c used to calculate Φ_c	Cohesion factor Φ_c	Comments
		ϕ_r	c			
c = 0	NA	ϕ_{tx} or ϕ_{ds}	0	0	1	If backfill soil strength properties are unknown, use conservative default value for ϕ_r
c > 0 (curved Mohr-Coulomb envelope due to particle interlocking)	≤ 6	ϕ_{tx} or ϕ_{ds} ⇨ ϕ_{sec}	0	0	1	If uncertain that matric suction is contributing to the cohesion intercept in soil shear test results, assume c = 0. If backfill soil is unknown at time of design, use conservative default value for ϕ_r
c > 0 (apparent cohesion due to matric suction)	≤ 6	ϕ_{tx} or ϕ_{ds}	0	0	1	Always assume c = 0, unless evaluating the influence of post-construction loss of matric suction on reinforcement loads
c > 0 (true cohesion)	> 6	ϕ_{tx} or ϕ_{ds}	0	> 0	< 1	If uncertain that soil cohesion will persist for design lifetime, assume c = 0. To investigate possible loss of cohesive shear strength component over life of wall, compare T_{max} using c > 0 with T_{max} using c = 0

Notes: PI = Plasticity Index, ϕ_r = peak friction angle for reinforced soil backfill, ϕ_{tx} = peak friction angle from triaxial test, ϕ_{ds} = peak friction angle from direct shear test, ϕ_{sec} = peak secant friction angle (determined as shown in Allen and Bathurst (2015, 2018).

15-E-2 Limit State Equations for Design

Limit states that need to be considered when doing internal stability design using the Stiffness Method include soil failure as a Service Limit State, and reinforcement strength, connection strength, and pullout as Strength and Extreme Event Limit States. The load and resistance factors applicable to the Stiffness Method for these limit states are provided in Section 15.5.3.10.2.

15-E-2.1 Soil Failure Limit State Design

Research indicates that if the average peak reinforcement strain in the wall exceeds approximately 2.5 to 3%, for typical granular backfill materials, soil failure as defined can be achieved (Allen et al. 2003; Allen and Bathurst 2013, and Allen and Bathurst 2015, 2018). However, AASHTO (2020) has limited the target reinforcement strain to 2% for stiff faced walls and 2.5% for flexible faced walls.

The soil failure limit state should be considered a service limit state for design because it is a deformation criterion. Furthermore, if this criterion is substantially exceeded the structure will not collapse but will more likely develop progressive increases in facing deformation. The soil failure limit state must be evaluated if the Stiffness Method is used to compute geosynthetic reinforcement loads (as well as for extensible steel grid) for working stress (operational) conditions. The goal of this limit state is to ensure that the factored reinforcement strain in any layer is less than the target maximum peak strain to prevent exceedance of the soil failure limit state. To calculate the reinforcement strain ε_{rein} in individual layers, see Equation 15-14 in Section 15.5.3.10.3.1.

If ε_{rein} in any individual layer exceeds the limit strain ε_{mxmx}, or if the target strain for the average ε_{rein} for all of the layers in the wall section is exceeded, then another product(s) with higher stiffness must be selected and this limit state checked again. For the same product line, increasing stiffness is associated with increasing T_{ult} values as can be seen in NTPEP reports (e.g., NTPEP 2019).

For design purposes, reinforcement used in the wall would be selected based on the tensile strength required to prevent reinforcement rupture and connection failure, and also selected based on the minimum reinforcement stiffness required in all the reinforcement layers to prevent the development of a contiguous shear surface through the reinforced soil zone.

In general, the soil failure limit state should be checked first, as this limit state often controls the amount of reinforcement required. The stiffness values coming out of that limit state analysis should then be used to determine T_{max} for reinforcement and connection rupture, and pullout. For systems with very poor connection strength, it is possible that connection strength could control design instead. If that is the case, the soil failure limit state may need to be reassessed to make sure that the reinforcement creep stiffness is consistent with the ultimate tensile strength needed. See Allen and Bathurst (2019) for information on the correlation between tensile strength and creep stiffness, as well as AASHTO NTPEP (2019) for product line specific correlations between tensile strength and creep stiffness.

15-E-2.2 Reinforcement Strength Design

The tensile strength reduction factor for a reinforcement product in a geosynthetic reinforced soil wall is computed as:

$$RF = RF_{ID} \times RF_{CR} \times RF_D \tag{15-E-3}$$

where,

RF_{ID} = installation damage reduction factor,

RF_{CR} = creep reduction factor, and

RF_D = durability reduction factor.

These reduction factors shall be determined in accordance with AASHTO R 69. Product specific data that can be used to assess the reduction factors can be obtained at NTPEP (2019).

The long-term (nominal) design strength is:

$$T_{al} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_{ID} RF_{CR} RF_D} \tag{15-E-4}$$

where,

T_{ult} = ultimate tensile strength of the reinforcement

The calibration of the load and resistance factors for the Stiffness Method assumes that the Minimum Average Roll Value (MARV) of the ultimate tensile strength is used for design to obtain T_{al}.

Equation 15-E-1 is the equation to calculate the unfactored reinforcement load T_{max} in each reinforcement layer using the Stiffness Method. The factored limit state design equation for tensile rupture for the case of dead loads only is expressed as:

$$\gamma_{p\text{-}EV}\, T_{max} < \phi_{rr} T_{al} R_c \tag{15-E-5}$$

where,

ϕ_{rr} = the resistance factor for reinforcement rupture.

All parameters are as defined previously.

Combining equations 15-E-1 and 15-E-5 for the case of dead loads only leads to:

$$T_{al}(required) = \left(\frac{\gamma_{p-EV}}{\varphi_{rr} R_c}\right) T_{max} = \left(\frac{\gamma_{p-EV}}{\varphi_{rr} R_c}\right) S_v \left[H \gamma_r D_{tmax} + \left(\frac{H_{ref}}{H}\right) S \gamma_f\right] K_{avh}\, \Phi_{fb}\, \Phi_g\, \Phi_{fs}\, \Phi_{local}\, \Phi_c \tag{15-E-6}$$

where,

T_{al} (required)

= the required minimum (factored) long-term reinforcement strength to resist the factored loads.

The equivalent expression for the case of an additional live load LL is:

$$T_{al}(required) = \left(\frac{\gamma_{p-EV}}{\varphi_{rr} R_c}\right) T_{max} = \left(\frac{\gamma_{EV}}{\varphi_{rr} R_c}\right) S_v \left[H \gamma_r D_{tmax} + \left(\frac{H_{ref}}{H}\right) S \gamma_f + LL \left(\frac{\gamma_{LL}}{\gamma_{p-EV}}\right)\right] K_{avh} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c \tag{15-E-7}$$

where,

LL	=	live load (kPa),
γ_{LL}	=	live load factor = 1.75.

All other factors are as previously defined. For other dead load scenarios such as footings with finite surface areas, conventional (elastic) solutions can be used and the resulting factored horizontal load added to the right-hand side of equations 15-E-5 and 15-E-6 as shown below:

$$T_{al}(required) = \frac{\left(\gamma_{p-EV} T_{max} + \gamma_{p-ES} S_v (K_a \Delta\sigma_v + \Delta\sigma_H\right)}{(\varphi_{rr} R_c)} = \left(\frac{\gamma_{p-EV}}{\varphi_{rr} R_c}\right) S_v \left[H \gamma_r D_{tmax} + \left(\frac{H_{ref}}{H}\right) S \gamma_f + LL \left(\frac{\gamma_{LL}}{\gamma_{p-EV}}\right)\right] K_{avh} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c + \frac{(\gamma_{p-ES} S_v (K_a \Delta\sigma_v + \Delta\sigma_H)}{(\varphi_{rr} R_c)} \tag{15-E-8}$$

where,

γ_{p-EV}	=	load factor for vertical earth pressure specified in Table 15-5 (dim.)
γ_{p-ES}	=	load factor for earth surcharge (ES) in the *AASHTO LRFD Bridge Design Manual*, Table 3.4.1-2
$\Delta\sigma_v$	=	vertical soil stress due to concentrated load such as a footing load (ksf)
$\Delta\sigma_H$	=	horizontal stress at reinforcement level resulting from a concentrated horizontal surcharge load (ksf)
S_v	=	tributary layer vertical thickness for reinforcement (ft)
K_a	=	active lateral earth pressure coefficient (dim)

However, the soil failure limit state would also need to be checked for the combined loading. If superposition principles are used to determine that combined loading, the soil failure limit state will become excessively conservative for footing load that are typical for bridges. Therefore, if designing an MSE bridge abutment, due to the high footing load that is likely, it is best to use the Simplified Method instead to do that design.

15-E-2.3 Connection Strength Design

AASHTO (2020) specifies that T_o is equal to $1.0 \times T_{max}$, although T_o could be significantly greater or less than T_{max}. In the absence of a method based on measured data, the AASHTO (2020) approach should be used, except that T_{max} is determined using the Stiffness Method. For design purposes, the minimum connection strength required is compared to the long-term connection strength available.

The reinforcement connection strength limit state equation is as follows:

$$\gamma_{p\text{-}EVc} T_o = \phi_{cr} T_{ac} R_c \tag{15-E-9}$$

where,

T_{ac}	=	nominal long-term reinforcement/facing connection strength per unit of wall width (kips/ft)
$\gamma_{p\text{-}EVc}$	=	connection load factor,
R_c	=	the reinforcement coverage ratio,
ϕ_{cr}	=	connection resistance factor for rupture or pullout of the reinforcement at the connection to the wall face, and
T_o	=	the reinforcement load at the connection, which is equal to $1.0T_{max}$, and T_{max} is determined using the Stiffness Method.

For geosynthetic block-faced walls, the reference (short-term) ultimate connection strength ($T_{ultconn}$) is determined from straight-line approximations to different ranges of normal load (or stress) applied to the connection system from the results of a standard laboratory testing protocol such as ASTM D6638 (2011), hence:

$$\begin{aligned} T_{ultconn} &= c_{conn} + N\, tan\phi_{conn} \\ &= c_{conn} + (b\sigma_n) tan\phi_{conn} = c_{conn} + (b[\gamma_{bk} \times z]) tan\phi_{conn} \end{aligned} \tag{15-E-10}$$

where,

c_{conn}	=	the vertical axis intercept (e.g., units of kips/ft) on a plot of connection capacity versus normal load N (e.g., units of kips/ft) or stress σ_n (in ksf) acting at the connection due to the facing column,
b	=	toe to heel dimension of the block,
γ_{bk}	=	the unit weight of the infilled block,
z	=	the depth of the connection below the crest of the wall (assuming the wall is vertical), and
ϕ_{conn}	=	the slope of the failure envelope line segment.

For many systems, the line segment for the normal load of interest may be horizontal (hence, $c_{conn} > 0$ and $\phi_{conn} = 0$) (Bathurst and Simac 1993).

T_{ac} is determined as follows:

$$T_{ac} = \frac{T_{ult} \times CR_{cr}}{RF_D} \tag{15-E-11}$$

where,

T_{ult}	=	minimum average roll value (MARV) ultimate tensile strength of soil reinforcement (kips/ft)
CR_{cr}	=	long-term connection strength reduction factor to account for reduced ultimate strength resulting from connection (dim.)
RF_D	=	reduction factor to prevent rupture of reinforcement due to chemical and biological degradation (dim.)

CR_{cr} is determined using RF_{CR} to reduce the short-term (i.e., ultimate) connection strength $T_{ultconn}$ to account for creep of the geosynthetic at the connection, or it may be based on long-term connection creep tests. If connection creep tests are not conducted, CR_{cr} shall be based on short-term connection tests and shall be determined as follows:

$$CR_{cr} = \frac{T_{ultconn}}{(RF_{CR} T_{lot})} \tag{15-E-12}$$

where,

$T_{ultconn}$	=	nominal short-term connection strength (lbs/ft)
RF_{CR}	=	strength reduction factor to prevent long-term creep rupture of reinforcement (dim.)
T_{lot}	=	ultimate wide width tensile strength (ASTM D4595 or D6637) of the geosynthetic material used in the connection tests (lbs/ft)

If traffic live load is present and treated as an equivalent uniformly distributed surface pressure, then the minimum T_{ac} required is:

$$T_{ac}(\text{required}) = \frac{T_{ultconn}}{(RF_D \times RF_{CR})} \geq \left(\frac{\gamma_{con}}{\varphi_{cr} R_c}\right)\left(T_o + LL\left(\frac{\gamma_{LL}}{\gamma_{con}}\right) S_v K_{avh} \Phi_g \Phi_{fb} \Phi_{fs} \Phi_{local} \Phi_c\right) \tag{15-E-13}$$

The value of T_{ult} to satisfy this requirement is therefore:

$$\begin{aligned} T_{ult}(required) &= \frac{T_{ultconn}}{CR_u} \\ &\geq \left(\frac{\gamma_{con}}{\varphi_{cr} R_c}\right)\left(\frac{RF_D \times RF_{CR}}{CR_u}\right)\left(T_o + LL\left(\frac{\gamma_{LL}}{\gamma_{con}}\right) S_v K_{avh} \Phi_g \Phi_{fb} \Phi_{fs} \Phi_{local} \Phi_c\right) \end{aligned} \tag{15-E-14}$$

All variables are defined previously. For other types of connections, minor modifications to these equations may be needed; see AASHTO (2020) for guidance on handling other facing connection systems.

15-E-2.4 Pullout Resistance Limit State Design

The following equations are used for the pullout resistance limit state to estimate the required reinforcement length in the anchorage zone beyond the active zone boundary:

$$\gamma_{p\text{-}EV} T_{max} = \phi_{po} P_c \tag{15-E-15}$$

where,

P_c	=	nominal calculated pullout resistance, and
ϕ_{po}	=	resistance factor applicable to pullout resistance.

Other variables are defined previously.

P_c is calculated as:

$$P_c = C(F^*\alpha)\sigma_v L_e R_c \tag{15-E-16}$$

where,

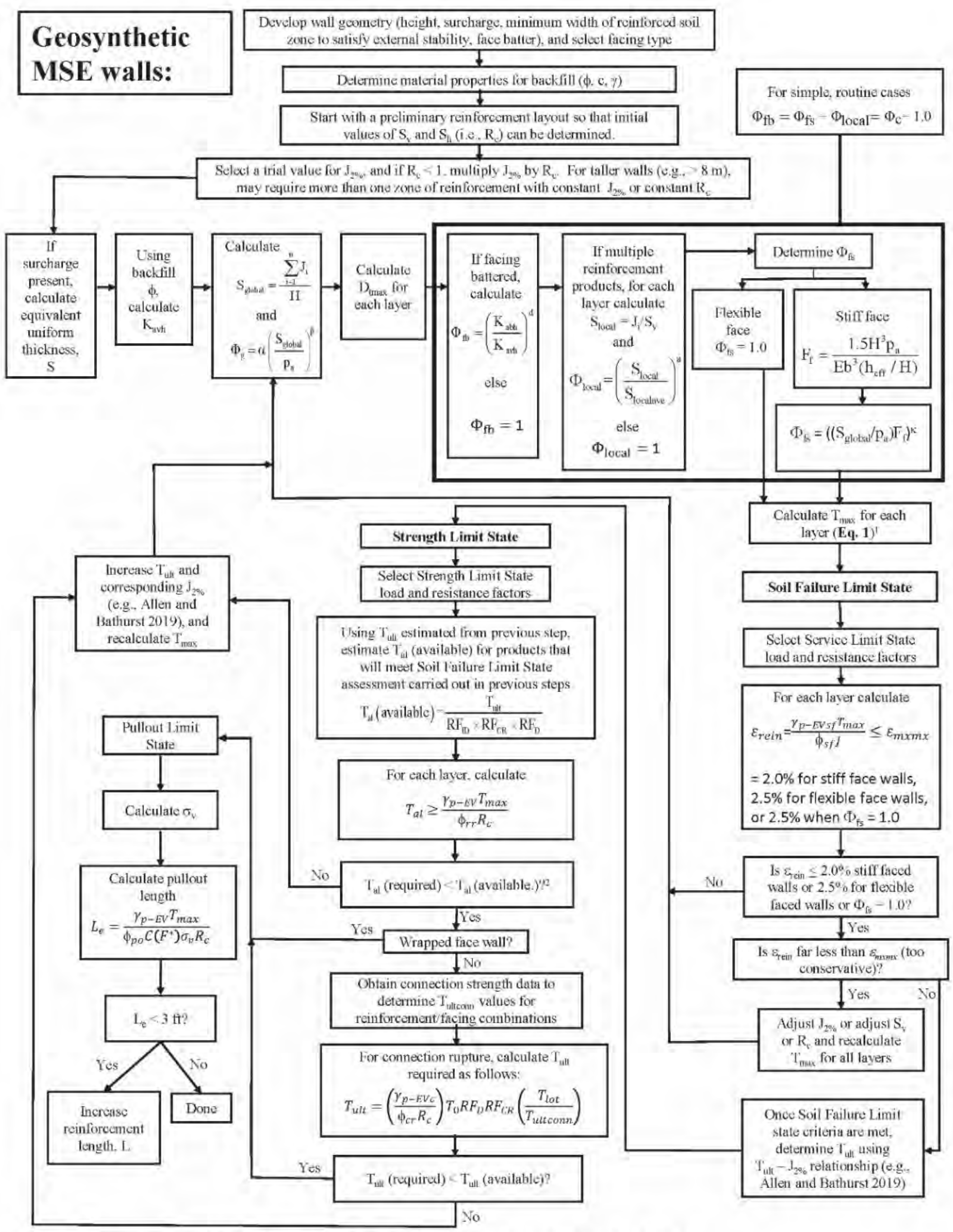
L_e	=	anchorage length,
F^* *and* α	=	dimensionless parameters based on reinforcement type,
σ_v	=	vertical stress acting on the reinforcement layer anchorage length, and
C	=	reinforcement surface geometry factor (set at 2 for strip, grid and sheet-type reinforcement).

Details how to determine α and F^*, vertical stress σ_v, and anchorage length L_e behind the active zone are provided in AASHTO (2020), Article 11.10.6.3.2.

15-E-2.5 Design Process for the Stiffness Method

Figure 15-E-1 illustrates the design process for the Stiffness Method, for geosynthetic walls (Allen and Bathurst 2018).

Figure 15-E-1 Design flowchart for the Stiffness Method for geosynthetic walls



¹ The cohesion factor Φ_c can be calculated here if PI > 6, but in this flow chart example c = 0 and therefore Φ_c = 1.0.
² Could multiply T_{al} by RF to get T_{ult}, and then compare to T_{ult} (available).

Figure 15-E-1 applies to internal stability Service and Strength Limit State design. If seismic design is required, seismic forces are considered outside of the Stiffness Method using superposition principles. See Section 15-E-3 for doing seismic design for internal stability.

15-E-3 Seismic Internal Stability Design when Using the Stiffness Method

The calculation of T_{max} using the Stiffness Method (Equation 15-E-1) is also applicable for seismic design. T_{md}, the incremental dynamic inertia force per reinforcement layer, must be added to T_{max} to determine the total reinforcement load for each layer during seismic loading.

T_{md} is calculated in accordance with Article 11.10.7.2 of AASHTO (2020). For convenience, the equations needed are as follows:

$$T_{md} = \left(\frac{P_i}{n}\right) \tag{15-E-17}$$

where,

P_i	=	internal inertia force due to the weight of backfill within the active zone, i.e., the shaded area in AASHTO (2020) Figure 11.10.7.2-1 (kips/ft)
n	=	total number of reinforcement layers in the wall at a specific wall section (dim)

k_h is dependent on the amount of horizontal movement of the reinforced soil mass during shaking that is allowed to occur or that will occur. Typically, if the wall is allowed to slide 1 to 2 inches, k_h can be assumed equal to $0.5A_s$, and A_s is equal to PGA x F_{pga}. F_{pga} is the site factor at a period of 0 seconds, and depends on the site class and the peak ground acceleration (PGA). See Table 6-4 in Chapter 6 of the GDM for values of F_{pga}.

If it is acceptable to allow more horizontal deformation during shaking (see GDM Section 15-4.10), k_h may be calculated as follows (AASHTO 2020):

$$k_h = 0.74A_s\left(\frac{A_s}{d}\right)^{0.25} \tag{15-E-18}$$

where,

k_h	=	horizontal seismic acceleration coefficient (dim)
A_s	=	earthquake ground acceleration coefficient as specified in Equation 3.10.4.2-2 in AASHTO (2020)
d	=	lateral wall displacement (in.)

Alternative formulations that may be used to estimate k_h as a function of wall displacement are provided in AASHTO (2020), specifically Appendix A11, Article A11.5.

Free-standing MSE walls may be designed to slide laterally up to 8 inches during earthquake shaking, provided that whatever is located above the wall can tolerate that amount of movement, and assuming that no collapse is the seismic performance objective for the wall.

P_i is determined as follows:

$$P_i = k_h\left(\gamma_r \times A_{active} + \gamma_{facing} \times T_f \times H\right) \tag{15-E-19}$$

where,

γ_r	=	unit weight of soil in reinforced backfill (kcf)
A_{active}	=	area of MSE reinforced backfill within active zone, plus soil surcharge above active zone as shown in Figure 11.10.7.2-1 in AASHTO (2020) (ft^2)
γ_{facing}	=	unit weight of structural facing or modular block facing (kcf)
T_f	=	thickness of facing, or for modular blocks, W_u (ft)
H	=	wall height at face (ft)

For thin or otherwise light weight facing elements, the weight of the facing may be ignored for this calculation.

The total load per reinforcement layer during seismic shaking, T_{totalf}, is then calculated using superposition as follows:

$$T_{totalf} = \gamma_{seis}(T_{max} + T_{md}) \tag{15-E-20}$$

where,

γ_{seis}	=	Extreme Event I load factor for reinforcement load due dead load plus seismically induced reinforcement load (dim)

Note that the reason γ_{seis} = 1.0 for both the static and dynamic portions of the load is that a significantly higher probability of failure is targeted due to the fact that the load has a small likelihood of occurring and also that some damage is acceptable during seismic loading. The Strength Limit State load factors are significantly greater than 1.0 because the probability of failure targeted is much lower, and significant damage due to the static loading is to be prevented by the design.

For seismic pullout design, T_{totalf} is used. However, for reinforcement rupture and connection rupture under seismic loading, the static and dynamic components of the reinforcement load must be handled separately. The reason for this is that the strength required to resist T_{max} must include the effects of creep because it is a sustained load, but the strength required to resist T_{md} should not include the effects of creep due to the transient nature of T_{md}. See AASHTO (2020) for additional details.

15-E-3.1 Reinforcement Rupture (Extreme Event I - Seismic)

The ultimate tensile strength of the reinforcement is determined by summing together the portion needed to resist the static force (i.e., T_{max}) and the portion needed to resist the dynamic portion (i.e., T_{md}). Therefore,

$$T_{ult} = S_{rs} + S_{rt} \tag{15-E-21}$$

where,

S_{rs} = ultimate reinforcement tensile resistance required to resist static load component (kips/ft)

S_{rt} = ultimate reinforcement tensile resistance required to resist dynamic load component (kips/ft)

For the static component,

$$S_{rs} = \frac{\gamma_{seis} T_{max} RF}{\phi R_c} \tag{15-E-22}$$

Again, T_{max} is determined using the Stiffness Method.

For the dynamic component,

$$S_{rt} = \frac{\gamma_{seis} T_{md} RF_{ID} RF_D}{\phi R_c} \tag{15-E-23}$$

where,

γ_{seis} = the Extreme Event I load factor for reinforcement load due to dead load plus seismically induced reinforcement load (dim.)

ϕ = resistance factor for combined static/earthquake loading from Article 11.5.8 (dim.)

R_c = reinforcement coverage ratio specified in Article 11.10.6.4.1 (dim.)

RF = combined strength reduction factor to account for potential long-term degradation due to installation damage, creep, and chemical aging specified in Article 11.10.6.4.3b (dim.)

RF_{ID} = strength reduction factor to account for installation damage to reinforcement specified in Article 11.10.6.4.3b (dim.)

RF_D = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation specified in Article 11.10.6.4.3b (dim.)

15-E-3.2 Connection Rupture (Extreme Event I - Seismic)

Similarly, the approach used for reinforcement rupture during seismic shaking is also used for connection rupture.

$$T_{ult} = S_{rsc} + S_{rtc} \tag{15-E-24}$$

where,

S_{rsc}	=	ultimate reinforcement tensile resistance required to resist static load component at connection (kips/ft)
S_{rtc}	=	ultimate reinforcement tensile resistance required to resist dynamic load component at connection (kips/ft)

$$S_{rsc} = \frac{\gamma_{seis} T_0 RF_D}{F_r \phi CR_{cr} R_c} \tag{15-E-25}$$

$$S_{rtc} = \frac{\gamma_{seis} T_{md} RF_D}{F_r \phi CR_u R_c} \tag{15-E-26}$$

where,

γ_{seis}	=	the Extreme Event I load factor for reinforcement load due to dead load plus seismically induced reinforcement load (dim)
T_0	=	applied load to reinforcement at facing connection (kip/ft)
RF_D	=	reduction factor to prevent rupture of reinforcement due to chemical and biological degradation specified in Article 11.10.6.4.4b (dim.)
ϕ	=	resistance factor applicable to seismic loading, typically 1.0 (dim.)
CR_{cr}	=	long-term connection strength reduction factor to account for reduced ultimate strength resulting from connection, equal to CR_u/RF_{CR} or T_{crc}/T_{lot}, in which T_{crc} is the creep limited connection strength at the desired design life if the creep limited connection strength is determined directly from the connection creep test data (dim.)
R_c	=	reinforcement coverage ratio from Article 11.10.6.4.1 (dim.)
T_{md}	=	factored incremental dynamic inertia force (kip/ft)
CR_u	=	short-term reduction factor to account for reduced ultimate strength resulting from connection as specified in AASHTO (2020) Article C11.10.6.4.4b, equal to $T_{ultconn}/T_{lot}$ (dim.)
F_r	=	reduction factor to account for reduced friction during shaking between facing blocks and geosynthetic reinforcement (equal to 0.8 if the connection relies primarily on friction, or 1.0 if the connection is structural, i.e., does not rely on friction)

15-E-3.3 Pullout (Extreme Event I - Seismic)

$$L_e = \frac{\gamma_{seis}(T_{max} + T_{md})}{\phi C (0.8 \alpha F^*) \sigma_v R_c} \tag{15-E-27}$$

where,

γ_{seis}	=	the Extreme Event I load factor for reinforcement load due to dead load plus seismically induced reinforcement load (dim)

15-E-4 Summary of MSE Wall Internal Stability Design Steps for Geosynthetic Walls Using the Stiffness Method

1. Establish wall geometry (height, surcharge, minimum width of reinforced soil zone to satisfy external stability, face batter), and select facing type
2. Establish soil backfill properties (ϕ, c, γ)
3. Develop preliminary wall reinforcement layout (S_v and S_h)
4. Calculate active earth pressure coefficient for reinforced zone
5. Calculate reinforcement load for each layer, T_{max}, using the creep stiffness required in each layer to meet Service Limit state requirements (i.e., the soil failure limit state) as a starting point
6. Calculate long-term tensile strength needed for each layer, T_{al}, for Strength Limit state, starting with creep stiffness values determined from Step 5
    a. Reinforcement rupture: $\phi T_{al} \geq \gamma_{p-EV} T_{max}$
    b. Connection rupture: $\phi T_{ac} \geq \gamma_{p-EV} T_0$
    c. In both cases, select geosynthetic reinforcement products with consideration to long-term strength reduction factors applicable to each product (i.e., RF_{ID}, RF_{CR}, and RF_D) and with consideration to the long-term connection strength available considering the block-geosynthetic combinations available
7. Calculate reinforcement length needed, $L_a + L_e$, for pullout, Strength Limit State
8. Calculate long-term strength needed for Extreme Event I Limit state (seismic design)
    a. $T_{totalf} = \gamma_{seis}(T_{max} + T_{md})$
    b. Reinforcement rupture
    c. Connection rupture
    d. In both cases, select geosynthetic reinforcement products with consideration to long-term strength reduction factors applicable to each product (i.e., RF_{ID} and RF_D, as RF_{CR} not important for seismic loading) and with consideration to the connection strength available considering the block-geosynthetic combinations available
    e. Pullout
9. If the connection strength is low and/or if the seismic acceleration is high and controls the reinforcement design with regard to strength and stiffness required, recalculate T_{max} using the increased stiffness required and recheck all limit states
10. Check compound stability (T_{al} and reinforcement length needed, both Strength and Extreme Event I limits)

A series of 20 ft tall wall design examples are provided in the sections that follow to illustrate these design steps for various cases. These design examples include:

1. Flexible face wall, coverage ratio, R_c, of 1.0,
2. Modular block face wall, coverage ratio, R_c, of 1.0, mechanical facing-reinforcement connection; same as Example 1 but with stiff, rather than flexible, facing,
3. Modular block face wall, coverage ratio, R_c, of 0.9, mechanical facing-reinforcement connection; same as Example 2 except coverage ratio is less than 1.0 to illustrate how the coverage ratio is addressed in design,
4. Modular block face wall, coverage ratio, R_c, of 0.9, frictional facing-reinforcement connection (proprietary wall system); same as Example 3 except that the facing reinforcement is frictional rather than mechanical, and
5. Flexible face wall, coverage ratio, R_c, of 1.0; same as Example 1, partial example to illustrate the effect of backfill cohesion, and how cohesion should, and should not be, handled in wall design.

15-E-5 Stiffness Method Design Example 1: Flexible Faced Geosynthetic Wall

15-E-5.1 General

This first example is a simple design case. Subsequent examples will add features that increase in complexity to illustrate how various scenarios are handled when using the Stiffness Method.

Figure 15-E-2 shows a cross-section of the wall for this design example, and material properties are provided in Table 15-E-3. Assume for this example that polyester (PET) geogrid will be used for the soil reinforcement. Assume a flexible facing will be used (e.g., welded wire baskets, or just a geosynthetic wrap facing). Assume that the connection between the geogrid and the facing is not an issue (i.e., the connection strength is 100% efficient) – while this may not be the case for welded wire baskets as shown, it will be the case for a wrapped face wall. So this assumption is made in this example to focus on the simplest case for illustration purposes. The wall backfill is assumed to be a well-graded gravelly sand, with no cohesion. The scope of this design example is limited to the service (soil failure limit only), strength, and Extreme Event I (seismic) limit states. Furthermore, for simplicity, live loads are not included in the calculations to follow. It is also assumed that the foundation soil has sufficient shear strength to prevent global instability, bearing capacity failure and excessive settlement. For seismic design, assume that the ground acceleration, A_s, is 0.50g.

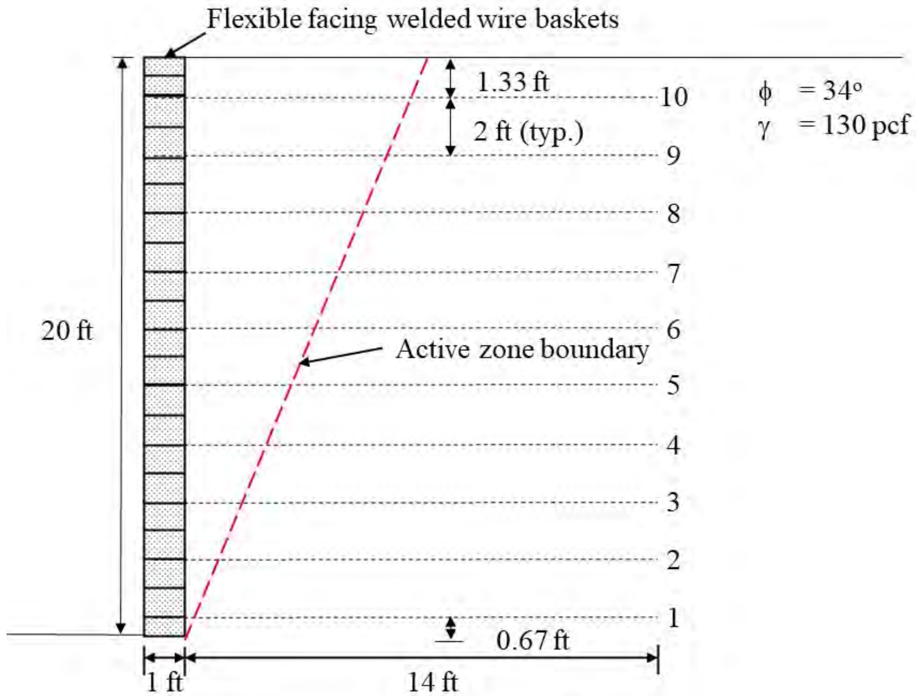
Note that a 2 ft vertical spacing of the reinforcement is used for this example. Normally, for a wrapped face geosynthetic wall, a vertical spacing of 2 ft is too large to keep face bulging, overall lateral deformation, and possibly vertical deformation, under control and is likely marginally too large even for a welded wire flexible faced wall. This spacing is being used in this example to facilitate comparisons with the stiff (dry cast concrete blocks) faced wall examples provided subsequent to this example. Also, a reinforcement coverage ratio, R_c, of 1.0 is used for this example. Also note that if a wrapped face geosynthetic wall is part of a two-stage wall system in which a concrete wall facing is added after the post-construction wall movement has ceased (e.g., the WSDOT Standard Plan geosynthetic wall), it is still designed as a flexible faced wall, since the more rigid concrete facia is added after the wrapped face wall is constructed.

Table 15-E-3 Design properties for wall

Property	Design Value
Moist soil unit weight (pcf)	130
Triaxial drained peak friction angle ϕ_{tx} (°)	34
Min. available, but not wall system specific, 1,000 hr, 2% secant J_i x R_c (kips/ft)	8.6 × 1.0 = 8.6
RF_{ID} RF_{CR} RF_D = RF	1.12x1.5x1.3 = 2.18
Coverage ratio, R_c	1.0
Facing welded wire basket height (ft)	1.0 (i.e., 2 baskets between reinforcement layers)
Facing welded wire basket width, W_u (face to tail) (ft)	1.0
Connection strength as fraction of T_{ult}, CR_u	1.0

In Table 15-E-3, and all subsequent uses, J_i is defined as the geosynthetic product secant creep stiffness at 1,000 hrs and 2% strain, per unit of reinforcement width. When J_i is multiplied by R_c, the resulting stiffness is per unit of wall width rather than per unit of reinforcement width.

Figure 15-E-2 Wall geometry and preliminary PET reinforcement layout for Design Example 1



Since, for this example, the wall is designed assuming that the wall face is flexible, Φ_{fs} = 1.0. It is assumed that the design can be completely generic (e.g., the WSDOT Standard Plan Geosynthetic wall). For this flexible wall face example, connection strength is assumed to not be a consideration, so either reinforcement stiffness or reinforcement rupture will likely control the design.

The wall geometry is based on Figure 15-E-2. Example calculations are demonstrated for one of the middle layers (i.e., Layer 6 in Figure 15-E-2, and for the soil failure and pullout limit states, Layer 10 is also used to illustrate calculations).

S = equivalent uniform height of surcharge = 0 ft

$K_{avh} = K_a = (1 - \sin \phi_r)/(1 + \sin \phi_r) = (1 - \sin 34°)/(1 + \sin 34°) = 0.283$

For walls with a facing batter ω > 0, the formula below is used to compute K_{abh} which appears in the facing batter factor equation (Φ_{fb}). Since the wall in this example is vertical (ω = 0), K_{abh} = K_{avh} as shown here:

$$K_{abh} = \frac{cos^2(\phi_r + \omega)}{cos^3\omega\left(1 + \frac{sin\phi_r}{\cos\omega}\right)^2} = \frac{cos^2(34^o + 0)}{cos^3 0\left(1 + \frac{sin 34^o}{\cos 0}\right)^2} = 0.283$$

The reinforcement stiffness values used in the calculations to follow need to be adjusted to account for the reinforcement coverage ratio, R_c. However, for this simple example, R_c is assumed to be 1.0, which is the typical case for flexible faced walls anyway.

Since the soil failure limit state usually controls the amount of reinforcement required, this limit state is evaluated first, and T_{max} calculated, for the wall design.

15-E-5.2 Calculations for Soil Failure Limit State Design (Service I)

The goal of this limit state is to ensure that the factored reinforcement strain in each layer is less than the target maximum strain in the wall required to prevent a contiguous shear surface through the backfill soil from developing (i.e., soil failure limit state). The AASHTO LRFD Bridge Design Specifications (AASHTO 2020) require that the factored reinforcement peak strain for each layer be 2.5% or less for a flexible faced wall. Some trial-and-error is typically required to establish what reinforcement stiffness values are required. As a first trial, use the minimum 1,000-hour 2% strain secant stiffness available for geosynthetic reinforcement products, J_i = 8.6 kips/ft for all layers (see Table 15-E-3).

Note that the creep stiffness and strength for specific products are available in the WSDOT Qualified Products List (QPL) Appendix E. Alternatively, this data can be obtained from NTPEP (2019) at the AASHTO NTPEP website (https://data.ntpep.org/REGEO/Products), and once there, click on "Construction" and then "Geosynthetic Reinforcement". In addition, Allen and Bathurst (2019) summarize all the NTPEP low strain 1,000 hour creep stiffness data available at that time, additional creep stiffness data found in the literature, and generic relationships between T_{ult} and the 1,000 hour 2% secant creep stiffness for various geosynthetic reinforcement product types.

The contributing factors, coefficients and parameters that comprise the T_{max} equation can be found in Allen and Bathurst (2015) and Table 15-E-1. To calculate T_{max}, the reinforcement stiffness must be per unit of wall width rather than per unit of reinforcement product width; hence, R_cJ_i must be used where the reinforcement stiffness value is required. Therefore, the parameters used to determine T_{max} are calculated as follows:

$$S_{\text{global}} = \frac{J_{\text{ave}}}{(\text{H/n})} = \frac{\sum_{i=1}^{n} R_c J_i}{H} = (10 \times 1.0 \times 8.6 \text{ kips/ft})/20 \text{ ft} = 4.30 \text{ ksf}$$

(applies to whole wall section)

$$\Phi_g = \alpha \left(\frac{S_{\text{global}}}{p_a}\right)^{\beta} = 0.16 \times (4.30 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.193 \text{ (applies to whole wall section)}$$

$$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}}\right)^{d} = (0.283/0.283)^{0.4} = 1.0 \text{ (applies to whole wall section)}$$

$$S_{\text{local}} = \left(\frac{R_c J}{S_v}\right)_i = (1.0 \times 8.6 \text{ kips/ft})/(2.0 \text{ ft}) = 4.30 \text{ ksf for Layer 6}$$

$$S_{localave} = \frac{\sum \left(R_c J/S_v\right)_i}{n} = \frac{3.69 + 8 \times 4.30 + 5.15}{10} = 4.32\ ksf$$

where, n = 10 is the number of reinforcement layers. Therefore, Φ_{local} is calculated as follows:

$$\Phi_{local} = \left(\frac{S_{local}}{S_{localave}}\right)^{0.5} = \left(\frac{4.30\ ksf}{4.32\ ksf}\right)^{0.5} = 1.00\ \ (layer\ 6)$$

The facing stiffness factor Φ_{fs} is equal to 1.0, since the facing is considered flexible.

Since c = 0, the cohesion factor, Φ_c = 1.0.

D_{tmax} is determined for Layer 6 as follows:

$z_b = C_h \times (H)^y \times \Phi_{fb} = (0.32 \times (20\ ft)^{1.2}) \times 1.0 = 11.65\ ft$

For $z \leq z_b$: $D_{tmax} = D_{tmax0} + (z/z_b) \times (1 - D_{tmax0}) = 0.12 + (9.33\ ft/11.65\ ft) \times (1 - 0.12) = 0.825$

For bottom layers where $z > z_b$: D_{tmax} = 1.0

T_{max} for layer 6 is calculated as follows:

$$T_{max} = S_v \left[H\gamma_r D_{tmax} + \left(\frac{H_{ref}}{H}\right) S\gamma_f + LL \right] K_{avh}\Phi_{fb}\Phi_g\Phi_{fs}\Phi_{local}\Phi_c$$

$$T_{max} = 2.0\ ft \left[20ft \times 0.130\ kcf \times 0.825 + \left(\frac{20\ ft}{20\ ft}\right) 0\ ft \times 0.130\ kcf + 0 \right] 0.283 \times 1.0 \times 0.193 \times 1.0 \times 1.0 \times 1.0$$

T_{max} = 0.233 kips/ft of wall width

Using Equation 15-14 with load factor γ_{sf} = 1.2, and resistance factor ϕ_{sf} = 1.0 (Table 15-5), the factored reinforcement strain corresponding to layer 10 is computed as:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.067 \frac{kips}{ft}}{1.0 \times 1.0 \times\ 8.6 \frac{kips}{ft}} \times 100\% = 0.94\% \leq 2.5\% \quad OK$$

For layer 6:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.233 \frac{kips}{ft}}{1.0 \times 1.0 \times 8.6 \frac{kips}{ft}} \times 100\% = 3.25\% \leq 2.5\% \quad No.$$

T_{max}, and the calculated parameters needed to calculate T_{max}, are summarized in Table 15-E-4 for the rest of the layers. As can be seen in the table and the calculations above, the calculated factored strains are greater than 2.5% in the lower half of the wall, which exceeds the soil failure limit state strain criterion provided in AASHTO (2020) of 2.5% for flexible faced walls.

Table 15-E-4 Summary of Example 1 wall design calculations using Stiffness Method considering only the Service Limit State, first trial using only the minimum stiffness geogrid product available.

	[+]T_{max} Equation (Eq. 15-E-1) Parameters												Soil Failure Limit State		
Layer Number	z (ft)	S_v (ft)	*R_cJ_i (kips/ft)	S_{global} (ksf)	S_{local} (ksf)	D_{tmax}	F_f	Φ_g	Φ_{local}	Φ_{fb}	Φ_{fs}	[+]T_{max} (kips/ft) (Equation 15-E-1)	Factored ε_{rein} (%)		[#]T_{al} Corresponding to J_i (kips/ft)
10 (top)	1.33	2.33	8.6	4.30	3.69	0.220	N/A	0.193	0.92	1.0	1.0	0.067	0.94		0.67
9	3.33	2.00	8.6	4.30	4.30	0.372	N/A	0.193	1.00	1.0	1.0	0.105	1.46		0.67
8	5.33	2.00	8.6	4.30	4.30	0.523	N/A	0.193	1.00	1.0	1.0	0.147	2.06		0.67
7	7.33	2.00	8.6	4.30	4.30	0.674	N/A	0.193	1.00	1.0	1.0	0.190	2.65		0.67
6	9.33	2.00	8.6	4.30	4.30	0.825	N/A	0.193	1.00	1.0	1.0	0.233	3.25	$\leq$ 2.5% (No; must increase stiffness)	0.67
5	11.33	2.00	8.6	4.30	4.30	0.976	N/A	0.193	1.00	1.0	1.0	0.275	3.84		0.67
4	13.33	2.00	8.6	4.30	4.30	1.00	N/A	0.193	1.00	1.0	1.0	0.282	3.94		0.67
3	15.33	2.00	8.6	4.30	4.30	1.00	N/A	0.193	1.00	1.0	1.0	0.282	3.94		0.67
2	17.33	2.00	8.6	4.30	4.30	1.00	N/A	0.193	1.00	1.0	1.0	0.282	3.94		0.67
1	19.33	1.67	8.6	4.30	5.15	1.00	N/A	0.193	1.09	1.0	1.0	0.258	3.60		0.67
Base of Wall	20											$\sum T_{max}$= 2.12			$\sum T_{al}$= 6.70

*Minimum stiffness needed to meet only the soil failure limit, considering all available geosynthetic reinforcement products.

[+]All values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (2020).

[#]For comparison to geogrid product MARV tensile strength that is not reduced by R_c (i.e., load per unit of reinforcement width basis).

To keep the peak reinforcement strains to less than 2.5% so that the soil failure limit state is met, the design stiffness values need to be increased. Through trial-and-error, the minimum stiffness values needed to keep the peak strains below 2.5% are as shown in Table 15-E-5.

Table 15-E-5 Creep stiffness values (i.e., at 2% strain and 1,000 h) for geogrids used in wall

Layer No.	Geogrid Designation	J_i (per Unit Width of Reinforcement, in kips/ft)	*R_c x J_i (per Unit Width of Wall, in kips/ft)
10	a	8.6	8.6
9	a	8.6	8.6
8	a	8.6	8.6
7	a	8.6	8.6
6	b	17.0	17.0
5	b	17.0	17.0
4	b	17.0	17.0
3	b	17.0	17.0
2	b	17.0	17.0
1	b	17.0	17.0

*This is the stiffness value which has been corrected for R_c to calculate T_{max}. Since R_c = 1.0, this stiffness is the same as the stiffness per unit of reinforcement width.

The input parameters to calculate T_{max} are recalculated as follows using the revised stiffness values:

$$S_{\text{global}} = \frac{J_{\text{ave}}}{(\text{H/n})} = \frac{\sum_{i=1}^{n} R_c J_i}{H} = (4 \times 1.0 \times 8.6 \text{ kips/ft} + 6 \times 1.0 \times 17.0 \text{ kips/ft})/20 \text{ ft} = 6.82 \text{ ksf}$$

$$\Phi_g = \alpha \left(\frac{S_{\text{global}}}{p_a}\right)^{\beta} = 0.16 \times (6.82 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.217$$

$$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}}\right)^{d} = (0.283/0.283)^{0.4} = 1.0$$

$$S_{\text{local}} = \left(\frac{R_c J}{S_v}\right)_i = (1.0 \times 17.0 \text{ kips/ft})/(2.0 \text{ ft}) = 8.50 \text{ ksf for Layer 6}$$

$$S_{localave} = \frac{\sum \left({}^{R_c J}/_{S_v}\right)_i}{n} = \frac{3.65 + 3 \times 4.25 + 5 \times 8.50 + 10.2}{10} = 6.91\ ksf$$

where, n = 10 is the number of reinforcement layers. Therefore, Φ_{local} is calculated as follows:

$$\Phi_{local} = \left(\frac{S_{local}}{S_{localave}}\right)^{0.5} = \left(\frac{8.5\ ksf}{6.91\ ksf}\right)^{0.5} = 1.11\ \ (layer\ 6)$$

For a flexible wall face, the facing stiffness factor is assumed to be 1.0.

Since c = 0, the cohesion factor, Φ_c = 1.0.

D_{tmax} does not change relative to the previous calculation (i.e., D_{tmax} for Layer 6 is 0.825).

T_{max} for Layer 6 is re-calculated as follows:

$$T_{max} = S_v \left[H\gamma_r D_{tmax} + \left(\frac{H_{ref}}{H} \right) S\gamma_f + LL \right] K_{avh} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c$$

$$T_{max} = 2.0\ ft \left[20ft \times 0.130\ kcf \times 0.825 + \left(\frac{20\ ft}{20\ ft} \right) 0\ ft \times 0.130\ kcf + 0 \right] 0.283 \times 1.0 \times 0.217 \times 1.0 \times 1.11 \times 1.0$$

T_{max} = 0.292 kips/ft

T_{max}, and the calculated parameters needed to calculate T_{max}, are summarized in Table 15-E-6 for the rest of the layers.

15-E-5.3 Calculations for Soil Failure Limit State Design (Service I)

Using Equation 15-14, with load factor γ_{sf} = 1.2 and resistance factor ϕ_{sf} = 1.0 (Table 15-5), the factored reinforcement strain corresponding to Layer 10 is computed as:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.060 \frac{kips}{ft}}{1.0 \times 1.0 \times 8.6 \frac{kips}{ft}} \times 100\% = 0.83\% \leq 2.5\% \quad OK$$

For Layer 6, using the revised layer stiffness values:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.292 \frac{kips}{ft}}{1.0 \times 1.0 \times 17.0 \frac{kips}{ft}} \times 100\% = 2.06\% \leq 2.5\% \quad OK$$

See Table 15-E-6 for the calculation results for the rest of the layers. As can be seen in the table, the new (increased) stiffness values are adequate to meet the soil failure limit criterion for all layers.

To estimate the equivalent tensile strength that corresponds to the stiffness values needed to meet the soil failure limit state criterion of 2.5% strain maximum, use the relationships provided in Allen and Bathurst (2019), or alternatively use the product line specific relationships provided in NTPEP (2019, or most current values). Allen and Bathurst (2019) recommend the following generic relationship between creep stiffness and ultimate tensile strength for geogrids:

$$T_{ult} = 0.17 J_i \tag{15-E-28}$$

For the two stiffness values required to meet the soil failure limit state, the approximate ultimate tensile strength per ft of wall width needed is:

T_{ult} = 0.17 x (8.6 kips/ft) = 1.46 kips/ft (applicable to Layer 10)

T_{ult} = 0.17 x (17.0 kips/ft) = 2.89 kips/ft (applicable to Layer 6)

To determine T_{al}, divide T_{ult} by RF = 1.12 x 1.5 x 1.3 = 2.18

Therefore, the approximate long-term tensile strengths implied by the stiffness required for the soil failure limit state for the reinforcement product (i.e., per ft of reinforcement product width) are:

T_{al} = (1.46 kips/ft)/2.18 = 0.67 kips/ft (applicable to Layer 10)

T_{al} = (2.89 kips/ft)/2.18 = 1.32 kips/ft (applicable to Layer 6)

15-E-5.4 Calculations for Reinforcement Rupture Limit State (Strength I)

Since no live load is present, the required long-term reinforcement strength (T_{al}) for Layer 6 is computed using Equation 15-E-5, solving for T_{al}.

The minimum required value of reinforcement product T_{al} and T_{ult}, on a strength per reinforcement width basis, for Layer 6 is therefore:

$$T_{al} \geq \frac{\gamma_{p-EV} T_{max}}{\phi_{rr} R_c} = \frac{1.35 \times 0.292 \frac{kips}{ft}}{0.80 \times 1.0} = 0.49 \frac{kips}{ft}$$

$$T_{ult} = T_{al} RF_{ID} RF_{CR} RF_D = 0.49 \times 1.12 \times 1.5 \times 1.3 = 1.07 \frac{kips}{ft}$$

For layer 6, T_{al} for reinforcement rupture is less than T_{al} needed to achieve the stiffness required for the soil failure limit state (i.e., 0.49 kips/ft << 1.32 kips/ft). Therefore, the soil failure limit state controls the design. See Table 15-E-6 for the calculation results for the rest of the layers.

15-E-5.5 Calculations for Pullout Limit State Design (Strength I)

The default pullout parameters α and F* specified in AASHTO (2020) are used for this example design. For geosynthetic reinforcement, α = 0.8 and F* = 0.67×tan ϕ. Since the design ϕ = 34°, F* = 0.67 × tan 34° = 0.452.

The vertical stress, σ_v, over the reinforcement anchorage length, L_e, can be approximated as:

$$\sigma_v = z\gamma_r + S_{sur}\gamma_f \tag{15-E-29}$$

Where,

z	=	depth of the reinforcement layer below the wall top,
S_{sur}	=	surcharge height directly above the active zone/resistant zone boundary at the layer,
γ_r	=	unit weight of reinforced soil backfill, and
γ_f	=	unit weight of surcharge soil.

Note that the AASHTO (2020) specifications allow the vertical stress to be calculated at the mid-point of L_e relative to the soil surface immediately above the layer at that location. Equation 15-E-29 is a simpler and slightly conservative version of this calculation (for design). Since pullout is usually most critical near the wall top, this example pullout calculation is carried out for Layer 10. Therefore, for Layer 10, the vertical stress used for the pullout calculation is:

$$\sigma_v = (1.33\ \text{ft})(0.13\ \text{kcf}) + (0)(\ 0.13\ \text{kcf}) = 0.173\ \text{ksf}$$

Combining equations 15-E-15 and 15-E-16 and solving for L_e, the required factored length of reinforcement in the resistant zone (i.e., behind the active wedge) is:

$$L_e = \frac{\gamma_{p-EV}T_{max}}{\phi_{po}C(\alpha F^*)\sigma_v R_c} \tag{15-E-30}$$

Where,

C	=	an overall reinforcement surface geometry factor (set at 2 for strip, grid and sheet type reinforcements).

As before, R_c = 1.0 and all other parameters and their values have been defined earlier. For layer 10:

$$L_e = \frac{1.35 \times 0.060\dfrac{kips}{ft}}{0.70 \times 2(0.8 \times 0.452)(0.173\ ksf)1.0} = 0.925\ ft$$

To determine the total reinforcement length needed, L, the length of reinforcement within the active zone, L_a, must be calculated. For geosynthetic walls, the active zone is assumed to be bounded by the Rankine active zone (wedge), illustrated in Figure 15-E-2. L_a is calculated as follows for a vertical wall (at layer 10):

$$L_a = (H - z) \tan (45° - \phi_r/2) = (20 \text{ ft} - 1.33 \text{ ft}) \tan (45° - 34°/2) = 9.93 \text{ ft}$$

The minimum length allowed for L_e is 3 ft (AASHTO 2020), which is greater than the calculated L_e required for pullout for layer 10. Therefore, using L_e = 3 ft, the total reinforcement length required for layer 10 is:

$$L = L_a + L_e = 9.93 \text{ ft} + 3 \text{ ft} = 12.9 \text{ ft}$$

Pullout calculation results for the other layers are summarized in Table 15-E-6.

It should be recalled that the minimum length of reinforcement for typical reinforced soil walls is 0.7H = 0.7×(20 ft) = 14 ft. Therefore, pullout does not control the reinforcement length required. Note that other limit state design calculations can result in greater reinforcement length, such as external stability (e.g., sliding) or global stability.

15-E-5.6 Calculations for Determination of T_{max} + T_{md} (Extreme Event I - Seismic)

The calculation of T_{max} as described and carried out earlier in this example using the Stiffness Method is also applicable for seismic design. T_{md}, the incremental dynamic inertia force per reinforcement layer, must be added to T_{max} using superposition to determine the total reinforcement load for each layer during seismic loading.

T_{md} is calculated using equations 15-E-17 through 15-E-19 as follows:

$$A_s = PGA \text{ x } F_{pga} = 0.50g$$

(Note: the site PGA and F_{pga} is determined from seismic maps provided in GDM Chapter 6; for this example the value of A_s has been arbitrarily picked for illustration purposes, as this example is not for a specific site).

Assume a maximum lateral deflection of 2 inches is allowed/anticipated. Using Equation 15-E-18, k_h is determined as follows:

$$k_h = 0.74 A_s \left(\frac{A_s}{d}\right)^{0.25} = 0.74 \times 0.50g \times \left(\frac{0.50}{2}\right)^{0.25} = 0.262g$$

The inertial force, P_i, is calculated using Equation 15-E-19 as follows:

$$P_i = k_h\left(\gamma_r \times A_{active} + \gamma_{facing} \times T_f \times H\right) = 0.262 \times \left(0.130\ kcf \times 0.5 \times \left(20\ ft \times Tan\left(45^o - \frac{34^o}{2}\right)\right) \times 20\ ft + 0.0\ kcf \times 0\ ft \times 0\ ft\right) = 3.62\frac{kips}{ft}$$

And therefore, T_{md} is calculated using Equation 15-E-17 as shown below:

$$T_{md} = \left(\frac{P_i}{n}\right) = \frac{3.62\frac{kips}{ft}}{10} = 0.362\frac{kips}{ft}$$

Note that the weight of any facing was not included in this calculation. If the facing is welded wire, the additional weight would be insignificant. If a second stage concrete facia is added, the weight of that facing should be included in the determination of T_{md}.

The load factor used for Extreme Event I (seismic) is equal to 1.0, and is applied to both the static portion and dynamic portion of the loading, as the probability of failure used for this limit state is much higher than what is used for the Strength Limit state. The use of a higher probability of failure is due to the low probability of occurrence of this load combination as well as greater tolerance for deformation and damage allowed, simply targeting no collapse for life safety. The total load per reinforcement layer during seismic shaking, T_{totalf}, is then calculated using superposition (Equation 15-E-20) as follows, for Layer 6):

$$T_{totalf} = \gamma_{seis}(T_{max} + T_{md}) = 1.0\left(0.292\frac{kips}{ft} + 0.362\frac{Kips}{ft}\right) = 0.654\frac{kips}{ft}$$

For seismic pullout design, T_{totalf} is used. However, for reinforcement rupture and connection rupture under seismic loading, the static and dynamic components of the reinforcement load must be handled separately. The reason for this is that the strength required to resist T_{max} must include the effects of creep because it is a sustained load, but the strength required to resist T_{md} should not include the effects of creep due to the transient nature of T_{md}.

15-E-5.7 Calculations for Reinforcement Rupture (Extreme Event I - Seismic)

Using equations 15-E-21, 15-E-22, and 15-E-23, T_{ult} for static portion of load at Layer 6 is calculated as follows:

$$S_{rs} = \frac{\gamma_{seis} T_{max} RF}{\phi R_c} = \frac{1.0 \times 0.292 \dfrac{kips}{ft} \times 2.18}{1.0 \times 1.0} = 0.636 \frac{kips}{ft}$$

T_{ult} for dynamic portion of load at Layer 6 is calculated as follows:

$$S_{rt} = \frac{\gamma_{seis} T_{md} RF_{ID} RF_D}{\phi R_c} = \frac{1.0 \times 0.362 \dfrac{kips}{ft} \times 1.12 \times 1.3}{1.0 \times 1.0} = 0.527 \frac{kips}{ft}$$

Therefore, the minimum required strength per unit width of reinforcement is as follows:

$T_{ult} = S_{rs} + S_{rt} = 0.636$ kips/ft + 0.527 kips/ft = 1.16 kips/ft

$T_{al} = 1.16$ kips/ft/2.18 = 0.532 kips/ft

For the soil failure limit in the Service Limit State, T_{al} that corresponds to the stiffness needed is 1.32 kips/ft >> 0.532 kips/ft. Therefore, the soil failure limit is still controlling the reinforcement design.

15-E-5.8 Calculations for Pullout Limit State Design (Extreme Event I - Seismic)

Using equations 15-E-27 and 15-E-29, L_e for Layer 10 (i.e., at the wall top) is determined as follows:

$\sigma_v = z\gamma_r + S_{sur}\gamma_f$

$\sigma_v = (1.33 \text{ ft})(0.13 \text{ kcf}) + (0)(0.13 \text{ kcf}) = 0.173 \text{ ksf}$

$$L_e = \frac{\gamma_{seis}(T_{max} + T_{md})}{\phi C(0.8\alpha F^*)\sigma_v R_c} = \frac{1.0\left(0.060 \dfrac{kips}{ft} + 0.362 \dfrac{kips}{ft}\right)}{1.0 \times 2 \times (0.8 \times 0.8 \times 0.452) \times 0.173\ kcf \times 1.0}$$

$L_e = 4.21$ ft

$L_a = (H - z) \tan(45° - \phi_r/2) = (20 \text{ ft} - 1.33 \text{ ft}) \tan(45° - 34°/2) = 9.93$ ft

$L = L_a + L_e = 9.93$ ft + 4.21 ft = 14.1 ft

See Table 15-E-6 for the calculation results for the rest of the layers with regard to pullout length required.

15-E-5.9 Calculations to Check to Make Sure Stiffness and Strength Required are Properly Matched (Extreme Event I, Seismic)

If a substantial increase in tensile strength is required to achieve internal stability for Strength Limit or Extreme Event I (seismic) Limit loading, the stiffness needed to obtain the increased tensile strength required must be determined, T_{max} recalculated, and all limit states recalculated. However, the tensile strength needed for strength and seismic limit design did not increase relative to what was required for the Service limit state. Therefore, no recalculations are required in this case.

15-E-5.10 Results of Example 1 Design Calculations

These calculation results for the Stiffness Method are summarized in Table 15-E-6, and are plotted and compared to design calculation results using the Simplified Method in figures 15-E-3, 15-E-4, and 15-E-5.

In summary, for the final internal stability design for Example 1, the following would be specified with regard to reinforcement properties and spacing for the optimized design, and the following observations can be made:

- The vertical spacing of reinforcements is 2 ft throughout the wall height, and the coverage ratio is a minimum of 1.0. Again, this large a spacing is used in this example to facilitate making direct comparisons with the examples that follow easier.
- The minimum reinforcement length for internal stability (i.e., pullout) is 14 ft for the Strength Limit, and all layers except the top layer for seismic design did not exceed this minimum length for both T_{max} methods. The top layer length required is 14.1 ft for seismic design for the Stiffness Method (slightly less than this for the Simplified Method), which is slightly greater than the 0.7H minimum. The longer reinforcement length due to pullout is consistent with good detailing for MSE walls in AASHTO (2020) LRFD Article 11.10.7.4.
- Assuming a PET uniaxial geogrid, to meet reinforcement rupture requirements, the reinforcement must have a minimum short-term (ultimate) and long-term (i.e., T_{al}) tensile strength as tabulated in Table 15-E-6. The strength and stiffness needed to meet the Soil Failure Limit State controls the design for all layers. Note that these values are based on strength per unit of reinforcement width. Final selection of reinforcements result in a total T_{al} for the wall section of 10.6 kips/ft for the Stiffness Method and 11.9 kips/ft for the Simplified Method if the minimum strength needed is set at 0.67 kips/ft based on product availability.
- Overall, the required strength and stiffness of the reinforcement needed is in the low to mid-range of the geogrid product lines available. The Stiffness Method requires less reinforcement than the Simplified Method in the bottom portion of the wall, and in this case seismic does not control the design. The ground acceleration used represents what would be included in Seismic Zone 4, which is the highest seismic zone.

Table 15-E-6 Summary of Example 1 wall design calculations using Stiffness Method and R_c = 1.0 (Service, Strength, and Extreme Event I Limit States): (a) Calculation of T_{max}, (b) Service and Strength Limit State calculations, and (c) Extreme Event I (seismic) Limit State calculations.

a)	T_{max} Equation (Eq. 15-E-1) Parameters											Unfactored maximum reinforcement load
Layer Number	**z (ft)**	**S_v (ft)**	**R_cJ_i (kips/ft)**	**S_{global} (ksf)**	**S_{local} (ksf)**	**D_{tmax}**	**F_f**	**Φ_g**	**Φ_{local}**	**Φ_{fb}**	**Φ_{fs}**	**+T_{max} (kips/ft)**
10 (top)	1.33	2.33	8.6	6.82	3.65	0.220	N/A	0.217	0.73	1.0	1.0	0.060
9	3.33	2.00	8.6	6.82	4.25	0.372	N/A	0.217	0.79	1.0	1.0	0.093
8	5.33	2.00	8.6	6.82	4.25	0.523	N/A	0.217	0.79	1.0	1.0	0.131
7	7.33	2.00	8.6	6.82	4.25	0.674	N/A	0.217	0.79	1.0	1.0	0.168
6	9.33	2.00	17.0	6.82	8.50	0.825	N/A	0.217	1.11	1.0	1.0	0.292
5	11.33	2.00	17.0	6.82	8.50	0.976	N/A	0.217	1.11	1.0	1.0	0.345
4	13.33	2.00	17.0	6.82	8.50	1.00	N/A	0.217	1.11	1.0	1.0	0.354
3	15.33	2.00	17.0	6.82	8.50	1.00	N/A	0.217	1.11	1.0	1.0	0.354
2	17.33	2.00	17.0	6.82	8.50	1.00	N/A	0.217	1.11	1.0	1.0	0.354
1	19.33	1.67	17.0	6.82	10.2	1.00	N/A	0.217	1.21	1.0	1.0	0.323
Base of wall	20											$\sum T_{max}$= 2.47

b)			Reinforcement Rupture (Strength Limit)		Soil Failure (Service Limit)			Pullout (Strength Limit)	
Layer Number	**z (ft)**	**Reinforcement Product**	#Minimum Required Strength per Unit Width of Reinforcement		**Factored ε_{rein} (%)**	#Tensile Strength Corresponding to Reinforcement Stiffness Required		Anchorage length L_e (ft)	
			T_{al} (kips/ft)	**$T_{ult} = T_{al} \times RF$ (kips/ft)**		**T_{al} (kips/ft)**	**$T_{ult} = T_{al} \times RF$ (kips/ft)**	**Required**	**Minimum allowed**
10 (top)	1.33	Geogrid a	0.10	0.22	0.83	0.67	1.46	0.923	
9	3.33	Geogrid a	0.16	0.34	1.30	0.67	1.46	0.575	
8	5.33	Geogrid a	0.22	0.48	1.85	0.67	1.46	0.506	
7	7.33	Geogrid a	0.29	0.62	2.38	0.67	1.46	0.474	
6	9.33	Geogrid b	0.49	1.07	2.06 ≤ 2.5% (OK)	1.32	2.89	0.641	< 3.0 (OK)
5	11.33	Geogrid b	0.58	1.27	2.44	1.32	2.89	0.625	
4	13.33	Geogrid b	0.60	1.30	2.50	1.32	2.89	0.544	
3	15.33	Geogrid b	0.60	1.30	2.50	1.32	2.89	0.473	
2	17.33	Geogrid b	0.60	1.30	2.50	1.32	2.89	0.418	
1	19.33	Geogrid b	0.55	1.19	2.28	1.32	2.89	0.343	
Base of wall	20		$\sum T_{al}$= 4.17	$\sum T_{ult}$= 9.12		$\sum T_{al}$= 10.6	$\sum T_{ult}$= 23.2		

Table 15-E-6, continued

Summary of Example 1 wall design calculations using Stiffness Method and R_c = 1.0 (Service, Strength, and Extreme Event I Limit States): (c) Extreme Event I (seismic) Limit State calculations.

c)		**Soil Failure (Service Limit, for Comparison)**	**Reinforcement Rupture (Extreme Event I Limit)**		**Pullout (Extreme Event I Limit)**		
Layer Number	**z (ft)**	**[#]Tensile Strength T_{al} Corresponding to Reinforcement Stiffness Required (kips/ft)**	**[#]Minimum Required Strength per Unit Width of Reinforcement**		**Anchorage length L_{eseis} (ft)**		**Total Reinforcement Length (ft)**
			T_{al} (kips/ft)	**$T_{ult} = T_{al} \times RF$ (kips/ft)**	**Required**	**Minimum allowed**	**(minimum is 0.7H = 14 ft)**
10 (top)	1.33	0.67	0.30	0.66	4.21	< 3.0 (No, so pullout controls length, but only at wall top)	14.1
9	3.33	0.67	0.33	0.73	1.82		10.7
8	5.33	0.67	0.37	0.81	1.23		9.0
7	7.33	0.67	0.41	0.90	0.96		7.7
6	9.33	1.32	0.53	1.16	0.93		6.6
5	11.33	1.32	0.59	1.28	0.83		5.4
4	13.33	1.32	0.59	1.30	0.71		4.3
3	15.33	1.32	0.59	1.30	0.62		3.1
2	17.33	1.32	0.59	1.30	0.55		2.0
1	19.33	1.32	0.56	1.23	0.47		0.8
Base of wall	20	$\sum T_{max}$= 10.6	$\sum T_{al}$= 4.89	$\sum T_{ult}$= 10.7			

[+]These values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (AASHTO 2020).

[#]For comparison to geogrid product tensile strength that is not reduced by R_c (i.e., load per unit of reinforcement width basis).

*T_{md} for all reinforcement layers is 0.362 kips/ft.

Figure 15-E-3 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Service, Strength, and Extreme Event I (seismic) limit states (Example 1).

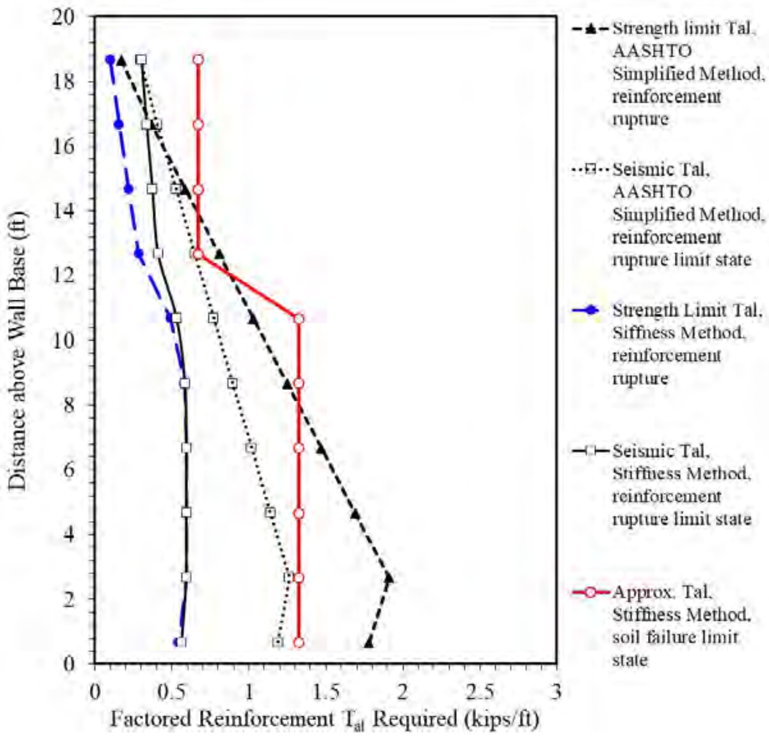
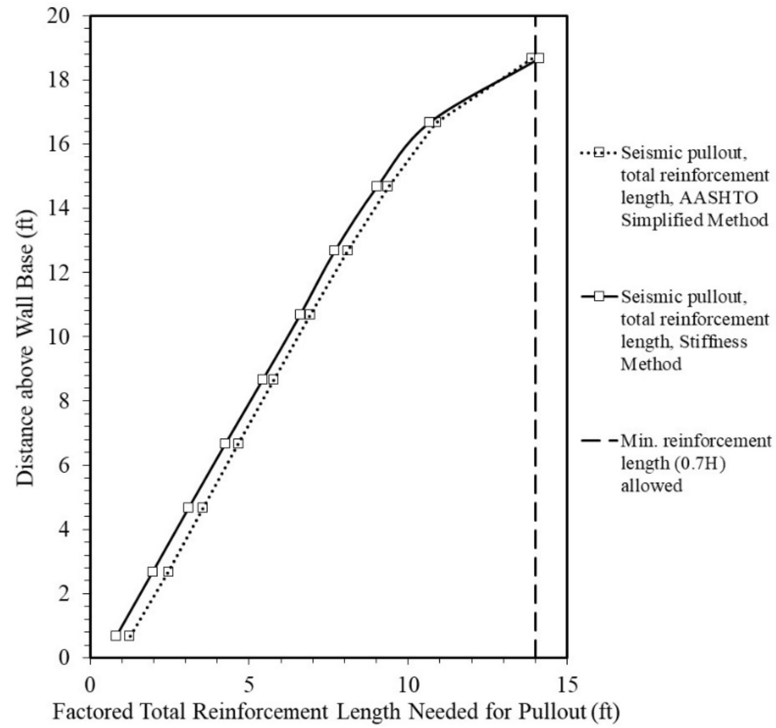


Figure 15-E-4 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Extreme Event I (seismic) limit state, pullout (Example 1).



15-E-6 Stiffness Method Design Example 2: Block Faced Geosynthetic Wall with Mechanical Connections, R_c = 1.0

15-E-6.1 General

This second example is also a fairly simple design case. Subsequent examples will add features that increase in complexity to illustrate how various scenarios are handled when using the Stiffness Method.

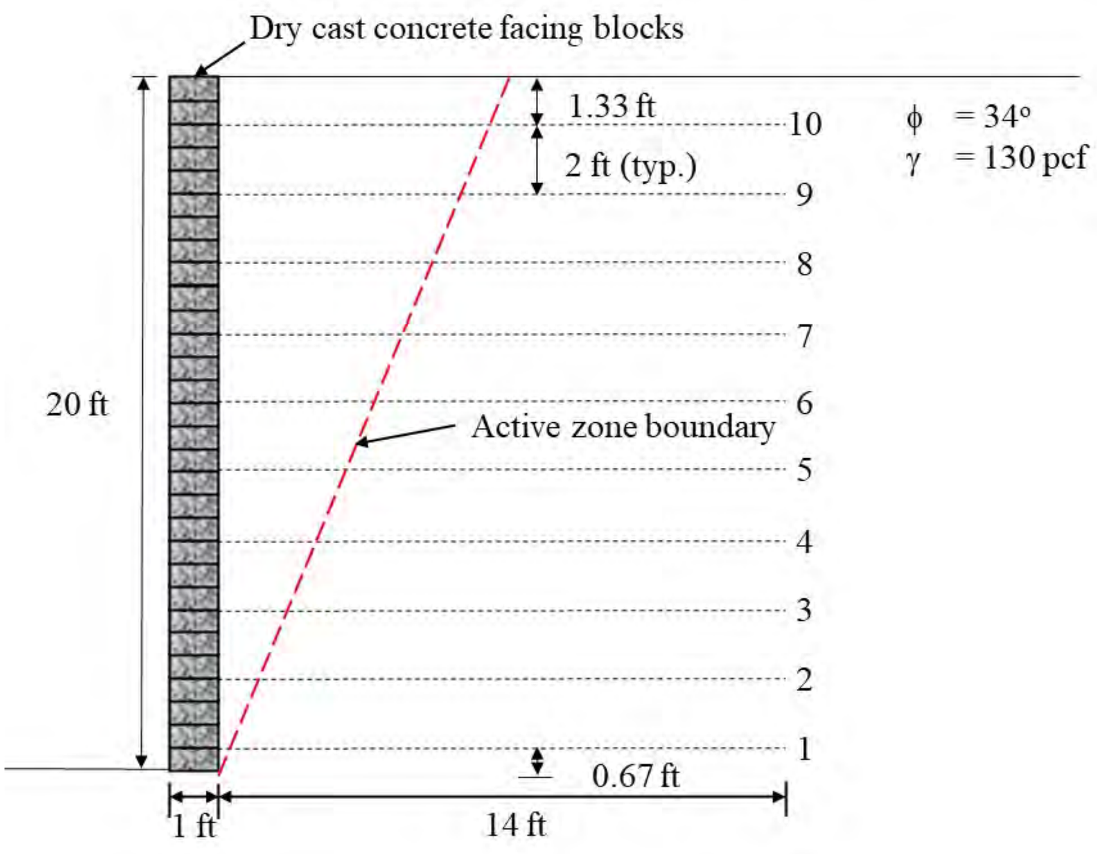
Figure 15-E-5 shows a cross-section of the wall for this design example, and material properties are provided in Table 15-E-7. The reinforcement coverage ratio, R_c is set at 1.0. Assume for this example that polyester (PET) geogrid will be used for the soil reinforcement. Assume dry cast concrete blocks will be used for the facing. A mechanical facing-geogrid connection will be assumed, so the connection strength is a constant fraction of the geogrid ultimate tensile strength and is not affected by the normal force between the blocks in the facing column. The wall backfill is assumed to be a well-graded gravelly sand, with no cohesion.

The scope of this design example is limited to the service (soil failure limit only), strength, and Extreme Event I (seismic) limit states. Furthermore, for simplicity, live loads are not included in the calculations to follow. It is also assumed that the foundation soil has sufficient shear strength to prevent global instability, bearing capacity failure and excessive settlement. For seismic design, assume that the ground acceleration, A_s, is 0.50g.

Table 15-E-7 Design properties for wall

Property	Design Value
Moist soil unit weight (pcf)	130
Triaxial drained peak friction angle ϕ_{tx} (°)	34
Min. available, but not wall system specific, 1,000 hr, 2% secant J_i x R_c (kips/ft)	8.6 × 1.0 = 8.6
RF_{ID} RF_{CR} RF_D = RF	1.12x1.5x1.3 = 2.18
Coverage ratio, R_c	1.0
Facing block unit weight, γ_{block} (pcf)	120
Facing block height (ft)	0.67
Facing block width, W_u (face to tail) (ft)	1.0
Connection strength as fraction of T_{ult}, CR_u	0.75

Figure 15-E-5 Wall geometry and preliminary PET reinforcement layout for Design Example 2.



The wall geometry is based on Figure 15-E-5. As is true in Example 1, Example 2 calculations are demonstrated for one of the middle layers (i.e., Layer 6 in Figure 15-E-5, and for the soil failure and pullout limit states, Layer 10 is also used to illustrate calculations).

S = equivalent uniform height of surcharge = 0 ft. Since the wall in this example is vertical (ω = 0) and is using the same soil as used for Example 1, K_{abh} = K_{avh} 0.283, and Φ_{fb} = 1.0.

Since the soil failure limit state usually controls the amount of reinforcement required, this limit state is evaluated first, and T_{max} is calculated, for the wall design.

15-E-6.2 Calculations for Soil Failure Limit State Design (Service I)

As is true for Example 1, the goal of this limit state is to ensure that the factored reinforcement strain in each layer is less than the target maximum strain in the wall required to prevent a contiguous shear surface through the backfill soil from developing (i.e., soil failure limit state). The factored reinforcement peak strain for each layer should be 2.0% or less per AASHTO (2020) for a stiff faced wall such as the modular block faced wall in this example. Some trial-and-error is typically required to establish what reinforcement stiffness values are required, starting with the stiffness values required to meet the soil failure limit, as the soil failure limit often controls the design of geosynthetic walls. As a first trial, use the minimum 1,000-hour 2% strain secant stiffness available for geosynthetic reinforcement products, J_i = 8.6 kips/ft per unit of reinforcement width for all layers (see Table 15-E-7).

The contributing factors, coefficients and parameters that comprise the T_{max} equation can be found in Allen and Bathurst (2015) and Table 15-E-1. To calculate T_{max}, the reinforcement stiffness must be per unit of wall width rather than per unit of reinforcement product width; hence, R_cJ_i must be used where the reinforcement stiffness value is required. Therefore, the parameters used to determine T_{max} are calculated as follows:

$S_{\text{global}} = \frac{J_{\text{ave}}}{(\text{H/n})} = \frac{\sum_{i=1}^{n} R_c J_i}{H}$ = (10 × 1.0 × 8.6 kips/ft)/20 ft = 4.30 ksf (applies to whole wall section)

$\Phi_g = \alpha \left(\frac{S_{\text{global}}}{p_a}\right)^{\beta}$ = 0.16 × (4.30 ksf/2.11 ksf)$^{0.26}$ = 0.193 (applies to whole wall section)

$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}}\right)^{d}$ = (0.283/0.283)$^{0.4}$ = 1.0 (applies to whole wall section)

$S_{\text{local}} = \left(\frac{J}{S_v}\right)_i$ = (8.6 kips/ft)/(2.0 ft) = 4.30 ksf for Layer 6

$$S_{localave} = \frac{\sum \left(J/S_v \right)_i}{n} = \frac{3.69 + 8 \times 4.30 + 5.15}{10} = 4.32\ ksf$$

where, n = 10 is the number of reinforcement layers. Therefore, Φ_{local} is calculated as follows:

$$\Phi_{local} = \left(\frac{S_{local}}{S_{localave}}\right)^{0.5} = \left(\frac{4.30\ ksf}{4.32\ ksf}\right)^{0.5} = 1.00\ \ (layer\ 6)$$

To determine the facing stiffness factor, one must first determine the facing stiffness parameter, F_f. To calculate the facing stiffness parameter F_f, the equivalent height of an unjointed facing column that is approximately 100% efficient in transmitting moment through the height of the facing column, h_{eff}, must be determined. Since the facing stiffness factor Φ_{fs} is intended to be a single value for the wall, a single representative value of h_{eff} must be selected. Typically, h_{eff} is set equal to the reinforcement vertical spacing in modular block-type structures since the reinforcement is located at the horizontal joints between facing units. For blocks that do not have a reinforcement layer at the horizontal joints, these facings will have better interlock and moment transfer from block to block. If the reinforcement spacing is non-uniform, the smallest predominate spacing (e.g., for 3 or more layers) should be used for this calculation. Smaller h_{eff} values will lead to more conservative (safer) design because the facing stiffness factor will be larger.

In this example, the vertical spacing is reasonably uniform throughout the wall height at 2.0 ft; thus, h_{eff} = 2.0 ft is selected in the calculations to follow. The facing stiffness parameter F_f is calculated using h_{eff} = 2.0 ft, H = 20 ft, W_u = b = 1 ft, and E = 157,000 ksf. This value for E is for dry cast concrete, which has a typical value of E = 209,000 ksf, but has been reduced here to reflect the non-uniform cross-section of the facing unit (typical for modular dry cast concrete blocks). Therefore:

$$F_f = \frac{1.5H^3 p_a}{Eb^3\left(\frac{h_{eff}}{H}\right)} = \frac{1.5(20\,ft)^3(2.11\,ksf)}{(157{,}000\,ksf)(1\,ft^3)\left(\frac{2\,ft}{20\,ft}\right)} = 1.61 \text{ (applies to whole wall section)}$$

and the facing stiffness factor is:

$$\Phi_{fs} = \eta\left(\left(\frac{S_{global}}{p_a}\right)F_f\right)^{\kappa} = 0.57 \times \left(\left(\frac{4.30\,ksf}{2.11\,ksf}\right) \times 1.61\right)^{0.15} = 0.681 \text{ (applies to whole wall section)}$$

Since c = 0, the cohesion factor, Φ_c = 1.0.

D_{tmax} does not change relative to the previous calculation (i.e., D_{tmax} for Layer 6 is 0.825).

T_{max} for Layer 6 is calculated as follows:

$$T_{max} = S_v\left[H\gamma_r D_{tmax} + \left(\frac{H_{ref}}{H}\right)S\gamma_f + LL\right]K_{avh}\Phi_{fb}\Phi_g\Phi_{fs}\Phi_{local}\Phi_c$$

$$T_{max} = 2.0\,ft\left[20ft \times 0.130\,kcf \times 0.825 + \left(\frac{20\,ft}{20\,ft}\right)0\,ft \times 0.130\,kcf + 0\right]0.283 \times 1.0 \times 0.193 \times 0.681 \times 1.00 \times 1.0 = 0.159\,\frac{\text{kips}}{\text{ft}} \text{ of wall width}$$

Using Equation 15-14 with load factor γ_{sf} = 1.2, and resistance factor ϕ_{sf} = 1.0 (Table 15-5), the factored reinforcement strain corresponding to layer 6 is computed as:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.159 \frac{kips}{ft}}{1.0 \times 1.0 \times 8.6 \frac{kips}{ft}} \times 100\% = 2.21\% \leq 2\% \quad No$$

As shown for Example 1 (Equation 15-E-28), the equivalent ultimate tensile strength that corresponds to the stiffness values needed to meet the soil failure limit state is equal to 0.17J_i. For the stiffness value used in this first trial, the approximate ultimate tensile strength per ft of wall width needed is:

T_{ult} = 0.17 x (8.6 kips/ft) = 1.46 kips/ft (applicable to all layers for first trial)

To determine T_{al}, divide T_{ult} by RF = 1.12 x 1.5 x 1.3 = 2.18. Therefore, T_{al} per ft of wall width and per ft of reinforcement width, since R_c = 1.0, is:

T_{al} = (1.46 kips/ft)/2.18 = 0.67 kips/ft (applicable to all layers)

T_{max}, the calculated parameters needed to calculate T_{max}, and the predicted reinforcement strains for all the layers are summarized in Table 15-E-8 for the rest of the layers.

As can be seen in the table, the calculated factored strains are greater than 2% in the lower half of the wall, which exceeds the soil failure limit state strain criterion of 2% for stiff faced walls. Therefore, the reinforcement stiffness needs to be increased in the lower half of the wall. Through some trial-and-error, the soil failure limit state is met for this example using the stiffness values provided in Table 15-E-9.

Table 15-E-8 Summary of Example 2 wall design calculations using Stiffness Method considering only the Service Limit State, first trial using only the minimum stiffness geogrid product available.

	+T_{max} Equation (Eq. 15-E-1) Parameters												Soil Failure Limit State		
Layer Number	**z (ft)**	**S_v (ft)**	***R_cJ_i (kips/ft)**	**S_{global} (ksf)**	**S_{local} (ksf)**	**D_{tmax}**	**F_f**	**Φ_g**	**Φ_{local}**	**Φ_{fb}**	**Φ_{fs}**	**+T_{max} (kips/ft) (Equation 15-E-1)**	**Factored ε_{rein} (%)**		**#T_{al} Corresponding to J_i (kips/ft)**
10 (top)	1.33	2.33	8.6	4.30	3.69	0.220	1.61	0.193	0.92	1.0	0.681	0.046	0.64		0.67
9	3.33	2.00	8.6	4.30	4.30	0.372	1.61	0.193	1.00	1.0	0.681	0.071	1.00		0.67
8	5.33	2.00	8.6	4.30	4.30	0.523	1.61	0.193	1.00	1.0	0.681	0.101	1.40		0.67
7	7.33	2.00	8.6	4.30	4.30	0.674	1.61	0.193	1.00	1.0	0.681	0.130	1.81		0.67
6	9.33	2.00	8.6	4.30	4.30	0.825	1.61	0.193	1.00	1.0	0.681	0.159	2.21	$\leq$ 2.0% (No; must increase stiffness)	0.67
5	11.33	2.00	8.6	4.30	4.30	0.976	1.61	0.193	1.00	1.0	0.681	0.188	2.62		0.67
4	13.33	2.00	8.6	4.30	4.30	1.00	1.61	0.193	1.00	1.0	0.681	0.192	2.68		0.67
3	15.33	2.00	8.6	4.30	4.30	1.00	1.61	0.193	1.00	1.0	0.681	0.192	2.68		0.67
2	17.33	2.00	8.6	4.30	4.30	1.00	1.61	0.193	1.00	1.0	0.681	0.192	2.68		0.67
1	19.33	1.67	8.6	4.30	5.15	1.00	1.61	0.193	1.09	1.0	0.681	0.176	2.45		0.67
Base of Wall	20											$\sum T_{max}$= 1.45			$\sum T_{al}$= 6.70

*Minimum stiffness needed to meet only the soil failure limit, considering all geosynthetic reinforcement products, and not just the specific products available for the wall system (i.e., weaker than Geogrid A in this example).

+All values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO *LRFD Bridge Design Manual* (AASHTO 2020).

#For comparison to geogrid product MARV tensile strength that is not reduced by R_c (i.e., load per unit of reinforcement width basis).

Table 15-E-9 Creep stiffness values (i.e., at 2% strain and 1,000 h) for geogrids used in wall

Layer No.	Geogrid Designation	J_i (per Unit Width of Reinforcement, in kips/ft)	*R_c x J_i (per Unit Width of Wall, in kips/ft)
10	a	8.6	8.6
9	a	8.6	8.6
8	a	8.6	8.6
7	a	8.6	8.6
6	b	14.5	14.5
5	b	14.5	14.5
4	b	14.5	14.5
3	b	14.5	14.5
2	b	14.5	14.5
1	b	14.5	14.5

*This is the stiffness value which has been corrected for R_c to calculate T_{max}.
Since R_c = 1.0, this stiffness is the same as the stiffness per unit of reinforcement width.

Using the Table 15-E-9 stiffness values, considering a total of 10 layers, S_{global} is recalculated as:

$S_{\text{global}} = \frac{J_{\text{ave}}}{(\text{H/n})} = \frac{\sum_{i=1}^{n} R_c J_i}{H}$ = (4 × 1.0 × 8.6 kips/ft + 6 × 1.0 × 14.5 kips/ft)/20 ft = 6.07 ksf

$$\Phi_g = \alpha \left(\frac{S_{\text{global}}}{p_a}\right)^{\beta} = 0.16 \times (6.07 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.211$$

S_{local} is recalculated as:

$$S_{\text{local}} = \left(\frac{R_c J}{S_v}\right)_i = (1.0 \times 14.5 \text{ kips/ft})/(2.0 \text{ ft}) = 7.25 \text{ ksf for Layer 6}$$

$$S_{localave} = \frac{\sum \left({}^{R_c J}/_{S_v}\right)_i}{n} = \frac{3.69 + 3 \times 4.30 + 5 \times 7.25 + 8.68}{10} = 6.15\ ksf$$

where, n = 10 is the number of reinforcement layers. Therefore, Φ_{local} is recalculated as follows:

$$\Phi_{local} = \left(\frac{S_{local}}{S_{localave}}\right)^{0.5} = \left(\frac{7.25\ ksf}{6.15\ ksf}\right)^{0.5} = 1.09\ \ (Layer\ 6)$$

To determine the facing stiffness factor, the facing stiffness parameter, F_f does not change, and the facing stiffness factor is recalculated as:

$$\Phi_{fs} = \eta\left(\left(\frac{S_{global}}{p_a}\right)F_f\right)^{\kappa} = 0.57 \times \left(\left(\frac{6.07\ ksf}{2.11\ ksf}\right) \times 1.61\right)^{0.15} = 0.718$$

Since c = 0, the cohesion factor, Φ_c = 1.0.

D_{tmax} remains unchanged.

T_{max} for Layer 6 is now recalculated as follows:

$$T_{max} = S_v\left[H\gamma_r D_{tmax} + \left(\frac{H_{ref}}{H}\right)S\gamma_f + LL\right]K_{avh}\Phi_{fb}\Phi_g\Phi_{fs}\Phi_{local}\Phi_c$$

$$T_{max} = 2.0\ ft\left[20ft \times 0.130\ kcf \times 0.825 + \left(\frac{20\ ft}{20\ ft}\right)0\ ft \times 0.130\ kcf + 0\right]0.283 \times 1.0 \times 0.211 \times 0.718 \times 1.09 \times 1.0$$

T_{max} = 0.199 kips/ft (load per unit of wall width)

T_{max}, and the recalculated parameters needed to calculate T_{max}, are summarized in Table 15-E-10 for the rest of the layers for the Service and Strength limit states.

Using Equation 15-14 with load factor γ_{sf} = 1.2, and resistance factor ϕ_{sf} = 1.0 (Table 15-5), the factored reinforcement strain corresponding to layer 10 is recomputed as:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf}T_{max}}{\phi_{sf}R_cJ_i} = \frac{1.2 \times 0.044\frac{kips}{ft}}{1.0 \times 1.0 \times 8.6\frac{kips}{ft}} \times 100\% = 0.62\% \leq 2\% \quad OK$$

For layer 6:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf}T_{max}}{\phi_{sf}R_cJ_i} = \frac{1.2 \times 0.199\frac{kips}{ft}}{1.0 \times 1.0 \times 14.5\frac{kips}{ft}} \times 100\% = 1.65\% \leq 2\% \quad OK$$

See Table 15-E-10 for the calculation results for the rest of the layers. Therefore, the soil failure limit state is met for all the reinforcement layers.

To estimate the equivalent tensile strength that corresponds to the stiffness values needed to meet the soil failure limit state criterion of 2% strain maximum, use the relationships provided in Allen and Bathurst (2019), i.e., $T_{ult} = 0.17J_i$, or alternatively use the product line specific relationships provided in NTPEP (2019).

For the two stiffness values required to meet the soil failure limit state, the approximate ultimate tensile strength per unit of wall width to obtain the stiffness needed is:

T_{ult} = 0.17 x (8.6 kips/ft) = 1.46 kips/ft (applicable to Layer 10)

T_{ult} = 0.17 x (14.5 kips/ft) = 2.47 kips/ft (applicable to Layer 6)

To determine T_{al}, divide T_{ult} by RF = 1.12 x 1.5 x 1.3 = 2.18

Therefore, the approximate long-term tensile strengths implied by the stiffness required for the soil failure limit state for the reinforcement product (i.e., per unit of wall width and reinforcement product width, since R_c = 1.0) are:

T_{al} = (1.46 kips/ft)/2.18 = 0.67 kips/ft (applicable to Layer 10)

T_{al} = (2.47 kips/ft)/2.18 = 1.13 kips/ft (applicable to Layer 6)

15-E-6.3 Calculations for Reinforcement Rupture Limit State (Strength I)

Since no live load is present, the required long-term reinforcement strength (T_{al}) for Layer 6 is computed using Equation 15-E-5.

The minimum required value of T_{al} and T_{ult} for Layer 6, on a strength per unit of reinforcement width basis (i.e., this value is what would be compared to the MARV of the tensile strength of specific reinforcement products), is therefore:

$$T_{al} \geq \frac{\gamma_{p-EV} T_{max}}{\phi_{rr} R_c} = \frac{1.35 \times 0.199 \frac{kips}{ft}}{0.80 \times 1.0} = 0.338 \frac{kips}{ft}$$

$$T_{ult} = T_{al} RF_{ID} RF_{CR} RF_D = 0.338 \times 1.12 \times 1.5 \times 1.3 = 0.737 \frac{kips}{ft}$$

For layer 6, T_{al} for reinforcement rupture is significantly less than T_{al} needed to achieve the stiffness required for the soil failure limit state (i.e., 0.338 kips/ft << 1.13 kips/ft).

15-E-6.4 Calculations for Connection Strength Design (Strength I)

To determine the minimum tensile strength needed at the connection to the facing, connection strength data for the facing block – geosynthetic combinations anticipated are needed. It has been assumed for this example that a mechanical type connection between the facing blocks and geogrid will be used (i.e., not dominated by friction).

For the purposes of this example and to be consistent with the approach taken in the current AASHTO (2020) specifications, the load and resistance factors for the connection rupture limit state are assigned the same values as those used for geosynthetic rupture limit state at locations away from the connection (i.e., $\gamma_{p-EVc} = \gamma_{p-EV}$ =1.35 and $\phi_{cr} = \phi_{rr}$ = 0.80), and $T_o = T_{max}$.

To determine the long-term connection strength available, since a mechanical connection is used in this example, the connection strength will not be a function of the normal load on the facing blocks. Expressed as a portion of T_{lot}, the roll specific tensile strength for the connection testing, the short-term connection strength $CR_u = {}^{T_{ultconn}}/_{T_{lot}}$ = 0.75. For this example, it will be assumed that this value of CR_u is applicable for all geogrids (this is likely not the case, but to keep the example as simple as possible, this assumption is made).

Equation 15-E-9 is used to calculate the minimum long-term connection strength needed, T_{ac}(required), which essentially is the factored connection load. For the purposes of this example and to be consistent with the approach taken in the current AASHTO (2020) specifications, the load and resistance factors for the connection rupture limit state are assigned the same values as those used for geosynthetic rupture limit state at locations away from the connection (i.e., $\gamma_{p\text{-}EVc} = \gamma_{p\text{-}EV} = 1.35$ and $\phi_{cr} = \phi_{rr} = 0.80$), and $T_o = T_{max}$.

Therefore, using the load side of the connection limit state design Equation 15-E-9, at Layer 6, the factored connection load is calculated as follows:

$$T_{ac}(required) = \left(\gamma_{p-EVc}\right)T_0 = (1.35) \times 0.199\frac{kips}{ft} = 0.269\frac{kips}{ft} \text{ of wall width}$$

To determine the long-term connection strength available, since a mechanical connection is used in this example, the connection strength will not be a function of the normal load on the facing blocks, N. Using the resistance side of Equation 15-E-9 (i.e., the limit state equation for connection design), and the equation for T_{ac} (Equation 15-E-11), the available long-term connection strength available is calculated as follows, assuming that the minimum T_{ult} needed is equal to the T_{ult} needed to obtain the stiffness required to meet the soil failure limit state:

$$T_{ac}(available) = \phi_{cr} T_{ac} R_c = \frac{\phi_{cr} \times T_{ult} \times \left(\frac{CR_u}{RF_{CR}}\right) R_c}{RF_D} \tag{15-E-31}$$

$$T_{ac}(available) = \frac{0.80 \times 2.47 kips/ft \times \left(\frac{0.75}{1.5}\right) 1.0}{1.3} = 0.760 \text{ kips/ft}$$

0.269 kips/ft < 0.760 kips/ft? OK

Combining Equation 15-E-9 with Equation 15-E-31 and solving for T_{ult}, at layer 6, can determine the minimum T_{ult} required to just satisfy connection strength requirements as follows:

$$T_{ult}(min.required) = \left(\frac{\gamma_{p-EVc}}{\phi_{cr} R_c}\right) T_0 RF_D RF_{CR} \left(\frac{1}{CR_u}\right) \tag{15-E-32}$$

$$T_{ult}(min.required) = \left(\frac{1.35}{0.8 \times 1.0}\right) \times 0.199 \times 1.3 \times 1.5 \left(\frac{1}{0.75}\right) = 0.873\frac{kips}{ft}$$

On a strength per unit of reinforcement width basis (and also strength per unit of wall width, since $R_c = 1.0$), this minimum required geosynthetic T_{ult} of 0.873 kips/ft is below the T_{ult} value of 2.47 kips/ft (i.e., a $T_{al} = 2.47/2.18 = 1.13$ kips/ft) estimated to provide the needed stiffness for the soil failure limit state. Therefore, the soil failure limit state is still controlling the tensile strength required at this point (i.e., only considering the Service and Strength Limit States).

15-E-6.5 Calculations for Pullout Limit State Design (Strength I)

The default pullout parameters α and F* specified in AASHTO (2020) are used for this example design and are the same as in Example 1 (α = 0.8 and F* = 0.452). The vertical stress, σ_v, over the reinforcement anchorage length, conservatively calculated using Equation 15-E-29, is the same as in Example 1 (0.173 ksf). Since pullout is usually most critical near the wall top, this example pullout calculation is carried out for Layer 10.

As before, R_c = 1.0. Using Equation 15-E-30, the required factored length of reinforcement in the resistant zone (i.e., behind the active wedge) for Layer 10 is calculated as follows:

$$L_e = \frac{\gamma_{p-EV} T_{max}}{\phi_{po} C(\alpha F^*)\sigma_v R_c}$$

$$L_e = \frac{1.35 \times 0.044 \frac{kips}{ft}}{0.70 \times 2(0.8 \times 0.452)(0.173\ ksf)1.0} = 0.678\ ft$$

To determine the total reinforcement length needed, L, the length of reinforcement within the active zone, L_a, must be calculated. For geosynthetic walls, the active zone is assumed to be bounded by the Rankine active zone (wedge). L_a is calculated as follows for a vertical wall (at layer 10):

$$L_a = (H - z)\tan(45° - \phi_r/2) = (20\ \text{ft} - 1.33\ \text{ft})\tan(45° - 34°/2) = 9.93\ \text{ft}$$

The minimum length allowed for L_e is 3 ft (AASHTO 2020), which is greater than the calculated L_e required for pullout for layer 10. Therefore, using L_e = 3 ft, the total reinforcement length required for layer 10 is:

$$L = L_a + L_e = 9.93\ \text{ft} + 3\ \text{ft} = 12.9\ \text{ft}$$

Pullout calculation results for the other layers are summarized in Table 15-E-10.

It should be recalled that the minimum length of reinforcement for typical reinforced soil walls is 0.7H = 0.7×(20 ft) = 14 ft. Therefore, pullout does not control the reinforcement length required. Note that other limit state design calculations can result in greater reinforcement length, such as external stability (e.g., sliding) or global stability.

15-E-6.6 Calculations for Determination of T_{max} + T_{md} (Extreme Event I - Seismic)

The calculation of T_{max} as described and carried out earlier in this example using the Stiffness Method is also applicable for seismic design for the static portion of the reinforcement load. T_{md}, the incremental dynamic inertia force per reinforcement layer, must be added to T_{max} to determine the total reinforcement load for each layer during seismic loading.

T_{md} is calculated using equations 15-E-17 through 15-E-19 as follows:

A_s = PGA x F_{pga} = 0.50g (same is in previous example)

Assume a maximum lateral deflection of 2 inches is allowed/anticipated. Therefore,

$$k_h = 0.74A_s\left(\frac{A_s}{d}\right)^{0.25} = 0.74 \times 0.50g \times \left(\frac{0.50}{2}\right)^{0.25} = 0.262g$$

Including the weight of the facing blocks, using Equation 15-E-19,

$$P_i = k_h\left(\gamma_r \times A_{active} + \gamma_{facing} \times T_f \times H\right)$$

$$= 0.262 \times \left(0.130\ kcf\ \times 0.5 \times \left(20\ ft \times Tan\left(45 - \frac{34^o}{2}\right)\right) \times 20\ ft + 0.12\ kcf \times 1\ ft \times 20\ ft\right) = 4.24\frac{kips}{ft}$$

Using Equation 15-E-17,

$$T_{md} = \left(\frac{P_i}{n}\right) = \frac{4.24\frac{kips}{ft}}{10} = 0.424\frac{kips}{ft}$$

The total load per reinforcement layer, on a load per unit of wall width basis, during seismic shaking, T_{totalf}, is then calculated using superposition as follows, for Layer 6:

$$T_{totalf} = \gamma_{seis}(T_{max} + T_{md}) = 1.0\left(0.199\frac{kips}{ft} + 0.424\frac{kips}{ft}\right) = 0.623\frac{kips}{ft}$$

For seismic pullout design, T_{totalf} is used. However, for reinforcement rupture and connection rupture under seismic loading, the static and dynamic components of the reinforcement load must be handled separately. The reason for this is that the strength required to resist T_{max} must include the effects of creep because it is a sustained load, but the strength required to resist T_{md} should not include the effects of creep due to the transient nature of T_{md}.

15-E-6.7 Calculations for Reinforcement Rupture (Extreme Event I - Seismic)

Using equations 15-E-21, 15-E-22, and 15-E-23, calculate T_{ult} for static portion of load at Layer 6:

$$S_{rs} = \frac{\gamma_{seis} T_{max} RF}{\phi R_c} = \frac{1.0 \times 0.199 \frac{kips}{ft} \times 2.18}{1.0 \times 1.0} = 0.434 \frac{kips}{ft}$$

T_{ult} for dynamic portion of load at Layer 6:

$$S_{rt} = \frac{\gamma_{seis} T_{md} RF_{ID} RF_D}{\phi R_c} = \frac{1.0 \times 0.424 \frac{kips}{ft} \times 1.12 \times 1.3}{1.0 \times 1.0} = 0.617 \frac{kips}{ft}$$

T_{ult} = S_{rs} + S_{rt} = 0.434 kips/ft + 0.617 kips/ft = 1.05 kips/ft of reinforcement unit width

T_{al} = 1.05 kips/ft/2.18 = 0.483 kips/ft of reinforcement unit width

15-E-6.8 Calculations for Connection Rupture (Extreme Event I - Seismic)

Using equations 15-E-24, 15-E-25, and 15-E-26, T_{ult} for static portion of load at Layer 6 is determined as follows:

$$CR_{cr} = \frac{CR_u}{RF_{CR}} = \frac{0.75}{1.5} = 0.500$$

Since this is a mechanical connection, F_r is set equal to 1.0.

$$S_{rsc} = \frac{\gamma_{seis} T_0 RF_D}{F_r \phi CR_{cr} R_c} = \frac{1.0 \times 0.199 \frac{kips}{ft} \times 1.3}{1.0 \times 1.0 \times 0.500 \times 1.0} = 0.517 \frac{kips}{ft}$$

$$S_{rtc} = \frac{\gamma_{seis} T_{md} RF_D}{F_r \phi CR_u R_c} = \frac{1.0 \times 0.424 \frac{kips}{ft} \times 1.3}{1.0 \times 1.0 \times (0.75) \times 1.0} = 0.735 \frac{kips}{ft}$$

T_{ult} = S_{rsc} + S_{rtc} = 0.517 kips/ft + 0.735 kips/ft = 1.25 kips/ft of reinforcement product width

On a strength per reinforcement width basis, this minimum required geosynthetic T_{ult} of 1.25 kips/ft is still below the T_{ult} value of 2.47 kips/ft (i.e., a T_{al} = 2.47/2.18 = 1.13 kips/ft) estimated to provide the needed stiffness for the soil failure limit state. So the soil failure limit state is still controlling the tensile strength required considering the Extreme Event I Limit State (i.e., seismic).

15-E-6.9 Calculations for Pullout Limit State Design (Extreme Event I - Seismic)

Using Equation 15-E-27, L_e for Layer 10 (i.e., at the wall top) is determined as follows:

$$L_e = \frac{\gamma_{seis}(T_{max} + T_{md})}{\phi C(0.8\alpha F^*)\sigma_v R_c} = \frac{1.0(0.044 + 0.424)}{1.0 \times 2 \times (0.8 \times 0.8 \times 0.452) \times 0.173\ kcf \times 1.0}$$

$L_e = 4.68$ ft

$L_a = (H - z) \tan(45° - \phi_r/2) = (20\text{ ft} - 1.33\text{ ft}) \tan(45° - 34°/2) = 9.93$ ft

$L = L_a + L_e = 9.93\text{ ft} + 4.68\text{ ft} = 14.6$ ft

This required length is just barely greater than 70% of the wall height, so it does control pullout length at the wall top.

15-E-6.10 Calculations to Check to Make Sure Stiffness and Strength Required are Properly Matched (Extreme Event I, Seismic)

Since the soil failure limit state is still controlling the design, the stiffness determined to meet the soil failure limit state is still the correct stiffness to use. Therefore, an additional iteration with stiffness values that are consistent with the tensile strengths needed is not required. Had one of the other limit states controlled the T_{ult} needed, then it would have been necessary to recheck the design using an increased stiffness value that is consistent with the higher tensile strength. Fortunately, this does not happen very often (may only occur for block faced walls with very inefficient connections between the geosynthetic and the facing blocks).

15-E-6.11 Summary for Example 2 Design

See Table 15-E-10 for the calculation results for all of the layers for the Service I, Strength I, and Extreme Event I limit states. These calculation results are plotted and compared to design calculation results using the Simplified Method in figures 15-E-6 through 15-E-8.

In summary, for the final internal stability design for Example 2 using the Stiffness Method, the following would be specified with regard to reinforcement properties and spacing for the optimized design, and the following observations can be made:

- The vertical spacing of reinforcements is 2 ft throughout the wall height, and the coverage ratio is a minimum of 1.0.
- The minimum reinforcement length for internal stability (i.e., pullout) is 14 ft for the Strength Limit, and all layers except the top layer for seismic did not exceed this minimum length. The top layer length required is 14.6 ft for seismic for the Stiffness Method, which is greater than the 0.7H minimum (see Figure 15-E-8). The longer reinforcement length due to pullout is consistent with good detailing for MSE walls in AASHTO LRFD Article 11.10.7.4.

- The lowest strength PET geogrid available was greater in strength and stiffness than required by the Stiffness Method design for the top 4 layers (see figures 15-E-6 and 15-E-7). This was not the case for the Simplified Method, as higher reinforcement strengths than the minimum available were required for most of the layers.
- The Simplified Method required a total long-term tensile strength T_{al} of 13.2 kips/ft for the entire wall section, whereas the Stiffness Method required 5.4 kips/ft for the entire wall section for seismic reinforcement connection rupture. However, the T_{al} needed to obtain the stiffness needed to meet the soil failure limit state controlled the design, for which the total T_{al} for the wall section was 9.4 kips/ft, which is just over 70% of the total tensile strength needed by the Simplified Method. Note that in the upper third of the wall, all limit states for both methods will be limited to the minimum strength shown in the plots as a dashed vertical line. If that is considered, the Simplified Method T_{al} required would increase to 13.9 kips/ft, making the Stiffness Method required soil reinforcement strength equal to 68% of what is required by the Simplified Method.
- Overall, the required strength and stiffness of the reinforcement needed is in the low to mid-range of the geogrid product lines available. Note that the ground acceleration used for the seismic design represents what would be included in Seismic Zone 4, which is the highest seismic zone.

Table 15-E-10 Summary of Example 2 wall design calculations using Stiffness Method and R_c = 1.0 (Service, Strength, and Extreme Event I Limit States): (a) Calculation of T_{max}, (b) Service and Strength Limit State calculations, and (c) Extreme Event I (seismic) Limit State calculations.

a)	T_{max} Equation (Eq. 15-E-1) Parameters											Unfactored
Layer Number	**z (ft)**	**S_v (ft)**	**R_cJ_i (kips/ft)**	**S_{global} (ksf)**	**S_{local} (ksf)**	**D_{tmax}**	**F_f**	**Φ_g**	**Φ_{local}**	**Φ_{fb}**	**Φ_{fs}**	***+T_{max} and T_0 (kips/ft)**
10 (top)	1.33	2.33	8.6	6.07	3.69	0.220	1.61	0.211	0.77	1.0	0.718	0.044
9	3.33	2.00	8.6	6.07	4.30	0.372	1.61	0.211	0.84	1.0	0.718	0.069
8	5.33	2.00	8.6	6.07	4.30	0.523	1.61	0.211	0.84	1.0	0.718	0.097
7	7.33	2.00	8.6	6.07	4.30	0.674	1.61	0.211	0.84	1.0	0.718	0.125
6	9.33	2.00	14.5	6.07	7.25	0.825	1.61	0.211	1.09	1.0	0.718	0.199
5	11.33	2.00	14.5	6.07	7.25	0.976	1.61	0.211	1.09	1.0	0.718	0.235
4	13.33	2.00	14.5	6.07	7.25	1.00	1.61	0.211	1.09	1.0	0.718	0.241
3	15.33	2.00	14.5	6.07	7.25	1.00	1.61	0.211	1.09	1.0	0.718	0.241
2	17.33	2.00	14.5	6.07	7.25	1.00	1.61	0.211	1.09	1.0	0.718	0.241
1	19.33	1.67	14.5	6.07	8.68	1.00	1.61	0.211	1.19	1.0	0.718	0.220
Base of wall	20											$\sum T_{max}$= 1.71

b)			Reinforcement Rupture (Strength Limit)		Connection Rupture (Strength Limit)			Soil Failure (Service Limit)			Pullout (Strength Limit)	
			#Minimum Required Strength per Unit Width of Reinforcement			#Minimum Required Strength per Unit Width of Reinforcement					Anchorage length L_e (ft)	
Layer Number	**z (ft)**	**Reinforcement Product**	**T_{al} (kips/ft)**	**T_{ult} (kips/ft)**	**Connection Capacity as Fraction of T_{lot}, CR_u**	**T_{al} (kips/ft)**	**T_{ult} (kips/ft)**	**Factored ε_{rein} (%)**		**#T_{al} Corresponding to J_i (kips/ft)**	**Required**	**Min. allowed**
10 (top)	1.33	Geogrid a	0.075	0.16	0.75	0.09	0.19	0.62		0.67	0.68	
9	3.33	Geogrid a	0.12	0.25	0.75	0.14	0.30	0.96		0.67	0.43	
8	5.33	Geogrid a	0.16	0.36	0.75	0.19	0.43	1.35		0.67	0.37	
7	7.33	Geogrid a	0.21	0.46	0.75	0.25	0.55	1.75		0.67	0.35	
6	9.33	Geogrid b	0.34	0.73	0.75	0.40	0.87	1.65	≤ 2.0%	1.13	0.44	< 3.0
5	11.33	Geogrid b	0.40	0.87	0.75	0.47	1.03	1.95	(OK)	1.13	0.43	(OK)
4	13.33	Geogrid b	0.41	0.89	0.75	0.48	1.06	2.00		1.13	0.37	
3	15.33	Geogrid b	0.41	0.89	0.75	0.48	1.06	2.00		1.13	0.32	
2	17.33	Geogrid b	0.41	0.89	0.75	0.48	1.06	2.00		1.13	0.29	
1	19.33	Geogrid b	0.37	0.81	0.75	0.44	0.97	1.82		1.13	0.23	
Base of wall	20		$\sum T_{al}$= 2.89	$\sum T_{ult}$= 6.31		$\sum T_{al}$= 3.44	$\sum T_{ult}$= 7.52			$\sum T_{al}$= 9.4		

Table 15-E-10, continued

Summary of Example 2 wall design calculations using Stiffness Method and R_c = 1.0 (Service, Strength, and Extreme Event I Limit States): (c) Extreme Event I (seismic) Limit State calculations.

c)		Reinforcement Product	Soil Failure (Service Limit, for Comparison)	Reinforcement Rupture (Extreme Event I Limit)		Connection Rupture (Extreme Event I Limit)			Pullout (Extreme Event I Limit)		
Layer Number	z (ft)		#Tensile Strength T_{al} Corresponding to Reinforcement Stiffness Required (kips/ft)	#Minimum Required Strength per Unit Width of Reinforcement		Connection Capacity as Fraction of T_{lot}, CR_u	#Minimum Required Strength per Unit Width of Reinforcement		Anchorage length L_{eseis} (ft)		Total Reinforcement Length (ft)
				T_{al} (kips/ft)	$T_{ult} = T_{al} \times RF$ (kips/ft)		T_{al} (kips/ft)	T_{ult} (kips/ft)	Required	Min. allowed	(min. is 0.7H = 14 ft)
10 (top)	1.33	Geogrid a	0.67	0.33	0.71	0.75	0.39	0.85	4.68	< 3.0 (No, so pullout controls length at wall top)	14.6
9	3.33	Geogrid a	0.67	0.35	0.77	0.75	0.42	0.92	1.97		10.8
8	5.33	Geogrid a	0.67	0.38	0.83	0.75	0.45	0.99	1.30		9.1
7	7.33	Geogrid a	0.67	0.41	0.89	0.75	0.49	1.06	1.00		7.7
6	9.33	Geogrid b	1.13	0.48	1.05	0.75	0.57	1.25	0.89		6.6
5	11.33	Geogrid b	1.13	0.52	1.13	0.75	0.62	1.35	0.77		5.4
4	13.33	Geogrid b	1.13	0.52	1.14	0.75	0.62	1.36	0.66		4.2
3	15.33	Geogrid b	1.13	0.52	1.14	0.75	0.62	1.36	0.58		3.1
2	17.33	Geogrid b	1.13	0.52	1.14	0.75	0.62	1.36	0.51		1.9
1	19.33	Geogrid b	1.13	0.50	1.10	0.75	0.60	1.31	0.44		0.8
Base of wall	20		$\sum T_{al}$= 9.4	$\sum T_{al}$= 4.54	$\sum T_{ult}$= 9.92		$\sum T_{al}$= 5.41	$\sum T_{ult}$= 11.8			

+These values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (AASHTO 2020).

#For comparison to geogrid product MARV tensile strength that is not reduced by Rc (i.e., load per unit of reinforcement width basis).

*T_{md} for all reinforcement layers is 0.424 kips/ft.

Figure 15-E-6 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Service and Strength limit states, block faced wall with mechanical connection, R_c = 1.0 (Example 2).

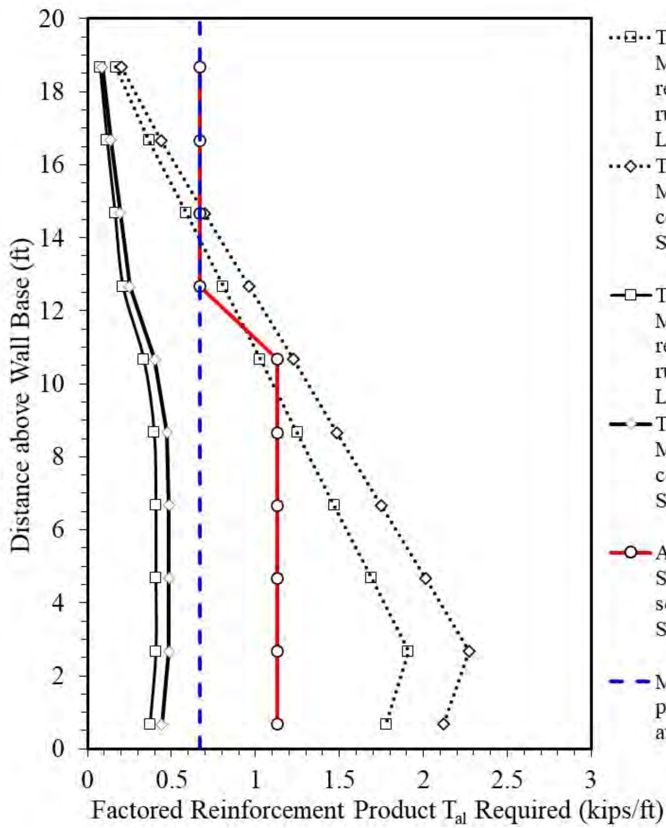


Figure 15-E-7 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Service and Extreme Event I (seismic) limit states, block faced wall with mechanical connection, R_c = 1.0 (Example 2).

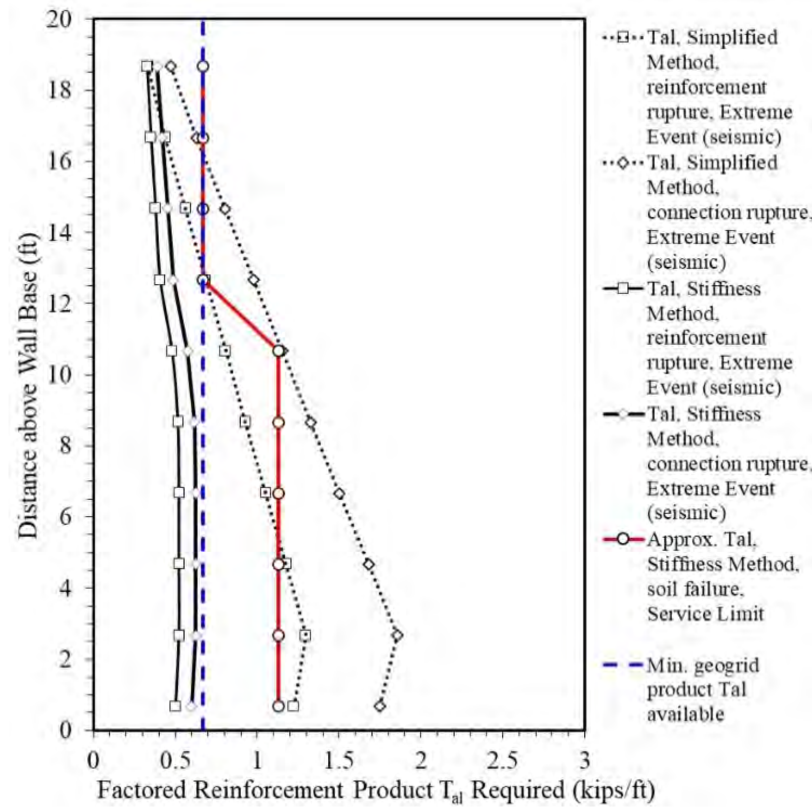
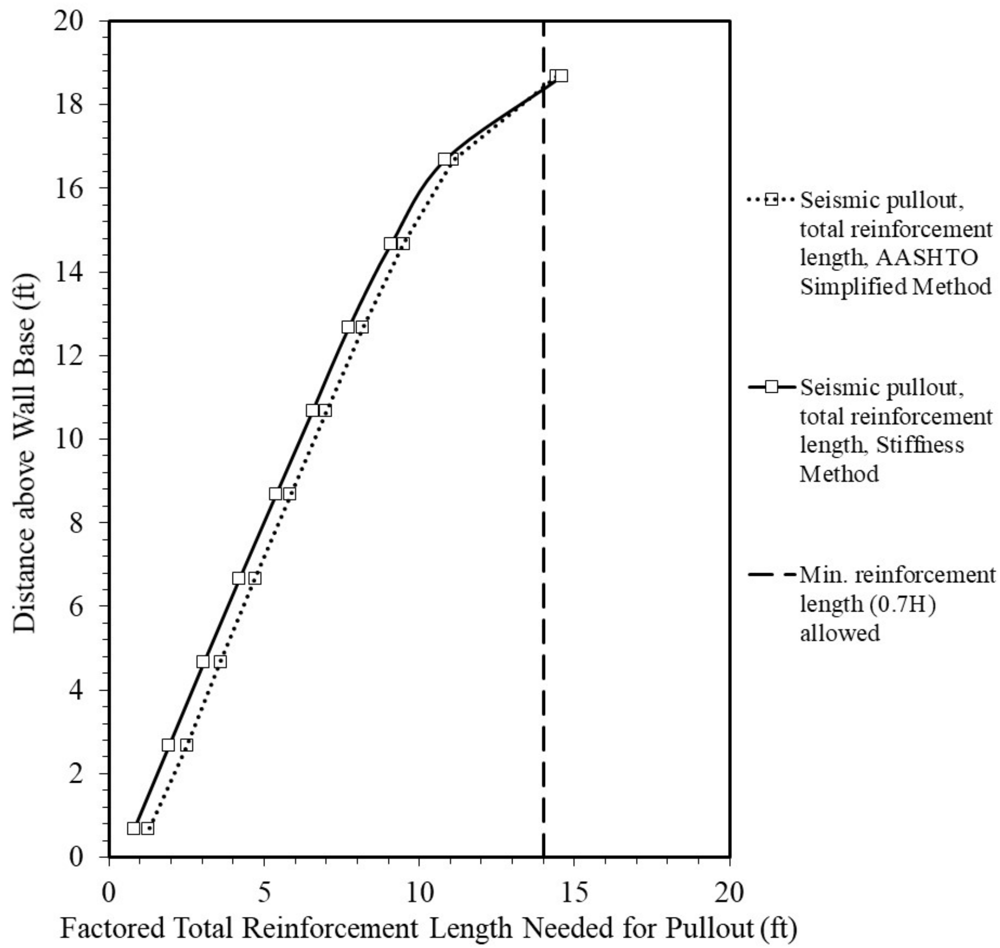


Figure 15-E-8 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Extreme Event I (seismic) limit state, pullout, block faced wall with mechanical connection, R_c = 1.0 (Example 2).



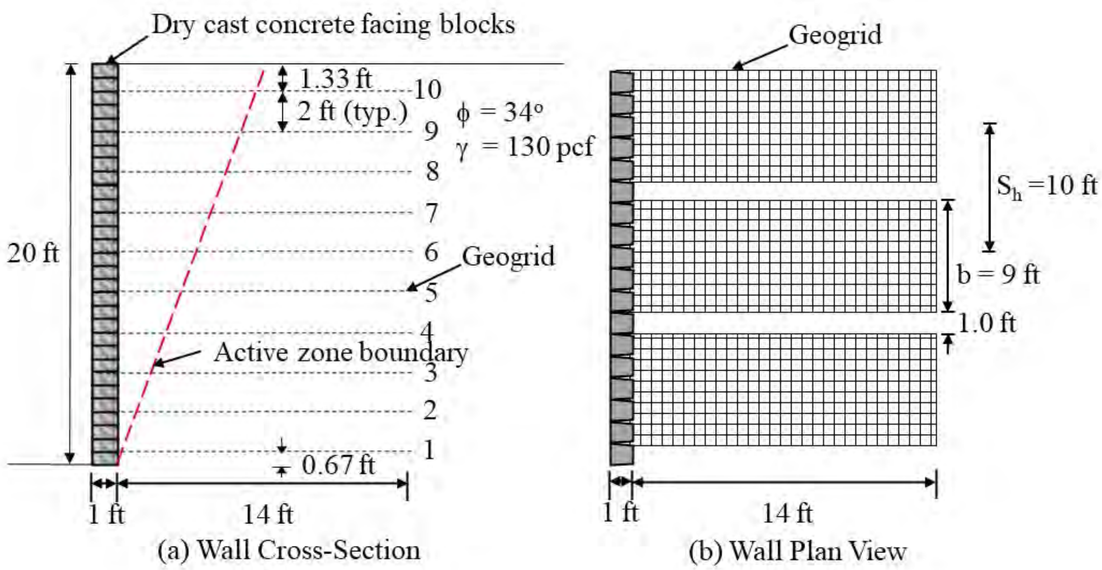
15-E-7 Stiffness Method Design Example 3: Block Faced Geosynthetic Wall with Mechanical Connections, R_c = 0.9

15-E-7.1 General

Figure 15-E-9 shows a cross-section of the wall for this design example. The procedures and results for the case when the coverage ratio R_c for the reinforcement is less than 1.0 (in this example R_c is set equal to 0.90) are illustrated. Material properties are provided in Table 15-E-7, except that the minimum geogrid stiffness available on a stiffness per unit of wall width basis is now reduced using R_c = 0.90 to J_iR_c = 8.6 x 0.9 = 7.7 kips/ft. All other aspects of this example are the same as Example 2.

As is true of Example 2, the scope of Example 3 is limited to the service (soil failure limit only), strength, and Extreme Event I (seismic) limit states. Furthermore, for simplicity, live loads are not included in the calculations to follow. It is also assumed that the foundation soil has sufficient shear strength to prevent global instability, bearing capacity failure and excessive settlement. For seismic design, assume that the ground acceleration, A_s, is 0.50g.

Figure 15-E-9 Wall geometry and preliminary PET reinforcement layout for Design Example 3



The wall geometry is based on Figure 15-E-9. As is true for Example 2, Example 3 calculations are demonstrated for one of the middle layers (i.e., Layer 6 in Figure 15-E-10, and for the soil failure and pullout limit states, Layer 10 is also used to illustrate calculations).

S = equivalent uniform height of surcharge = 0 ft

K_{avh} and K_{abh} remain unchanged relative to Example 1 and Example 2 at 0.283, and Φ_{fb} = 1.0.

The reinforcement stiffness values used in the calculations to follow need to be adjusted to account for the reinforcement coverage ratio, R_c of 0.90.

Since the soil failure limit state usually controls the amount of reinforcement required, this limit state is evaluated first, and T_{max} is calculated, for the wall design.

15-E-7.2 Calculations for Soil Failure Limit State Design (Service I)

As is true for Example 2, for Example 3 the factored reinforcement peak strain for each layer should be 2.0% or less for a modular block faced wall (i.e., stiff face) in this example. Some trial-and-error is typically required to establish what reinforcement stiffness values are required, starting with the stiffness values required to meet the soil failure limit, as the soil failure limit often controls the design of geosynthetic walls. Using trial-and-error, use the minimum 1,000-hour 2% strain secant stiffness available for geosynthetic reinforcement products, J_i = 7.7 kips/ft for the top 4 layers, and increase the stiffness of the bottom 6 layers to 14.5 kips/ft, as shown in Table 15-E-11.

Table 15-E-11 Creep stiffness values (i.e., at 2% strain and 1,000 h) for geogrids used in wall with R_c = 0.90.

Layer No.	Geogrid Designation	J_i (per Unit Width of Reinforcement, in kips/ft)	*R_c x J_i (per Unit Width of Wall, in kips/ft)
10	a	8.6	7.7
9	a	8.6	7.7
8	a	8.6	7.7
7	a	8.6	7.7
6	b	16.1	14.5
5	b	16.1	14.5
4	b	16.1	14.5
3	b	16.1	14.5
2	b	16.1	14.5
1	b	16.1	14.5

*This is the stiffness value, corrected for R_c, which is used to calculate T_{max}.

The contributing factors, coefficients and parameters that comprise the T_{max} equation can be found in Allen and Bathurst (2015) and Table 15-E-1. To calculate T_{max}, the reinforcement stiffness must be per unit of wall width rather than per unit of reinforcement product width; hence, R_cJ_i must be used to calculate T_{max} where the reinforcement stiffness value is required. Using the Table 15-E-11 stiffness values, considering a total of 10 layers, the parameters used to determine T_{max} are calculated as follows:

$$S_{\text{global}} = \frac{J_{\text{ave}}}{(\text{H/n})} = \frac{\sum_{i=1}^{n} R_c J_i}{H} = (4 \times 0.9 \times 8.6 \text{ kips/ft} + 6 \times 0.9 \times 16.1 \text{ kips/ft})/20 \text{ ft} = 5.89 \text{ ksf}$$

$$\Phi_g = \alpha \left(\frac{S_{\text{global}}}{p_a}\right)^{\beta} = 0.16 \times (5.89 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.209$$

S_{local} is calculated as:

$$S_{\text{local}} = \left(\frac{R_c J}{S_v}\right)_i = (0.9 \times 16.1 \text{ kips/ft})/(2.0 \text{ ft}) = 7.25 \text{ ksf for Layer 6}$$

$$S_{localave} = \frac{\sum \left({R_c J}/{S_v}\right)_i}{n} = \frac{3.30 + 3 \times 3.85 + 5 \times 7.25 + 8.68}{10} = 5.98\ ksf$$

where, n = 10 is the number of reinforcement layers. Therefore, Φ_{local} is recalculated as follows:

$$\Phi_{local} = \left(\frac{S_{local}}{S_{localave}}\right)^{0.5} = \left(\frac{7.25\ ksf}{5.98\ ksf}\right)^{0.5} = 1.10\ \ (Layer\ 6)$$

To determine the facing stiffness factor, the facing stiffness parameter, F_f does not change, and the facing stiffness factor is calculated as:

$$\Phi_{fs} = \eta \left(\left(\frac{S_{global}}{p_a}\right) F_f\right)^{\kappa} = 0.57 \times \left(\left(\frac{5.89\ ksf}{2.11\ ksf}\right) \times 1.61\right)^{0.15} = 0.714$$

Since c = 0, the cohesion factor, Φ_c = 1.0.

D_{tmax} does not change relative to the previous calculation (i.e., D_{tmax} for Layer 6 is 0.825).

T_{max} for Layer 6 is now recalculated using the updated values as follows:

$$T_{max} = S_v \left[H\gamma_r D_{tmax} + \left(\frac{H_{ref}}{H}\right) S\gamma_f + LL\right] K_{avh} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c$$

$$T_{max} = 2.0\ ft \left[20ft \times 0.130\ kcf \times 0.825 + \left(\frac{20\ ft}{20\ ft}\right) 0\ ft \times 0.130\ kcf + 0\right] 0.283 \times 1.0 \times 0.209 \times 0.714 \times 1.10 \times 1.0$$

T_{max} = 0.199 kips/ft of wall width

T_{max}, and the calculated parameters needed to calculate T_{max}, are summarized in Table 15-E-12 for the rest of the layers for the Service and Strength limit states.

Using Equation 15-14 with load factor γ_{sf} = 1.2, and resistance factor ϕ_{sf} = 1.0 (Table 15-5), the factored reinforcement strain corresponding to Layer 10 is computed as:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.042 \frac{kips}{ft}}{1.0 \times 0.9 \times 8.6 \frac{kips}{ft}} \times 100\% = 0.65\% \leq 2\% \quad OK$$

For Layer 6:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.199 \frac{kips}{ft}}{1.0 \times 0.9 \times 16.1 \frac{kips}{ft}} \times 100\% = 1.65\% \leq 2\% \quad OK$$

See Table 15-E-12 for the calculation results for the rest of the layers. As shown in the table, the soil failure limit state is met by all the reinforcement layers.

As shown for examples 1 and 2, the equivalent ultimate tensile strength that corresponds to the stiffness values needed to meet the soil failure limit state is equal to approximately $0.17J_i$. For the two stiffness values required to meet the soil failure limit state, the approximate ultimate tensile strength per ft of wall width needed is:

$T_{ult} = 0.17J_i = 0.17 \text{ x } (8.6 \text{ kips/ft}) = 1.46 \text{ kips/ft}$ (applicable to Layer 10)

$T_{ult} = 0.17 \text{ x } (16.1 \text{ kips/ft}) = 2.74 \text{ kips/ft}$ (applicable to Layer 6)

To determine T_{al}, divide T_{ult} by RF = 1.12 x 1.5 x 1.3 = 2.18. Therefore, the approximate long-term tensile strengths implied by the stiffness required for the soil failure limit state for the reinforcement product (i.e., per ft of wall width) are:

$T_{al} = (1.46 \text{ kips/ft})/2.18 = 0.67 \text{ kips/ft}$ (applicable to Layer 10)

$T_{al} = (2.74 \text{ kips/ft})/2.18 = 1.26 \text{ kips/ft}$ (applicable to Layer 6)

15-E-7.3 Calculations for Reinforcement Rupture Limit State (Strength I)

Since no live load is present, the required long-term reinforcement strength (T_{al}) for Layer 6 is computed using Equation 15-E-5.

The minimum required value of T_{al} and T_{ult} for Layer 6, on a strength per unit of reinforcement width basis (i.e., this value is what would be compared to the MARV of the tensile strength of specific reinforcement products), is therefore:

$$T_{al} \geq \frac{\gamma_{p-EV} T_{max}}{\phi_{rr} R_c} = \frac{1.35 \times 0.199 \frac{kips}{ft}}{0.80 \times 0.90} = 0.373 \frac{kips}{ft}$$

$$T_{ult} = T_{al} RF_{ID} RF_{CR} RF_D = 0.373 \times 1.12 \times 1.5 \times 1.3 = 0.815 \frac{kips}{ft}$$

For Layer 6, T_{al} for reinforcement rupture is significantly less than T_{al} needed to achieve the stiffness required for the soil failure limit state (i.e., 0.373 kips/ft << 1.26 kips/ft).

15-E-7.4 Calculations for Connection Strength Design (Strength I)

With regard to connection strength, the same value as was used in Example 2 is used for Example 3 (i.e., a mechanical type connection between the facing blocks and geogrid with a CR_u of 0.75). For the connection design, it is assumed that $\gamma_{p\text{-}EVc} = \gamma_{p\text{-}EV} = 1.35$, $\phi_{cr} = \phi_{rr} = 0.80$, and $T_o = T_{max}$. It will also be assumed that this value of CR_u is applicable for all geogrids.

Therefore, using the load side of the connection limit state design Equation 15-E-9, at Layer 6, the factored connection load is calculated as follows:

$$T_{ac}(required) = (\gamma_{p-EVc})T_0 = (1.35) \times 0.199\frac{kips}{ft} = 0.269\frac{kips}{ft} \text{ of wall width}$$

To determine the long-term connection strength available, since a mechanical connection is used in this example, the connection strength will not be a function of the normal load on the facing blocks, N. Using the resistance side of Equation 15-E-9 (i.e., the limit state equation for connection design), and the equation for T_{ac} (Equation 15-E-11), the available long-term connection strength available is calculated as follows, assuming that the minimum T_{ult} needed is equal to the T_{ult} needed (strength per unit of wall width) to obtain the stiffness required to meet the soil failure limit state:

$$T_{ac}(available) = \phi_{cr} T_{ac} R_c = \frac{\phi_{cr} \times T_{ult} \times \left(\frac{CR_u}{RF_{CR}}\right) R_c}{RF_D} \qquad (15\text{-E-}33)$$

$$T_{ac}(available) = \frac{0.80 \times 2.47 kips/ft \times \left(\frac{0.75}{1.5}\right) 0.9}{1.3} = 0.684 \text{ kips/ft}$$

$$0.269 \text{ kips/ft} < 0.684 \text{ kips/ft?} \qquad \text{OK}$$

Combining Equation 15-E-9 with Equation 15-E-33 and solving for T_{ult}, at layer 6, can determine the minimum T_{ult} required to just satisfy connection strength requirements as follows:

$$T_{ult}(min.required) = \left(\frac{\gamma_{p-EVc}}{\phi_{cr} R_c}\right) T_0 RF_D RF_{CR} \left(\frac{1}{CR_u}\right)$$

$$T_{ult}(min.required) = \left(\frac{1.35}{0.8 \times 0.90}\right) \times 0.199 \times 1.3 \times 1.5 \left(\frac{1}{0.75}\right) = 0.970\frac{kips}{ft}$$

The only difference between examples 2 and 3 regarding the connection strength and the T_{ult} needed to meet connection strength requirements is R_c (i.e., 0.873/0.970 = 0.90). These calculations demonstrate that R_c has been handled correctly.

On a strength per unit of reinforcement width basis, this minimum required geosynthetic T_{ult} of 0.970 kips/ft is below the T_{ult} value of 2.74 kips/ft (i.e., a T_{al} = 2.74/2.18 = 1.26 kips/ft) estimated to provide the needed stiffness for the soil failure limit state. So the soil failure limit state is still controlling the tensile strength required at this point (i.e., only considering the Service and Strength Limit States).

15-E-7.5 Calculations for Pullout Limit State Design (Strength I)

The default pullout parameters α and F* specified in AASHTO (2020) are used for this example design and are the same as in Example 1 (α = 0.8 and F* = 0.452). R_c in this example is smaller than in the previous two examples (Rc = 0.90). The vertical stress, σ_v, over the reinforcement anchorage length, conservatively calculated using Equation 15-E-29, is the same as in Example 1 (0.173 ksf). Since pullout is usually most critical near the wall top, this example pullout calculation is carried out for Layer 10.

Using Equation 15-E-30, the required factored length of reinforcement in the resistant zone (i.e., behind the active wedge) for Layer 10 is:

$$L_e = \frac{\gamma_{p-EV} T_{max}}{\phi_{po} C(\alpha F^*) \sigma_v R_c}$$

$$L_e = \frac{1.35 \times 0.042 \dfrac{kips}{ft}}{0.70 \times 2(0.8 \times 0.452)(0.173\ ksf)0.90} = 0.719\ ft$$

To determine the total reinforcement length needed, L, the length of reinforcement within the active zone, L_a, must be calculated. For geosynthetic walls, the active zone is assumed to be bounded by the Rankine active zone (wedge). L_a is calculated as follows for a vertical wall (at Layer 10):

$$L_a = (H - z) \tan(45° - \phi_r/2) = (20\text{ ft} - 1.33\text{ ft}) \tan(45° - 34°/2) = 9.93\text{ ft}$$

The minimum length allowed for L_e is 3 ft (AASHTO 2020), which is greater than the calculated L_e required for pullout for layer 10. Therefore, using L_e = 3 ft, the total reinforcement length required for layer 10 is:

$$L = L_a + L_e = 9.93\text{ ft} + 3\text{ ft} = 12.9\text{ ft}$$

Pullout calculation results for the other layers are summarized in Table 15-E-12.

It should be recalled that the minimum length of reinforcement for typical reinforced soil walls is 0.7H = 0.7×(20 ft) = 14 ft. Therefore, pullout does not control the reinforcement length required. Note that other limit state design calculations can result in greater reinforcement length, such as external stability (e.g., sliding) or global stability.

15-E-7.6 Calculations for Determination of T_{max} + T_{md} (Extreme Event I - Seismic)

T_{md} is calculated as shown for Example 2, Layer 6, and is equal to 0.424 kips/ft. T_{totalf} is also the same as shown for Example 2 and is equal to 0.623 kips/ft of wall width.

15-E-7.7 Calculations for Reinforcement Rupture (Extreme Event I - Seismic)

Using equations 15-E-21, 15-E-22, and 15-E-23, calculate T_{ult} for static portion of load at Layer 6:

$$S_{rs} = \frac{\gamma_{seis} T_{max} RF}{\phi R_c} = \frac{1.0 \times 0.199 \frac{kips}{ft} \times 2.18}{1.0 \times 0.90} = 0.482 \frac{kips}{ft}$$

T_{ult} for dynamic portion of load at Layer 6:

$$S_{rt} = \frac{\gamma_{seis} T_{md} RF_{ID} RF_D}{\phi R_c} = \frac{1.0 \times 0.424 \frac{kips}{ft} \times 1.12 \times 1.3}{1.0 \times 0.90} = 0.686 \frac{kips}{ft}$$

$T_{ult} = S_{rs} + S_{rt}$ = 0.482 kips/ft + 0.686 kips/ft = 1.17 kips/ft of reinforcement width

T_{al} = 1.17 kips/ft/2.18 = 0.537 kips/ft of reinforcement width

The only difference between these calculated values and those calculated for Example 2 is the coverage ratio of 0.90 (i.e., for T_{al}, 0.434/0.537 = 0.90).

15-E-7.8 Calculations for Connection Rupture (Extreme Event I - Seismic)

Using equations 15-E-24, 15-E-25, and 15-E-26, T_{ult} for static portion of load at Layer 6:

$$CR_{cr} = \frac{CR_u}{RF_{CR}} = \frac{0.75 \frac{kips}{ft}}{1.5 \frac{Kips}{ft}} = 0.500$$

Since this is a mechanical connection, F_r is set equal to 1.0.

$$S_{rsc} = \frac{\gamma_{seis} T_0 RF_D}{F_r \phi CR_{cr} R_c} = \frac{1.0 \times 0.199 \frac{kips}{ft} \times 1.3}{1.0 \times 1.0 \times 0.500 \times 0.90} = 0.575 \frac{kips}{ft}$$

$$S_{rtc} = \frac{\gamma_{seis} T_{md} RF_D}{F_r \phi CR_u R_c} = \frac{1.0 \times 0.424 \frac{kips}{ft} \times 1.3}{1.0 \times 1.0 \times (0.75) \times 0.90} = 0.817 \frac{kips}{ft}$$

$T_{ult} = S_{rsc} + S_{rtc}$ = 0.575 kips/ft+0.817 kips/ft = 1.39 kips/ft of reinforcement product width.

On a strength per unit of reinforcement width basis, this minimum required geosynthetic T_{ult} of 1.39 kips/ft is still below the T_{ult} value of 2.74 kips/ft (i.e., a T_{al} = 2.74/2.18 = 1.26 kips/ft) estimated to provide the needed stiffness for the soil failure limit state. So the soil failure limit state is still controlling the tensile strength required considering the Extreme Event I Limit State (i.e., seismic).

Again, the only difference between these calculated results and those determined for Example 2 is the coverage ratio of 0.90.

15-E-7.9 Calculations for Pullout Limit State Design (Extreme Event I - Seismic)

Using Equation 15-E-27, L_e for Layer 10 (i.e., at the wall top) is determined as follows:

$\sigma_v = z\gamma_r + S_{sur}\gamma_f$

$\sigma_v = (1.33\ \text{ft})(0.13\ \text{kcf}) + (0)(\ 0.13\ \text{kcf}) = 0.173\ \text{ksf}$

$$L_e = \frac{\gamma_{seis}(T_{max} + T_{md})}{\phi C(0.8\alpha F^*)\sigma_v R_c} = \frac{1.0(0.042 + 0.424)}{1.0 \times 2 \times (0.8 \times 0.8 \times 0.452) \times 0.173\ kcf \times 0.90}$$

$L_e = 5.18\ \text{ft}$

$L_a = (H - z)\tan(45° - \phi_r/2) = (20\ \text{ft} - 1.33\ \text{ft})\tan(45° - 34°/2) = 9.93\ \text{ft}$

$L = L_a + L_e = 9.93\ \text{ft} + 5.18\ \text{ft} = 15.1\ \text{ft}$

This required length is greater than 70% of the wall height, so it does control pullout length at the wall top.

15-E-7.10 Calculations to Check to Make Sure Stiffness and Strength Required are Properly Matched (Extreme Event I, Seismic)

Since the soil failure limit state is still controlling the design, the stiffness determined to meet the soil failure limit state is still the correct stiffness to use. Therefore, an additional iteration with higher stiffness values that are consistent with the tensile strengths needed is not required. Had one of the other limit states controlled the T_{ult} needed, then it would have been necessary to recheck the design using a stiffness value that is consistent with the higher tensile strength. Fortunately, this does not happen very often (only would occur for block faced walls with very inefficient connections between the geosynthetic and the facing blocks).

15-E-7.11 Summary for Example 3 Design

See Table 15-E-12 for the calculation results for all of the layers for the Service I, Strength I, and Extreme Event I limit states. These calculation results are plotted and compared to design calculation results using the Simplified Method in figures 15-E-10 through 15-E-12.

In summary, for the final internal stability design for Example 3 using the Stiffness Method, the following would be specified with regard to reinforcement properties and spacing for the optimized design, and the following observations can be made:

- The vertical spacing of reinforcements is 2 ft throughout the wall height, and the coverage ratio is a minimum of 0.90.
- The minimum reinforcement length for internal stability (i.e., pullout) is 14 ft for the Strength Limit, and all layers except the top layer for seismic did not exceed this minimum length. The top layer length required is 15.1 ft for seismic for the Stiffness Method, which is greater than the 0.7H minimum. The longer reinforcement length due to pullout is consistent with good detailing for MSE walls in AASHTO (2020) LRFD Article 11.10.7.4.

- The lowest strength PET geogrid available was greater in strength and stiffness than required by the Stiffness Method design for the top 4 layers (see figures 15-E-11 and 15-E-12). This was not the case for the Simplified Method, as higher reinforcement strengths than the minimum available were required for most of the layers.
- The Simplified Method required a total long-term tensile strength T_{al} of 13.5 kips/ft for the entire wall section (seismic connection rupture controlled the design), whereas the Stiffness Method required only 5.99 kips/ft for the entire wall section for seismic reinforcement connection rupture. However, the T_{al} needed to obtain the stiffness needed to meet the soil failure limit state controlled the design, for which the total T_{al} for the wall section was 10.2 kips/ft, which is just over 75% of the total tensile strength needed by the Simplified Method. Note that in the upper third of the wall, all limit states for both methods will be limited to the minimum strength shown in the plots as a dashed vertical line. If that is considered, the Simplified Method T_{al} required would increase to 13.7 kips/ft, making the Stiffness Method required soil reinforcement strength equal to 74% of what is required by the Simplified Method.
- Overall, the required strength and stiffness of the reinforcement needed is in the low to mid-range of the geogrid product lines available. Note that the ground acceleration used for the seismic design represents what would be included in Seismic Zone 4, which is the highest seismic zone.

Table 15-E-12 Summary of Example 3 wall design calculations using Stiffness Method and R_c = 0.90 (Service, Strength, and Extreme Event I Limit States): (a) Calculation of T_{max}, (b) Service and Strength Limit State calculations, and (c) Extreme Event I (seismic) Limit State calculations.

a)	T_{max} Equation (Eq. 15-E-1) Parameters											Unfactored
Layer Number	**z (ft)**	**S_v (ft)**	**R_cJ_i (kips/ft)**	**S_{global} (ksf)**	**S_{local} (ksf)**	**D_{tmax}**	**F_f**	**Φ_g**	**Φ_{local}**	**Φ_{fb}**	**Φ_{fs}**	**$^+T_{max}$ and T_0 (kips/ft)**
10 (top)	1.33	2.33	7.7	5.89	3.30	0.220	1.61	0.209	0.74	1.0	0.714	0.042
9	3.33	2.00	7.7	5.89	3.85	0.372	1.61	0.209	0.80	1.0	0.714	0.065
8	5.33	2.00	7.7	5.89	3.85	0.523	1.61	0.209	0.80	1.0	0.714	0.092
7	7.33	2.00	7.7	5.89	3.85	0.674	1.61	0.209	0.80	1.0	0.714	0.119
6	9.33	2.00	14.5	5.89	7.25	0.825	1.61	0.209	1.10	1.0	0.714	0.199
5	11.33	2.00	14.5	5.89	7.25	0.976	1.61	0.209	1.10	1.0	0.714	0.236
4	13.33	2.00	14.5	5.89	7.25	1.00	1.61	0.209	1.10	1.0	0.714	0.242
3	15.33	2.00	14.5	5.89	7.25	1.00	1.61	0.209	1.10	1.0	0.714	0.242
2	17.33	2.00	14.5	5.89	7.25	1.00	1.61	0.209	1.10	1.0	0.714	0.242
1	19.33	1.67	14.5	5.89	8.68	1.00	1.61	0.209	1.21	1.0	0.714	0.221
Base of wall	20											$\sum T_{max}$= 1.70

b)			Reinforcement Rupture (Strength Limit)		Connection Rupture (Strength Limit)			Soil Failure (Service Limit)		Pullout (Strength Limit)	
Layer Number	**z (ft)**	**Reinforcement Product**	**#Minimum Required Strength per Unit Width of Reinforcement**		**Connection Capacity as Fraction of T_{lot}, CR_u**	**#Minimum Required Strength per Unit Width of Reinforcement**		**Factored ε_{rein} (%)**	**#T_{al} Corresponding to J_i (kips/ft)**	**Anchorage length L_e (ft)**	
			T_{al} (kips/ft)	**T_{ult} (kips/ft)**		**T_{al} (kips/ft)**	**T_{ult} (kips/ft)**			**Required**	**Min. allowed**
10 (top)	1.33	Geogrid a	0.079	0.17	0.75	0.09	0.20	0.65	0.67	0.72	
9	3.33	Geogrid a	0.12	0.27	0.75	0.15	0.32	1.02	0.67	0.45	
8	5.33	Geogrid a	0.17	0.38	0.75	0.21	0.45	1.43	0.67	0.39	
7	7.33	Geogrid a	0.22	0.49	0.75	0.26	0.58	1.85	0.67	0.37	
6	9.33	Geogrid b	0.37	0.82	0.75	0.44	0.97	1.65 ≤ 2.0% (OK)	1.26	0.49	< 3.0 (OK)
5	11.33	Geogrid b	0.44	0.97	0.75	0.53	1.15	1.95	1.26	0.47	
4	13.33	Geogrid b	0.45	0.99	0.75	0.54	1.18	2.00	1.26	0.41	
3	15.33	Geogrid b	0.45	0.99	0.75	0.54	1.18	2.00	1.26	0.36	
2	17.33	Geogrid b	0.45	0.99	0.75	0.54	1.18	2.00	1.26	0.32	
1	19.33	Geogrid b	0.41	0.90	0.75	0.49	1.08	1.83	1.26	0.26	
Base of wall	20		$\sum T_{al}$= 3.18	$\sum T_{ult}$= 6.96		$\sum T_{al}$= 3.79	$\sum T_{ult}$= 8.28		$\sum T_{al}$= 10.2		

Table 15-E-12, continued

Summary of Example 3 wall design calculations using Stiffness Method and R_c = 0.90 (Service, Strength, and Extreme Event I Limit States): (c) Extreme Event I (seismic) Limit State calculations.

c)			Soil Failure (Service Limit, for Comparison)	Reinforcement Rupture (Extreme Event I Limit)		Connection Rupture (Extreme Event I Limit)			Pullout (Extreme Event I Limit)		
				[#]Minimum Required Strength per Unit Width of Reinforcement			[#]Minimum Required Strength per Unit Width of Reinforcement		Anchorage length L_{eseis} (ft)		Total Reinforcement Length (ft)
Layer Number	z (ft)	Reinforcement Product	[#]Tensile Strength T_{al} Corresponding to Reinforcement Stiffness Required (kips/ft)	T_{al} (kips/ft)	T_{ult} (kips/ft)	Capacity as Fraction of T_{lot}, CR_u	T_{al} (kips/ft)	T_{ult} (kips/ft)	Required	Min. allowed	(min. is 0.7H = 14 ft)
10 (top)	1.33	Geogrid a	0.67	0.36	0.79	0.75	0.43	0.94	5.18	< 3.0 (No, so pullout controls length at wall top)	15.1
9	3.33	Geogrid a	0.67	0.39	0.85	0.75	0.46	1.01	2.17		11.0
8	5.33	Geogrid a	0.67	0.42	0.91	0.75	0.50	1.08	1.43		9.2
7	7.33	Geogrid a	0.67	0.45	0.97	0.75	0.53	1.16	1.09		7.8
6	9.33	Geogrid b	1.25	0.54	1.17	0.75	0.64	1.39	0.99		6.7
5	11.33	Geogrid b	1.25	0.58	1.26	0.75	0.69	1.50	0.86		5.5
4	13.33	Geogrid b	1.25	0.58	1.27	0.75	0.69	1.52	0.74		4.3
3	15.33	Geogrid b	1.25	0.58	1.27	0.75	0.69	1.52	0.64		3.1
2	17.33	Geogrid b	1.25	0.58	1.27	0.75	0.69	1.52	0.57		2.0
1	19.33	Geogrid b	1.25	0.56	1.22	0.75	0.67	1.46	0.49		0.8
Base of wall	20		$\sum T_{al}$= 10.2	$\sum T_{al}$= 5.03	$\sum T_{ult}$= 11.0		$\sum T_{al}$= 5.99	$\sum T_{ult}$= 13.1			

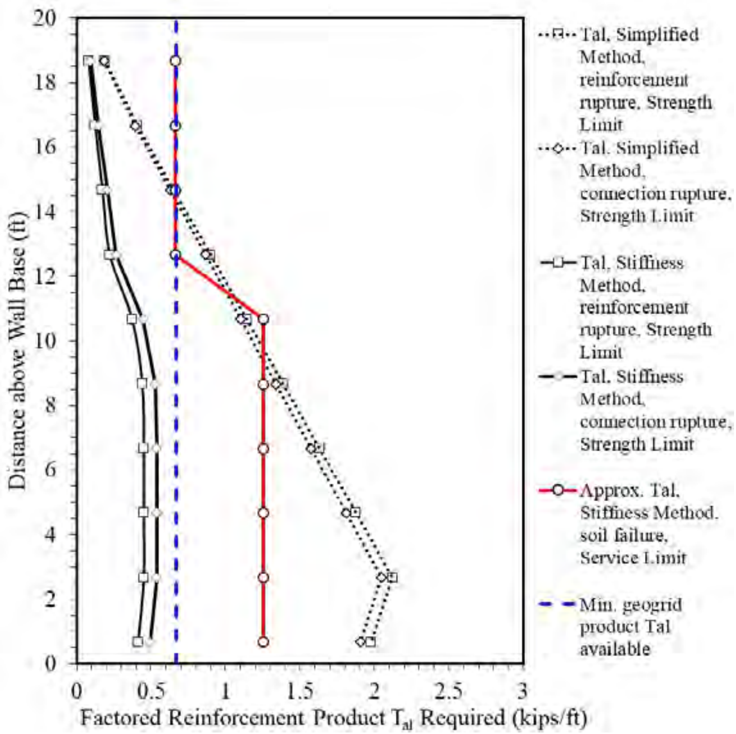
[+]These values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (AASHTO 2020).

[#]For comparison to geogrid product MARV tensile strength that is not reduced by R_c (i.e., load per unit of reinforcement width basis).

*T_{md} for all reinforcement layers is 0.424 kips/ft.

Figure 15-E-10 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Service and Strength limit states, block faced wall with mechanical connection, R_c = 0.90 (Example 3).

Figure 15-E-11 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Service and Extreme Event I (seismic) limit states, block faced wall with mechanical connection, R_c = 0.90 (Example 3).



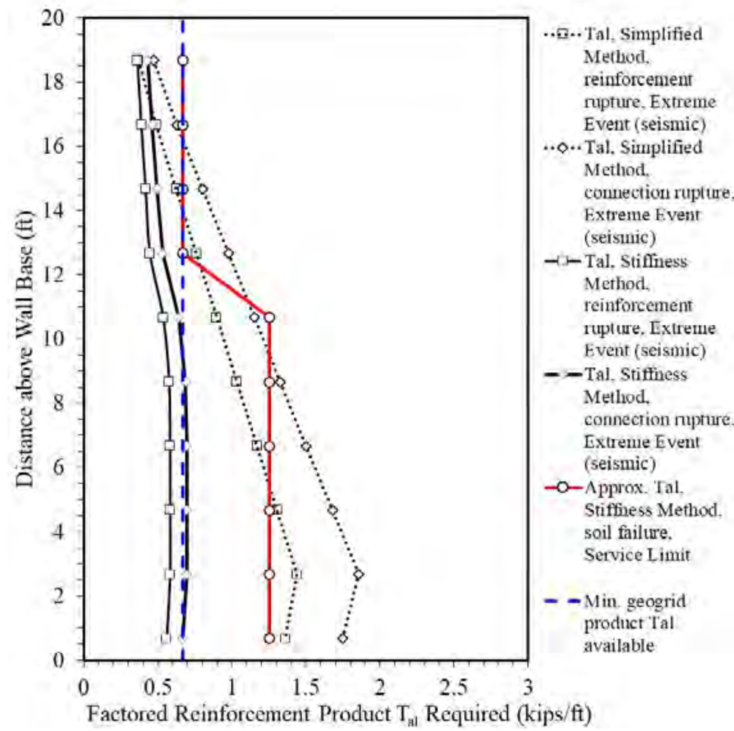
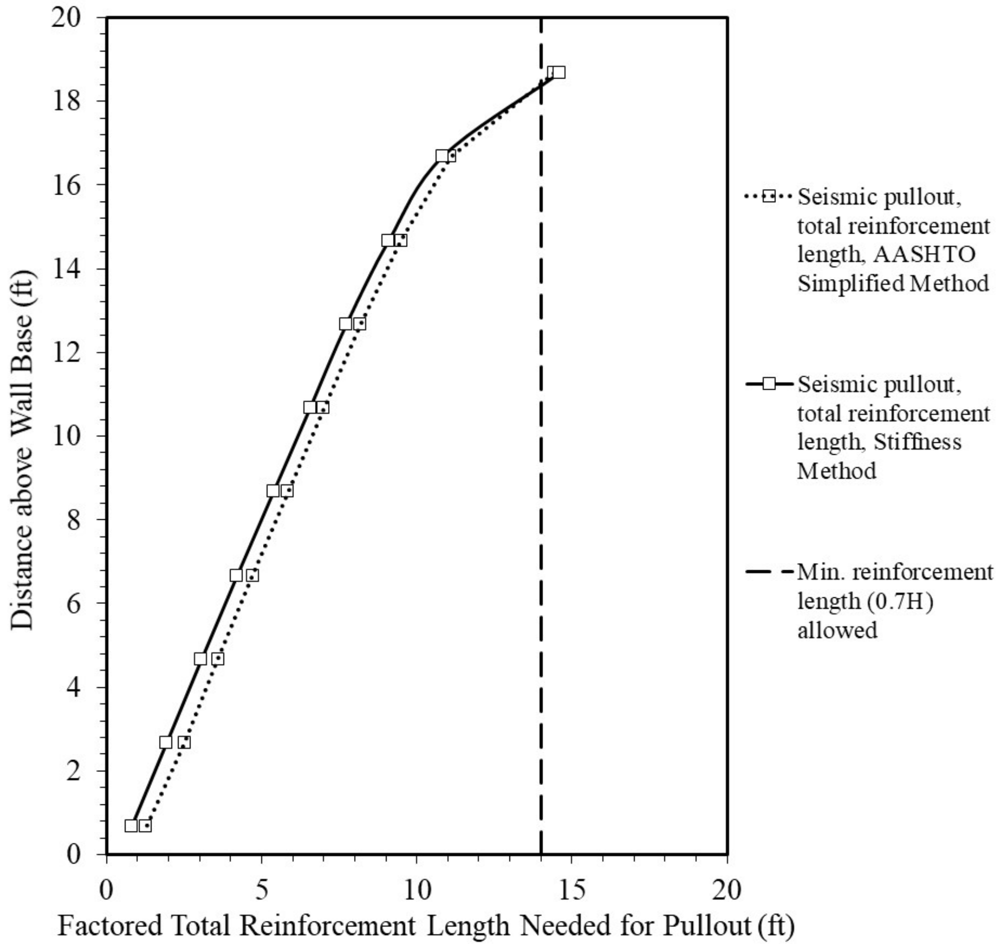


Figure 15-E-12 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Extreme Event I (seismic) limit state, pullout, block faced wall with mechanical connection, R_c = 0.90 (Example 3).



15-E-8 Stiffness Method Design Example 4: Block Faced Geosynthetic Wall System with Frictional Facing Connection

15-E-8.1 General

Figure 15-E-9 shows a cross-section of the wall for this design example. Material properties are provided in Table 15-E-7, except that the minimum geogrid stiffness available on a stiffness per unit of wall width basis is reduced using R_c = 0.90 (i.e., same as Example 3). All other aspects of this example are the same as Example 2. Assume for this example that polyester (PET) geogrid will be used for the soil reinforcement, and dry cast facing blocks with a frictional connection between the geogrid and facing blocks are used. Because of the need to assess connection strength, and because connection strength is geosynthetic and facing block specific (i.e., wall system specific), example system specific ultimate connection strength data ($T_{ultconn}$), and properties for the geosynthetic used with the wall system, are provided in Figure 15-E-13 and Table 15-E-13. The wall backfill is assumed to be a well-graded gravelly sand, with no cohesion.

Figure 15-E-13 Block-geogrid connection test results for Design Example 4, in units of peak tensile capacity (i.e., $T_{ultconn}$) per unit of reinforcement width.

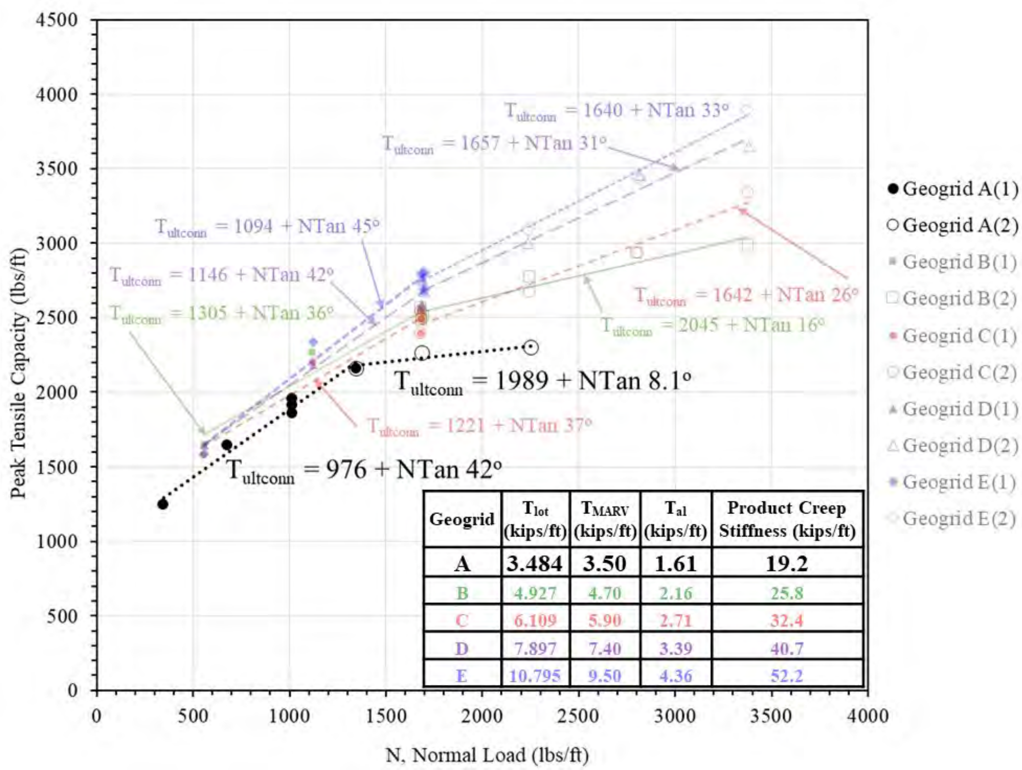


Table 15-E-13 Connection strength equations for example wall system.

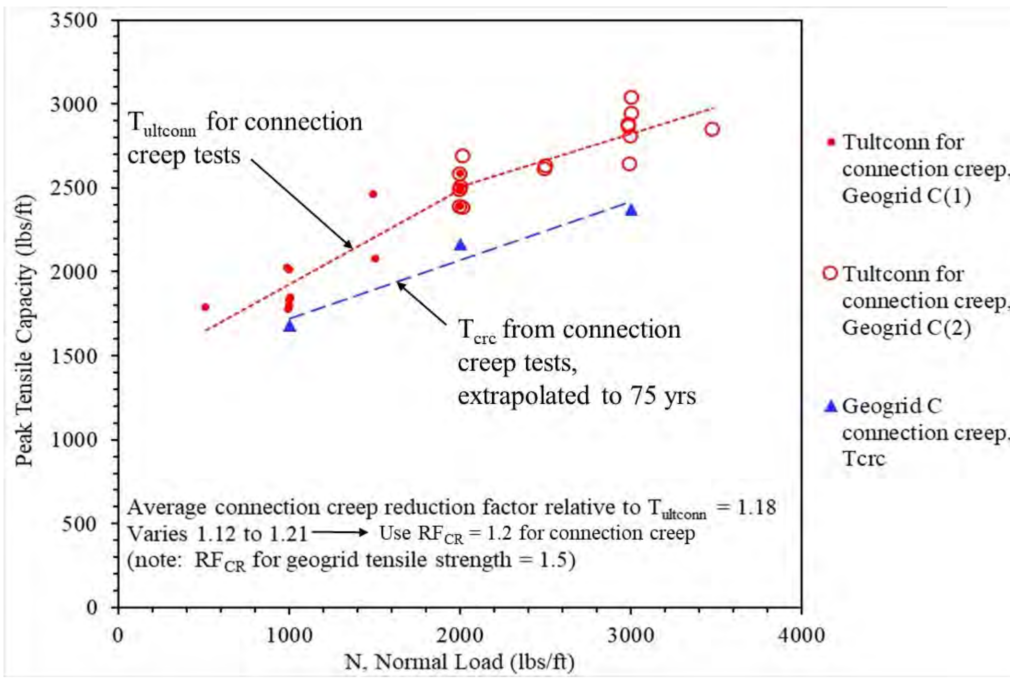
Geogrid Product	Approx. Wall Height, H (ft)	Normal Load, N (lbs/ft)	$T_{ultconn}$ (lbs/ft)	T_{lot} (lbs/ft)
Geogrid A	H < 9	N < 1344	976 + N tan 42°	3484
	H < 20	1344 < N < 2268	1989 + N tan 8.1°	
	H > 20	N > 2268	2416	
Geogrid B	H < 16	N < 1724	1305 + N tan 36°	4927
	H < 30	1724 < N < 3424	2045 + N tan 16°	
	H > 30	N > 3424	3030	
Geogrid C	H < 16	N < 1681	1221 + N tan 37°	6109
	H < 30	1681 < N < 3479	1642 + N tan 26°	
	H > 30	N > 3479	3339	
Geogrid D	H < 16	N < 1695	1146 + N tan 42°	7897
	H < 30	1695 < N < 3380	1657 + N tan 31°	
	H > 30	N > 3380	3688	
Geogrid E	H < 16	N < 1695	1094 + N tan 45°	10795
	H > 16	1695 < N < 3373	1640 + N tan 33°	
	H > 30	3373	3830	

Data for geogrids B through E are faded in the figure and table since, as will be shown later, for the Stiffness Method, only the weakest geogrid will be needed. However, for the comparison to the Simplified Method provided at the end of this example, the other geogrids will be needed. Showing the other geogrids is also useful to demonstrate how the Geogrid A connection strength plot compares with the stronger geogrids. The pattern shown in Figure 15-E-13 is typical of modular block wall system connection strength data in which the connections are mostly frictional. Only when the normal stress between blocks gets high enough do significant connection strength differences between geogrids with a range of tensile strengths occur, transitioning from mostly friction controlled to reinforcement rupture controlled. Because of this, at facing block normal loads that are relatively low, increasing the geogrid tensile strength may not help much, and the only choice may be to reduce the reinforcement spacing. As is shown later, this will not be an issue for the Stiffness Method, but this will be an issue for the Simplified Method. Another approach to address this problem is to conduct 1,000 hour connection creep tests, as for frictional systems, it is likely that a lower reduction factor for creep, RF_{CR}, could be used instead of the RF_{CR} determined for the geogrid in-isolation (in this case, RF_{CR} for the geogrid is 1.5, but for the connection, a RF_{CR} of only 1.2 or lower could be used, as shown in Figure 15-E-14). However, for this example, to keep the example as simple as possible, the RF_{CR} for the geogrid of 1.5 is used (i.e., the data in Figure 15-E-5 is not used in this example, but is for information only).

As is true of examples 2 and 3, the scope of this design example is limited to the service (soil failure limit only), strength, and Extreme Event I (seismic) limit states. Furthermore, for simplicity, live loads are not included in the calculations to follow. It is also assumed that the foundation soil has sufficient shear strength to prevent global instability, bearing capacity failure and excessive settlement. For seismic design, assume that the ground acceleration, A_s, is 0.50g.

The wall geometry is based on Figure 15-E-9 (i.e., same as for Example 3). As is true for examples 2 and 3, Example 4 calculations are demonstrated for one of the middle layers (i.e., Layer 6 in Figure 15-E-10, and for the soil failure and pullout limit states, Layer 10 is also used to illustrate calculations).

Figure 15-E-14 Example block-geogrid creep connection test results for a wall system, in units of peak tensile capacity per unit of reinforcement width.



S = equivalent uniform height of surcharge = 0 ft

K_{avh} and K_{abh} remain unchanged relative to Example 1 and Example 2 at 0.283, and Φ_{fb} = 1.0.

The reinforcement stiffness values used in the calculations to follow, the geogrid stiffness needs to be adjusted to account for the reinforcement coverage ratio, R_c of 0.90.

Since the soil failure limit state usually controls the amount of reinforcement required, this limit state is evaluated first, and T_{max} is calculated, for the wall design.

15-E-8.2 Calculations for Soil Failure Limit State Design (Service I)

As is true for examples 2 and 3, for Example 4 the factored reinforcement peak strain for each layer should be 2.0% or less for a modular block faced wall (i.e., stiff face) in this example. Some trial-and-error is typically required to establish what reinforcement stiffness values are required, starting with the stiffness values required to meet the soil failure limit, as the soil failure limit often controls the design of geosynthetic walls. However, since this is a specific hypothetical wall system, the weakest geogrid available for the wall system should be used as the starting point to check the soil failure limit state. Therefore, begin by calculating T_{max} using the minimum stiffness reinforcement product available for the wall system, which is Geogrid A in Figure 15-E-13. The creep stiffness, J_i, of Geogrid A is 19.2 kips/ft (per unit of reinforcement product width), and its tensile strength T_{MARV} is 3.50 kips/ft and T_{al} is 1.61 kips/ft (also per unit of reinforcement product width).

The contributing factors, coefficients and parameters that comprise the T_{max} equation can be found in Allen and Bathurst (2015) and Table 15-E-1. To calculate T_{max}, the reinforcement stiffness must be per unit of wall width rather than per unit of reinforcement product width; hence, R_cJ_i must be used where the reinforcement stiffness value is required. Therefore, considering a total of 10 layers, the parameters used to determine T_{max} are calculated as follows:

$$S_{\text{global}} = \frac{J_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} R_c J_i}{H} = (10 \times 0.90 \times 19.2 \text{ kips/ft})/20 \text{ ft} = 8.65 \text{ ksf}$$

$$\Phi_g = \alpha \left(\frac{S_{\text{global}}}{p_a}\right)^{\beta} = 0.16 \times (8.65 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.231$$

$$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}}\right)^{d} = (0.283/0.283)^{0.4} = 1.0$$

$$S_{\text{local}} = \left(\frac{R_c J}{S_v}\right)_i = (0.90 \times 19.2 \text{ kips/ft})/(2.0 \text{ ft}) = 8.65 \text{ ksf for Layer 6}$$

$$S_{localave} = \frac{\sum \left({}^{R_cJ}/_{S_v}\right)_i}{n} = \frac{7.42 + 8 \times 8.65 + 10.4}{10} = 8.70\ ksf$$

where, n = 10 is the number of reinforcement layers. Therefore, Φ_{local} is calculated as follows:

$$\Phi_{local} = \left(\frac{S_{local}}{S_{localave}}\right)^{0.5} = \left(\frac{8.70\ ksf}{8.70\ ksf}\right)^{0.5} = 1.00\ \ (Layer\ 6)$$

To determine the facing stiffness factor, the facing stiffness parameter, F_f does not change, and the facing stiffness factor is calculated as:

$$\Phi_{fs} = \eta\left(\left(\frac{S_{global}}{p_a}\right) F_f\right)^{\kappa} = 0.57 \times \left(\left(\frac{8.65\ ksf}{2.11\ ksf}\right) \times 1.61\right)^{0.15} = 0.757$$

Since c = 0, the cohesion factor, Φ_c = 1.0.

D_{tmax} does not change relative to the previous calculation (i.e., D_{tmax} for Layer 6 is 0.825).

T_{max} for Layer 6 is calculated as follows:

$$T_{max} = S_v \left[H\gamma_r D_{tmax} + \left(\frac{H_{ref}}{H}\right) S\gamma_f + LL \right] K_{avh}\Phi_{fb}\Phi_g\Phi_{fs}\Phi_{local}\Phi_c$$

$$T_{max} = 2.0\ ft \left[20ft \times 0.130\ kcf \times 0.825 + \left(\frac{20\ ft}{20\ ft}\right) 0\ ft \times 0.130\ kcf + 0 \right] 0.283 \times 1.0 \times 0.231 \times 0.757 \times 1.10 \times 1.0$$

T_{max} = <u>0.211 kips/ft of wall width</u>

T_{max}, and the calculated parameters needed to calculate T_{max}, are summarized in Table 15-E-14 for the rest of the layers.

Using Equation 15-14 with load factor γ_{sf} = 1.2, and resistance factor ϕ_{sf} = 1.0 (Table 15-5), the factored reinforcement strain corresponding to Layer 10 is computed as:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.061 \frac{kips}{ft}}{1.0 \times 0.90\ \times 19.2 \frac{kips}{ft}} \times 100\% = 0.42\% \leq 2\% \quad OK$$

For Layer 6:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.211 \frac{kips}{ft}}{1.0 \times 0.90 \times 19.2 \frac{kips}{ft}} \times 100\% = 1.47\% \leq 2\% \quad OK$$

See Table 15-E-14 for the calculation results for the rest of the layers for the soil failure limit state for Example 4. Note that the calculated factored strains are significantly less than the target maximum strain of 2.0%. This means that if a weaker geogrid reinforcement was available for this wall system, a weaker product could have been used. Alternatively, the coverage ratio R_c could have been reduced further (i.e., to less than 0.90).

15-E-8.3 Calculations for Reinforcement Rupture Limit State (Strength I)

Since no live load is present, the required long-term reinforcement strength (T_{al}) for Layer 6 is computed using Equation 15-E-5.

The minimum required value of T_{al} and T_{ult} for Layer 6 is therefore:

$$T_{al} \geq \frac{\gamma_{p-EV} T_{max}}{\phi_{rr} R_c} = \frac{1.35 \times 0.211 \frac{kips}{ft}}{0.80 \times 0.90} = 0.398 \frac{kips}{ft}$$

$$T_{ult} = T_{al} RF_{ID} RF_{CR} RF_D = 0.398 \times 1.12 \times 1.5 \times 1.3 = 0.868 \frac{kips}{ft}$$

For Layer 6, the calculated T_{al} for reinforcement rupture, using only a stiffness that is consistent with available wall system specific reinforcement products, is significantly less than T_{al} needed to achieve the stiffness required for the soil failure limit state (i.e., 0.398 kips/ft << 1.26 kips/ft) and significantly less that the T_{al} for the weakest geogrid product available for the wall system (i.e., 0.398 kips/ft << 1.61 kips/ft). Therefore, to meet the Strength Limit State, reinforcement rupture, Geogrid A can be used. See Table 15-E-14 for the calculation results for the rest of the wall layers.

15-E-8.4 Calculations for Connection Strength Design (Strength I)

To determine the minimum tensile strength needed at the connection to the facing, connection strength data for the facing block – geosynthetic combinations anticipated are needed. It has been assumed for this example that a frictional type connection between the facing blocks and geogrid will be used. Short-term connection test results for the geogrids under consideration in this example are shown in Figure 15-E-13 and Table 15-E-13. T_{lot} values for the connection tests are also summarized in the figure.

Equation 15-E-9 is used to calculate the minimum long-term connection strength needed, T_{ac}(required). For the purposes of this example and to be consistent with the approach taken in the current AASHTO (2020) specifications, the load and resistance factors for the connection rupture limit state are assigned the same values as those used for geosynthetic rupture limit state at locations away from the connection (i.e., $\gamma_{p-EVc} = \gamma_{p-EV} = 1.35$ and $\phi_{cr} = \phi_{rr} = 0.80$), and $T_o = T_{max}$.

Therefore, using Equation 15-E-9, at Layer 6, the factored connection load is:

$$T_{ac}(required) = (\gamma_{p-EVc}) T_0 = (1.35) \times 0.211 \frac{kips}{ft} = 0.285 \frac{kips}{ft} \text{ of wall width}$$

To determine the long-term connection strength available, since a dominantly frictional connection is used in this example, the connection strength will be a function of the normal load on the facing blocks, N, which is affected by the depth of the connection below the wall top if the facing is vertical (i.e., no facing batter), but is limited by the hinge height (see AASHTO 2020 LRFD Bridge Design Manual, Art. 11.10.6.4.4b) if the facing is battered. For this example, the facing is assumed to have no batter (i.e., is vertical).

To determine $T_{ultconn}$, the normal force, N, on the facing blocks, in units of load per unit of wall width, must be determined at each layer depth. The following equation can be used for this purpose for a vertical wall:

$$N = \gamma_{block} \times z \times W_u \tag{15-E-34}$$

where,

γ_{block}	=	average unit weight of block plus any soil placed in block hollow areas, if any are present (kcf)
z	=	depth below wall top at face to reinforcement layer (for battered walls, use the hinge height as a limit) (ft)
W_u	=	facing block width (face to back of block) (ft)

Using the relationships presented in Figure 15-E-13 and Table 15-E-13, $T_{ultconn}$ is calculated as follows for Layer 6, using the connection test results for Geogrid A (i.e., this geogrid is the minimum strength geogrid available for the specific wall system that meets or exceeds soil failure limit state requirements):

$$N = 0.12\ kcf \times 9.33ft \times 1.0ft = 1.12 \text{ kips/ft}$$

Therefore,

$$T_{ultconn} = 0.976 + 1.12 \times \text{Tan } 42^{o} = 1.98 \text{ kips/ft}$$

Expressed as a portion of T_{lot}, the short-term connection strength CR_u is (1.98 kips/ft)/(3.484 kips/ft) = 0.568.

Combining equations 15-E-11 and 15-E-12, the available, factored long-term connection strength is calculated as follows:

$$T_{ac}(available) = \frac{\phi_{cr} \times T_{ult} \times \left(\frac{T_{ultconn}}{RF_{CR} \times T_{lot}}\right) R_c}{RF_D} \tag{15-E-35}$$

For Layer 6,

$$T_{ac}(available) = \frac{0.8 \times \frac{3.50kips}{ft} \times \frac{1.98\frac{kips}{ft}}{1.5 \times 3.484\frac{kips}{ft}}(0.9)}{1.3} = 0.734\frac{kips}{ft}$$

Note that T_{ult} in this equation is a Minimum Average Roll Value (MARV), whereas T_{lot} is the tensile strength of the geogrid used for the connection testing.

At Layer 6,

T_{ac} (available) > T_{ac} (required (i.e., 0.734 kips/ft >> 0.285 kips/ft).

Therefore, connection strength does not control the design.

Focusing instead on the minimum geogrid tensile strength needed to safely meet the demand at the connection, determine T_{ult} as follows:

$$T_{ult}(min.required) = \left(\frac{\gamma_{p-EVc}}{\phi_{cr} R_c}\right) T_0 RF_D RF_{CR} \left(\frac{T_{lot}}{T_{ultconn}}\right) \tag{15-E-36}$$

Note that $T_{lot}/T_{ultconn}$, which essentially is $1/CR_u$, will be specific to the geosynthetic reinforcement and block/connector system used, in addition to being a function of the normal force between blocks at the connection, N. This equation can be used to estimate the ultimate geogrid tensile strength, T_{ult}, required at the connections to compare to the T_{ult} required for the other limit states, to help determine which limit state is controlling the design.

With the above in mind, for Layer 6, T_{ult} (min. required, on a strength per unit of reinforcement width basis) for connection strength is as follows, using RF_{CR} determined from the in-isolation geogrid creep rupture data:

$$T_{ult}(min.required) = \left(\frac{1.35}{0.8 \times 0.9}\right) \times 0.211 \times 1.3 \times 1.5 \left(\frac{3.484}{1.98}\right) = 1.36\frac{kips}{ft}, \text{ and}$$

T_{al} = (1.36 kips/ft)/2.18 = 0.623 kips/ft.

On a strength per unit of reinforcement width basis, this minimum required geosynthetic T_{al} of 0.623 kips/ft is below the T_{al} value of the weakest geogrid product available for this wall system (i.e., Geogrid A, in which T_{MARV} = 3.50 kips/ft and a T_{al} = 3.50/2.18 = 1.61 kips/ft). So at this point, the minimum strength product available for this wall system is in fact controlling design.

15-E-8.5 Calculations for Pullout Limit State Design (Strength I)

The default pullout parameters α and F* specified in AASHTO (2020) are used for this example design and are the same as in Example 1 (α = 0.8 and F* = 0.452). The vertical stress, σ_v, over the reinforcement anchorage length, conservatively calculated using Equation 15-E-29, is the same as in Example 1 (0.173 ksf). Since pullout is usually most critical near the wall top, this example pullout calculation is carried out for Layer 10.

Using Equation 15-E-30, the required factored length of reinforcement in the resistant zone (i.e., behind the active wedge) for Layer 10 is:

$$L_e = \frac{\gamma_{p-EV} T_{max}}{\phi_{po} C(\alpha F^*)\sigma_v R_c}$$

T_{max} used here corresponds to the minimum stiffness product available for the wall system (i.e., J_i = 19.2 kips/ft). As before, R_c = 0.90 and all other parameters and their values have been defined earlier. For layer 10, per ft of reinforcement width:

$$L_e = \frac{1.35 \times 0.061\frac{kips}{ft}}{0.70 \times 2(0.8 \times 0.452)(0.173\ ksf)0.90} = 1.04\ ft$$

To determine the total reinforcement length needed, L, the length of reinforcement within the active zone, L_a, must be calculated. For geosynthetic walls, the active zone is assumed to be bounded by the Rankine active zone (wedge). L_a is calculated as follows for a vertical wall (at Layer 10):

L_a = (H – z) tan (45° – ϕ_r/2) = (20 ft – 1.33 ft) tan (45° – 34°/2) = 9.93 ft

The minimum length allowed for L_e is 3 ft (AASHTO 2020), which is greater than the calculated L_e required for pullout for layer 10. Therefore, using L_e = 3 ft, the total reinforcement length required for layer 10 is:

$L = L_a + L_e = 9.93\ ft + 3\ ft = 12.9\ ft$

Strength Limit State calculation results for all the layers (i.e., reinforcement rupture, connection rupture, and pullout) are summarized in Table 15-E-15 (a and b).

It should be recalled that the minimum length of reinforcement for typical reinforced soil walls is 0.7H = 0.7×(20 ft) = 14 ft. Therefore, pullout does not control the reinforcement length required. Note that other limit state design calculations can result in greater reinforcement length, such as external stability (e.g., sliding) or global stability.

15-E-8.6 Calculations for Determination of T_{max} + T_{md} (Extreme Event I - Seismic)

The calculation of T_{max} as described and carried out earlier in this example using the Stiffness Method is also applicable for seismic design for the static portion of the reinforcement load. T_{md}, the incremental dynamic inertia force per reinforcement layer, must be added to T_{max} to determine the total reinforcement load for each layer during seismic loading.

T_{md} is calculated as shown for Example 2, considering the weight of the facing blocks.

$$T_{md} = \left(\frac{P_i}{n}\right) = \frac{4.24\,\frac{kips}{ft}}{10} = 0.424\,\frac{kips}{ft}$$

The total load per reinforcement layer during seismic shaking, T_{totalf}, is then calculated using superposition as follows, for Layer 6):

$$T_{totalf} = \gamma_{seis}(T_{max} + T_{md}) = 1.0\left(0.211\,\frac{kips}{ft} + 0.424\,\frac{kips}{ft}\right) = 0.635\,\frac{kips}{ft}$$

For seismic pullout design, T_{totalf} is used. However, for reinforcement rupture and connection rupture under seismic loading, the static and dynamic components of the reinforcement load must be handled separately. The reason for this is that the strength required to resist T_{max} must include the effects of creep because it is a sustained load, but the strength required to resist T_{md} should not include the effects of creep due to the transient nature of T_{md}.

15-E-8.7 Calculations for Reinforcement Rupture (Extreme Event I - Seismic)

Using equations 15-E-21, 15-E-22, and 15-E-23, T_{ult} for the static portion of load at Layer 6 is:

$$S_{rs} = \frac{\gamma_{seis} T_{max} RF}{\phi R_c} = \frac{1.0 \times 0.211 \frac{kips}{ft} \times 2.18}{1.0 \times 0.90} = 0.511 \frac{kips}{ft}$$

T_{ult} for dynamic portion of load at Layer 6 is:

$$S_{rt} = \frac{\gamma_{seis} T_{md} RF_{ID} RF_D}{\phi R_c} = \frac{1.0 \times 0.424 \frac{kips}{ft} \times 1.12 \times 1.3}{1.0 \times 0.90} = 0.686 \frac{kips}{ft}$$

$T_{ult} = S_{rs} + S_{rt} = 0.511$ kips/ft + 0.687 kips/ft = 1.20 kips/ft

$T_{al} = 1.20$ kips/ft/2.18 = 0.550 kips/ft of reinforcement product width.

15-E-8.8 Calculations for Connection Rupture (Extreme Event I - Seismic)

To determine the T_{ult} needed to prevent connection rupture during seismic loading, need CR_u and CR_{cr}, which are calculated as follows for Layer 6:

$$CR_u = \frac{T_{ultconn}}{T_{lot}} = \frac{1.98 \frac{kips}{ft}}{3.484 \frac{Kips}{ft}} = 0.568$$

$$CR_{cr} = \frac{T_{ultconn}}{RF_{CR} T_{lot}} = \frac{1.98 \frac{kips}{ft}}{1.5 \times 3.484 \frac{Kips}{ft}} = 0.379$$

Because this is a frictional connection, the connection resistance is reduced using a factor, F_r, of 0.8 to account for potential loss of frictional resistance due to the earthquake ground motion.

$$S_{rsc} = \frac{\gamma_{seis} T_0 RF_D}{F_r \phi CR_{cr} R_c} = \frac{1.0 \times 0.211 \frac{kips}{ft} \times 1.3}{0.8 \times 1.0 \times 0.379 \times 0.90} = 1.01 \frac{kips}{ft}$$

$$S_{rtc} = \frac{\gamma_{seis} T_{md} RF_D}{F_r \phi CR_u R_c} = \frac{1.0 \times 0.424 \frac{kips}{ft} \times 1.3}{0.8 \times 1.0 \times (0.568) \times 0.90} = 1.35 \frac{kips}{ft}$$

$T_{ult} = S_{rsc} + S_{rtc} = 1.01$ kips/ft+1.35 kips/ft = 2.36 kips/ft

$T_{al} = 2.36$ kips/ft/2.18 = 1.08 kips/ft of reinforcement product width.

On a load per reinforcement product width basis, this minimum required geosynthetic T_{al} of 1.08 kips/ft is still below the T_{al} value of the minimum geogrid product tensile strength T_{al} available for this wall system (i.e., Geogrid A) of 1.61 kips/ft. Therefore, the minimum strength product available for this wall system is still controlling the design and can be used for all the reinforcement layers.

15-E-8.9 Calculations for Pullout Limit State Design (Extreme Event I - Seismic)

Using equations 15-E-29 and 15-E-30, L_e for Layer 10 (i.e., at the wall top) is determined as follows:

$\sigma_v = z\gamma_r + S_{sur}\gamma_f$

$\sigma_v = (1.33 \text{ ft})(0.13 \text{ kcf}) + (0)(0.13 \text{ kcf}) = 0.173 \text{ ksf}$

$$L_e = \frac{\gamma_{seis}(T_{max} + T_{md})}{\phi C(0.8\alpha F^*)\sigma_v R_c} = \frac{1.0(0.061 + 0.424)}{1.0 \times 2 \times (0.8 \times 0.8 \times 0.452) \times 0.173\ kcf \times 0.90}$$

$L_e = 5.39 \text{ ft}$

$L_a = (H - z) \tan(45° - \phi_r/2) = (20 \text{ ft} - 1.33 \text{ ft}) \tan(45° - 34°/2) = 9.93 \text{ ft}$

$L = L_a + L_e = 9.93 \text{ ft} + 5.39 \text{ ft} = 15.3 \text{ ft}$

Therefore, at the wall top, pullout is controlling the reinforcement length needed.

15-E-8.10 Calculations to Check to Make Sure Stiffness and Strength Required are Properly Matched (Extreme Event I, Seismic)

A substantial increase in tensile strength was required to achieve internal stability for seismic loading, given the high ground acceleration required for this hypothetical site in the Puget Sound region of western Washington. However, the weakest geogrid product available for the example wall system (i.e., Geogrid A) has a T_{al} of 1.61 kips/ft of reinforcement product width, which is significantly greater than the T_{al} needed of 1.08 kips/ft. Therefore, an additional iteration to match the available reinforcement strength and stiffness to the demand is not required.

Typically, this check will show that another iteration to complete the wall design is not required, except possibly for very inefficient facing/reinforcement connections combined with high seismic loading.

15-E-8.11 Summary for Example 4 Design

See Table 15-E-14 for the calculation results for all of the layers for the Service I, Strength I, and Extreme Event I limit states. These calculation results are plotted in figures 15-E-15 through 15-E-17. In Figure 15-E-15, the minimum tensile strength needed to just meet the soil failure limit state is also shown for illustration purposes, which demonstrates that the strength required to just meet the soil failure limit is significantly less than the strength of the minimum tensile strength geogrid (i.e., Geogrid A) available for the wall system.

In summary, for the final internal stability design for Example 4 using the Stiffness Method, the following would be specified with regard to reinforcement properties and spacing for the optimized design, and the following observations can be made:

- The vertical spacing of reinforcements is 2 ft throughout the wall height, and the coverage ratio is a minimum of 0.90.
- The minimum reinforcement length for internal stability (i.e., pullout) is 14 ft for the Strength Limit, and all layers except the top layer for seismic did not exceed this minimum length. The top layer length required is 15.3 ft for seismic for the Stiffness Method, which is greater than the 0.7H minimum. The longer reinforcement length due to pullout is consistent with good detailing for MSE walls in AASHTO LRFD Article 11.10.7.4.
- The lowest strength PET geogrid included with the wall system was greater in strength and stiffness than required by the Stiffness Method design (see figures 15-E-15 and 15-E-16). Comparative calculations were done with the Simplified Method, but those calculations showed that greater tensile strength than the minimum strength product for the wall system and reduced vertical spacing were required to have enough reinforcement for equilibrium (not shown in figures 15-E-15 and 15-E-16, as a layer by layer comparison between methods was not possible due to the increase in the number of layers needed for the Simplified Method).
- Using Geogrid A as the minimum strength geogrid available, the Simplified Method required a total long-term tensile strength T_{al} of 30.1 kips/ft for the entire wall section (distributed among 15 reinforcement layers), whereas the Stiffness Method would allow Geogrid A to be used for all layers, but distributed among only 10 reinforcement layers, for a total of 16.1 kips/ft for the entire wall section. Therefore, the Stiffness Method would require only 53% of the total reinforcement strength required by the Simplified Method. The Stiffness method could have allowed even less total reinforcement strength to be used if the coverage ratio R_c was reduced to less than 0.90, or if a weaker geogrid was available for this system.
- Overall, the required strength and stiffness of the reinforcement needed is in the low to mid-range of the geogrid product lines available. Note that the ground acceleration used for the seismic design represents what would be included in Seismic Zone 4, which is the highest seismic zone.

Table 15-E-14 Summary of Example 4 wall design calculations using Stiffness Method, R_c = 0.90, and frictional wall face connection (Service, Strength, and Extreme Event I Limit States): (a) Calculation of T_{max} using minimum strength product available for wall system, (b) Service and Strength Limit State calculations, and (c) Extreme Event I (seismic) Limit State calculations.

a)	T_{max} Equation (Eq. 15-E-1) Parameters											+Unfactored maximum reinforcement load
Layer Number	**z (ft)**	**S_v (ft)**	**$^*R_cJ_i$ (kips/ft)**	**S_{global} (ksf)**	**S_{local} (ksf)**	**D_{tmax}**	**F_f**	**Φ_g**	**Φ_{local}**	**Φ_{fb}**	**Φ_{fs}**	**$^xT_{max}$ and T_0 (kips/ft)**
10 (top)	1.33	2.33	17.3	8.65	7.42	0.220	1.61	0.231	0.92	1.0	0.757	0.061
9	3.33	2.00	17.3	8.65	8.65	0.372	1.61	0.231	1.00	1.0	0.757	0.095
8	5.33	2.00	17.3	8.65	8.65	0.523	1.61	0.231	1.00	1.0	0.757	0.134
7	7.33	2.00	17.3	8.65	8.65	0.674	1.61	0.231	1.00	1.0	0.757	0.173
6	9.33	2.00	17.3	8.65	8.65	0.825	1.61	0.231	1.00	1.0	0.757	0.211
5	11.33	2.00	17.3	8.65	8.65	0.976	1.61	0.231	1.00	1.0	0.757	0.250
4	13.33	2.00	17.3	8.65	8.65	1.00	1.61	0.231	1.00	1.0	0.757	0.256
3	15.33	2.00	17.3	8.65	8.65	1.00	1.61	0.231	1.00	1.0	0.757	0.256
2	17.33	2.00	17.3	8.65	8.65	1.00	1.61	0.231	1.00	1.0	0.757	0.256
1	19.33	1.67	17.3	8.65	10.4	1.00	1.61	0.231	1.09	1.0	0.757	0.234
Base of wall	20											$\sum T_{max}$= 1.93

b)		Reinforcement Rupture (Strength Limit)				Connection Rupture (Strength Limit)			Soil Failure (ServiceLimit)	Pullout (Strength Limit)	
				#Minimum Required Strength per Unit Width of Reinforcement			#Minimum required strength per Unit Width of Reinforcement			Anchorage length L_e (ft)	
Layer Number	**z (ft)**	**#Reinforce-ment Product Available for Wall System**	**#Minimum Available Product Tensile Strength, T_{al} (kips/ft)**	**T_{al} (kips/ft)**	**$T_{ult} = T_{al} \times RF$ (kips/ft)**	**Connection Capacity $T_{ultconn}$ (kips/ft)/% of T_{ult}**	**T_{al} (kips/ft)**	**$T_{ult} = T_{al} \times RF$ (kips/ft)**	**Factored ε_{rein} (%)**	**Required**	**Minimum allowed**
Column→	1			3	4	7	8	9	10	11	12
10 (top)	1.33	Geogrid A	1.61	0.11	0.25	1.09/0.31	0.32	0.69	0.42	1.04	
9	3.33	Geogrid A	1.61	0.18	0.39	1.32/0.38	0.42	0.91	0.66	0.65	
8	5.33	Geogrid A	1.61	0.25	0.55	1.55/0.45	0.50	1.10	0.93	0.57	
7	7.33	Geogrid A	1.61	0.32	0.71	1.79/0.51	0.57	1.24	1.20	0.54	
6	9.33	Geogrid A	1.61	0.40	0.87	1.95/0.56	0.62	1.36	1.47 ≤ 2.0%	0.52	< 3.0
5	11.33	Geogrid A	1.61	0.47	1.02	2.05/0.59	0.67	1.46	1.73 (OK)	0.50	(OK)
4	13.33	Geogrid A	1.61	0.48	1.05	2.15/0.62	0.67	1.47	1.78	0.44	
3	15.33	Geogrid A	1.61	0.48	1.05	2.24/0.64	0.66	1.45	1.78	0.38	
2	17.33	Geogrid A	1.61	0.48	1.05	2.34/0.67	0.65	1.43	1.78	0.34	
1	19.33	Geogrid A	1.61	0.44	0.96	2.42/0.69	0.59	1.29	1.62	0.28	
Base of wall	20		$\sum T_{al}$= 16.1	$\sum T_{al}$= 3.61	$\sum T_{ult}$= 7.89		$\sum T_{al}$= 5.67	$\sum T_{ult}$= 12.4			

Table 15-E-14, continued.

Summary of Example 4 wall design calculations using Stiffness Method, R_c = 0.90, and frictional wall face connection (Service, Strength, and Extreme Event I Limit States): (c) Extreme Event I (seismic) Limit State calculations.

c)		Reinforcement Rupture (Extreme Event I Limit)				Connection Rupture (Extreme Event I Limit)			Pullout (Extreme Event I Limit)		
Layer Number	z (ft)	#Reinforce-ment Product Available for Wall System	#Minimum Available Product Tensile Strength, T_{al} (kips/ft)	#Minimum Required Strength per Unit Width of Reinforcement		Connection capacity $T_{ultconn}$ (kips/ft)/% of T_{ult}	#Minimum Required Strength per Unit Width of Reinforcement		Anchorage length L_{eseis} (ft)		Total Reinforcement Length (ft)
				T_{al} (kips/ft)	$T_{ult} = T_{al} \times RF$ (kips/ft)		T_{al} (kips/ft)	$T_{ult} = T_{al} \times RF$ (kips/ft)	Required	Minimum allowed	(minimum is 0.7H = 14 ft)
Column→	1			3	4	7	8	9		10	12
10 (top)	1.33	Geogrid A	1.61	0.38	0.83	1.09/0.31	1.33	2.90	5.39	< 3.0 (No, so pullout controls length, but only at wall top))	15.3
9	3.33	Geogrid A	1.61	0.42	0.92	1.32/0.38	1.22	2.67	2.31		11.2
8	5.33	Geogrid A	1.61	0.46	1.01	1.55/0.45	1.16	2.53	1.55		9.3
7	7.33	Geogrid A	1.61	0.51	1.11	1.79/0.51	1.11	2.43	1.20		7.9
6	9.33	Geogrid A	1.61	0.55	1.20	1.95/0.56	1.08	2.35	1.01		6.7
5	11.33	Geogrid A	1.61	0.59	1.29	2.05/0.59	1.05	2.30	0.88		5.5
4	13.33	Geogrid A	1.61	0.60	1.31	2.15/0.62	1.05	2.29	0.75		4.3
3	15.33	Geogrid A	1.61	0.60	1.31	2.24/0.64	1.03	2.26	0.66		3.1
2	17.33	Geogrid A	1.61	0.60	1.31	2.34/0.67	1.02	2.23	0.58		2.0
1	19.33	Geogrid A	1.61	0.57	1.25	2.42/0.69	0.96	2.10	0.50		0.9
Base of wall	20		$\sum T_{al}$= 16.1	$\sum T_{al}$= 5.28	$\sum T_{ult}$= 11.5		$\sum T_{al}$= 11.0	$\sum T_{ult}$= 24.1			

*Minimum geosynthetic stiffness available for wall system (i.e., Geogrid A).

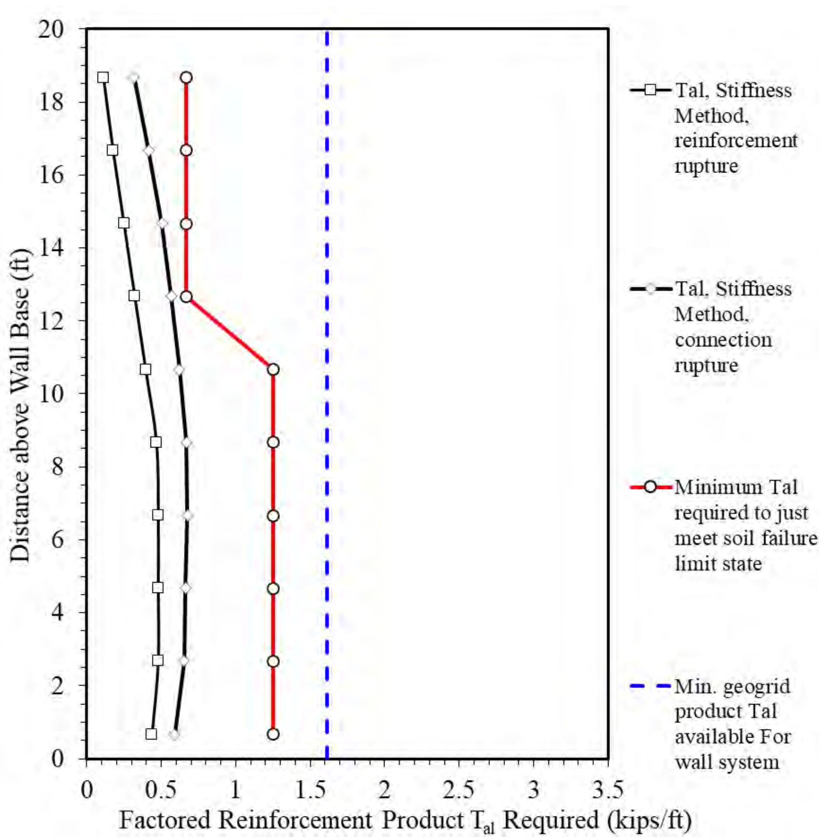
+These values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (AASHTO 2020).

#For comparison to geogrid product tensile strength that is not reduced by Rc (i.e., strength per unit of reinforcement width basis).

xTmd for all reinforcement layers is 0.424 kips/ft.

Figure 15-E-15 Stiffness Method internal stability design for Service and Strength limit states (Example 4).

Figure 15-E-16 Stiffness Method internal stability design to the Simplified Method design, for Extreme Event I (seismic) limit state (Example 4).



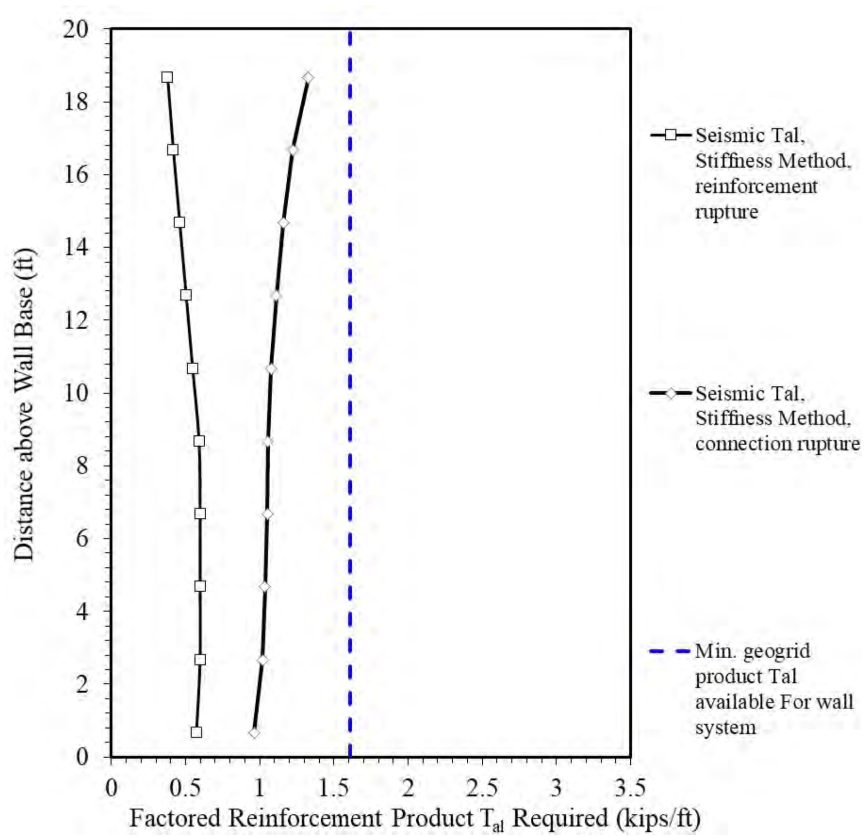
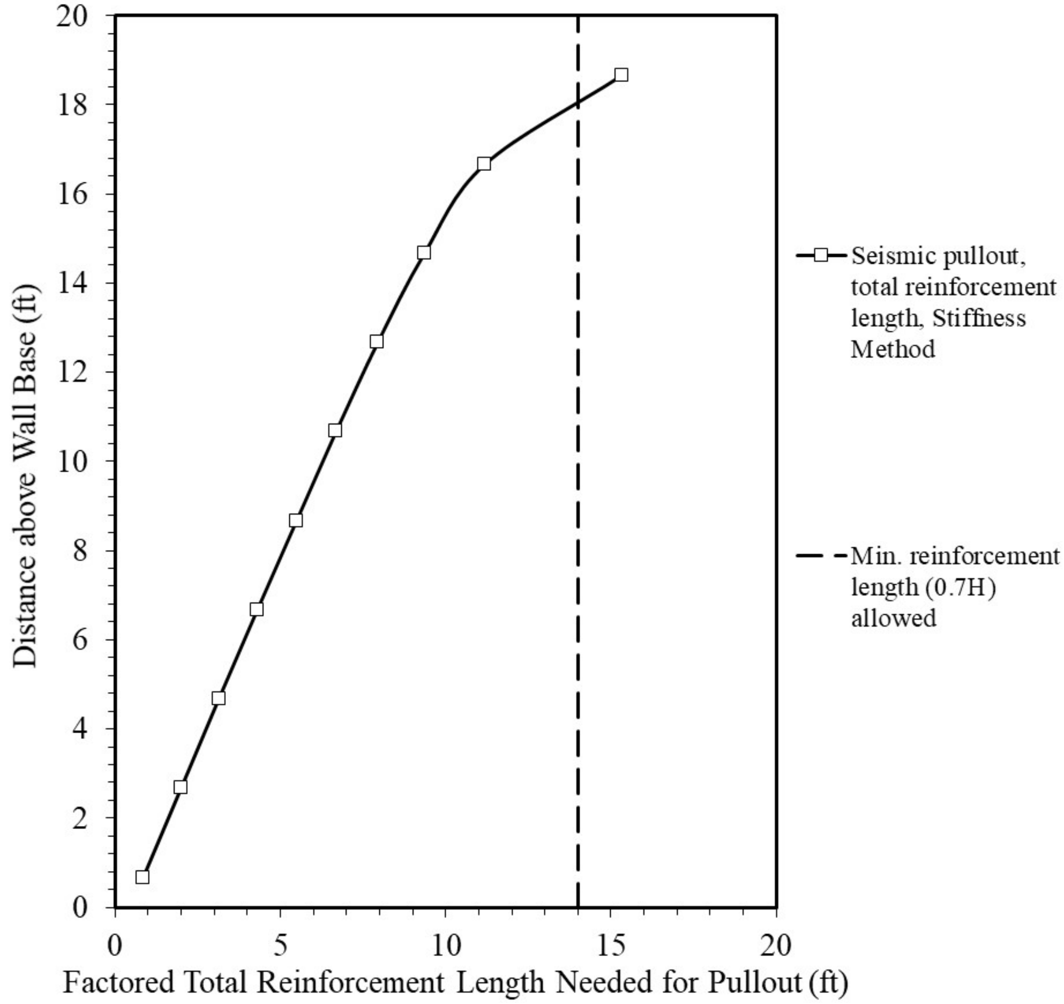


Figure 15-E-17 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Extreme Event I (seismic) limit state, pullout (Example 4).



15-E-9 Stiffness Method Design Example 5: Wrapped (Flexible) Faced Geosynthetic Wall Using Backfill with Small Cohesion

15-E-9.1 General

This example is an extension of Example 1 to consider the effect of wall backfill soil cohesion on wall behavior during and after wall construction. Therefore, this is not a completely developed example. The only purpose of this example is to demonstrate the types of design problems, and possibly long-term wall performance problems, that may occur if cohesion is present, especially if the design is conducted taking into account the “beneficial” effect that cohesion can have in reducing the reinforcement load, T_{max}. The key issue here is the reliability of the cohesion long-term (e.g., will changes in moisture content, softening of the clayey backfill, or soil creep over time occur, reducing the cohesion and allowing the reinforcement layer T_{max} values to increase over time?). In addition to this, as the fines content and plasticity of the backfill increase, the more likely is the buildup of water in the backfill to occur, causing large increases in reinforcement load and wall face deformations (Allen and Bathurst 2009).

Material properties are the same as in Example 1, which are provided in Table 15-E-3, with the exception that the backfill is assumed to have some clayey fines, resulting in a relatively small soil cohesion of 0.20 ksf in addition to the friction angle of 34°.

Table 15-E-2 provides requirements for how to address soil cohesion in the wall backfill. In general, backfill materials that have some cohesion should be avoided, especially in western Washington where rainfall is relatively plentiful. However, in the unusual case in which MSE wall backfill with a limited amount of cohesion cannot be avoided, the effect of that soil cohesion on wall strains and deformations can be assessed using the Stiffness Method.

In this example, the effect of this cohesion on T_{max} at end of construction (i.e., EOC) for the wall, and potential loss of that cohesion over time, is investigated. The results generated in this example will be compared to the Example 1 design, which in effect is the design that would be done if some cohesion is present, but the cohesion is ignored (i.e., using ϕ = 34° and c = 0). The reinforcement stiffness for this example is assumed to be the same as is used in Example 1 (i.e., see Table 15-E-3).

For this example design, determination of the minimum reinforcement stiffness required to keep the peak strain level in the reinforcement layers at 2.5% or less required trial-and-error to optimize the reinforcement design. As in the previous example, Layer 6 will be the primary focus to illustrate the method, except for pullout, in which the uppermost layer (Layer 10) is the focus to illustrate the method. Note that the coverage ratio, R_c, is equal to 1.0 for this example.

$$S_{\text{global}} = \frac{J_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} R_{ci} J_i}{H} = (4 \times 1.0 \times 8.6 \text{ kips/ft} + 6 \times 1.0 \times 17.0 \text{ kips/ft})/20 \text{ ft} = 6.82 \text{ ksf}$$

$$\Phi_g = \alpha \left(\frac{S_{\text{global}}}{p_a}\right)^{\beta} = 0.16 \times (6.82 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.217$$

$$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}}\right)^{d} = (0.283/0.283)^{0.4} = 1.0$$

$$S_{\text{local}} = \left(\frac{R_c J}{S_v}\right)_i = (1.0 \times 17.0 \text{ kips/ft})/(2.0 \text{ ft}) = 8.50 \text{ ksf for Layer 6}$$

$$S_{localave} = \frac{\sum \left({R_c J}/{S_v}\right)_i}{n} = \frac{3.65 + 3 \times 4.25 + 5 \times 8.50 + 10.2}{10} = 6.91\ ksf$$

where, n = 10 is the number of reinforcement layers. Therefore, Φ_{local} is calculated as follows:

$$\Phi_{local} = \left(\frac{S_{local}}{S_{localave}}\right)^{0.5} = \left(\frac{8.5\ ksf}{6.91\ ksf}\right)^{0.5} = 1.11\ \ (layer\ 6)$$

For a flexible wall face, the facing stiffness factor is assumed to be 1.0.

Given c = 0.20 ksf, the cohesion factor, Φ_c is calculated as follows:

$$\Phi_c = e^{-16(c/(\gamma_r H))} = e^{-16(0.20\ ksf/(0.13\ kcf \times 20\ ft))} = 0.292$$

D_{tmax} is determined for Layer 6 as follows:

$z_b = C_h \times (H)^y \times \Phi_{fb} = (0.32 \times (20 \text{ ft})^{1.2}) \times 1.0 = 11.65$ ft

For $z \le z_b$: $D_{tmax} = D_{tmax0} + (z/z_b) \times (1 - D_{tmax0}) = 0.12 + (9.33 \text{ ft}/11.65 \text{ ft}) \times (1 - 0.12) = 0.825$

For bottom layers where $z > z_b$: $D_{tmax} = 1.0$

T_{max} for Layer 6 is calculated as follows:

$$T_{max} = S_v \left[H\gamma_r D_{tmax} + \left(\frac{H_{ref}}{H}\right) S\gamma_f + LL \right] K_{avh} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c$$

$$T_{max} = 2.0\ ft \left[20ft \times 0.130\ kcf \times 0.825 + \left(\frac{20\ ft}{20\ ft}\right) 0\ ft \times 0.130\ kcf + 0 \right] 0.283 \times 1.0 \times 0.217 \times 1.0 \times 1.11 \times 0.292$$

T_{max} = <u>0.0853 kips/ft</u>

T_{max}, and the calculated parameters needed to calculate T_{max}, are summarized in Table 15-E-15 for the rest of the layers.

Note that T_{max} for this layer not considering cohesion is 0.292 kips/ft.

Using Equation 15-14 with load factor γ_{sf} = 1.2, and resistance factor ϕ_{sf} = 1.0 (Table 15-5), the factored reinforcement strain corresponding to Layer 10 is computed as:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.017 \frac{kips}{ft}}{1.0 \times 1.0 \times 8.6 \frac{kips}{ft}} \times 100\% = 0.25\% \leq 2.5\% \quad OK$$

Assuming no cohesion (i.e., as shown in Example 1), for Layer 10,

$$\varepsilon_{rein} = 0.83\%\ not\ considering\ cohesion \geq 0.25\%\ considering\ cohesion$$

For Layer 6:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.0853 \frac{kips}{ft}}{1.0 \times 1.0 \times 17.0 \frac{kips}{ft}} \times 100\% = 0.60\% \leq 2.5\% \quad OK$$

Assuming no cohesion (i.e., as shown in Example 2), for Layer 6,

$$\varepsilon_{rein} = 2.06\%\ not\ considering\ cohesion \geq 0.60\%\ considering\ cohesion$$

See Table 15-E-15 for the calculation results for the rest of the layers.

Based on these calculations, the reinforcement strains increase by a factor of approximately 3.5 post-construction (i.e., 0.83%/0.25%, and 2.06%/0.60%), as when the cohesion is ignored during design, the cohesion will still be present during construction and reduce the reinforcement loads and strains accordingly, as illustrated here. However, the final reinforcement strains and loads long-term will still be as designed with the cohesion ignored and will meet standards. The key is the effect that cohesion loss over time, if it occurs, will have on post-construction wall face deformation, and whether or not the wall, and whatever it supports, can successfully handle that post-construction deformation. Based on experience, a reinforcement strain increase of approximately 1 to 1.5% could result in a face deformation increase of approximately 1 inch for a 20 ft high wall.

Figure 15-E-18 provides plots that compare the strains that result for various assumptions regarding the short-term and long-term presence of cohesion. If the wall is designed considering this limited amount of cohesion (i.e., 0.20 ksf) and the absolute minimum creep stiffness needed to meet the design criteria, reinforcement strains quickly become excessive if that cohesion is lost over time post-construction (i.e., as high as 8.6% as shown in Table 15-E-15 and over 6% strain post-construction), and does not consider the effect of water build-up in the wall backfill due to reduced drainage characteristics (see Allen and Bathurst 2009 for an assessment of wall backfill water build-up on the probability of failure).

It is for this reason that completing the wall design taking into account the soil cohesion, which will result in a reduced reinforcement strength and stiffness, if backfill that has a small to moderate amount of cohesion is the only backfill available, **shall not** be done (i.e., for final wall design, always assume that Φ_c = 1.0 whether or not some limited cohesion in the wall backfill is present). However, if a limited amount of cohesion is present in the backfill, the Stiffness Method may be used to assess how much post-construction strain in the reinforcement may occur if that cohesion disappears over time.

Table 15-E-15 Summary of Example 5 wall design calculations using Simplified Stiffness Method (Service and Strength Limit States).

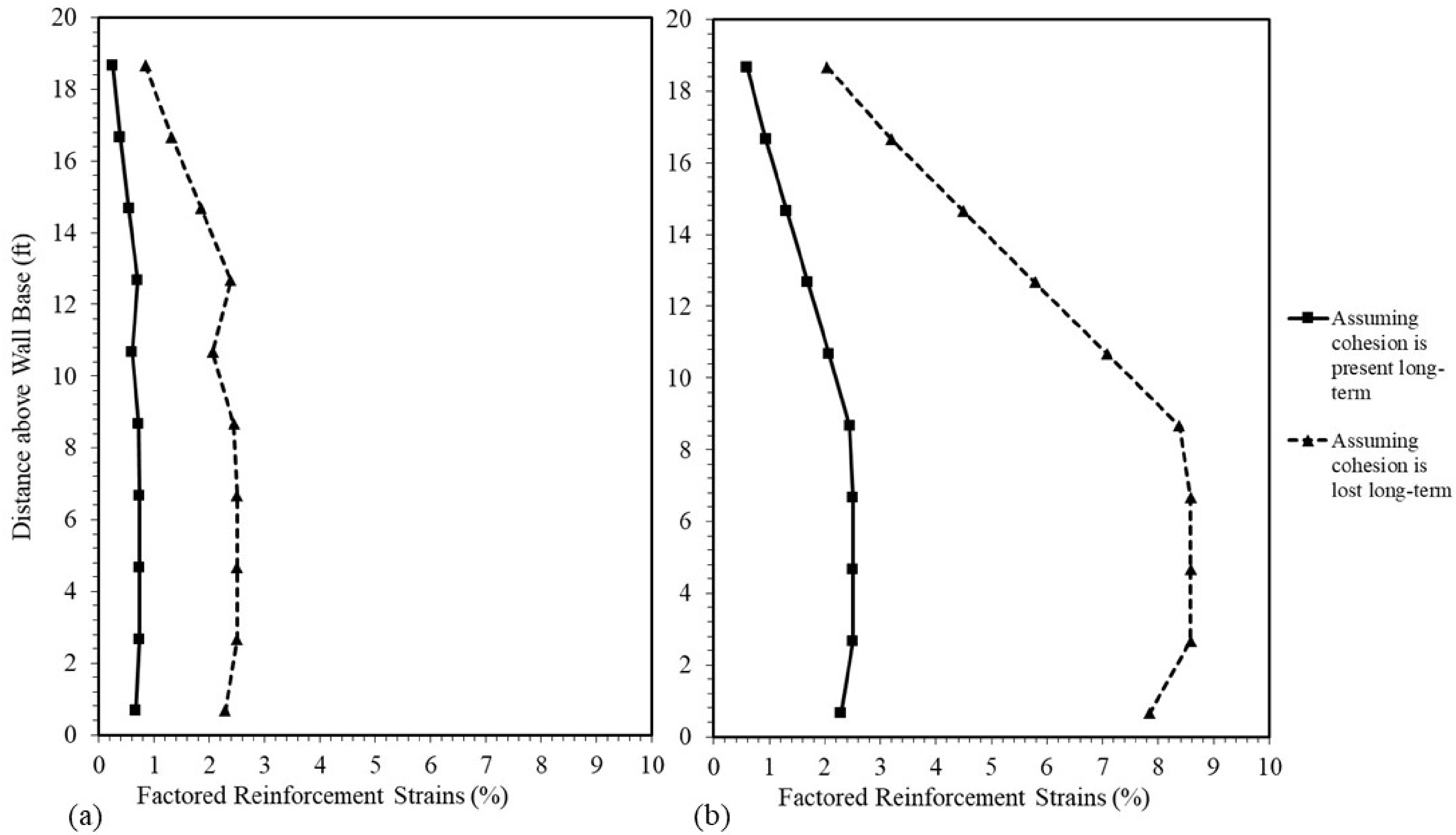
a)	T_{max} Equation (Eq. 15-E-1) Parameters (including cohesion)													Unfactored maximum reinforcement load
Layer Number	**z (ft)**	**S_v (ft)**	**J_i (kips/ft)**	**S_{global} (ksf)**	**S_{local} (ksf)**	**D_{tmax}**	**F_f**	**Φ_g**	**Φ_{local}**	**Φ_{fb}**	**Φ_{fs}**	**Φ_c**		**T_{max} (kips/ft)**
10 (top)	1.33	2.33	8.5	6.82	3.65	0.220	N/A	0.217	0.73	1.0	1.0	0.292		0.017
9	3.33	2.00	8.5	6.82	4.25	0.372	N/A	0.217	0.78	1.0	1.0	0.292		0.027
8	5.33	2.00	8.5	6.82	4.25	0.523	N/A	0.217	0.78	1.0	1.0	0.292		0.038
7	7.33	2.00	8.5	6.82	4.25	0.674	N/A	0.217	0.78	1.0	1.0	0.292		0.049
6	9.33	2.00	17.0	6.82	8.50	0.825	N/A	0.217	1.11	1.0	1.0	0.292		0.085
5	11.33	2.00	17.0	6.82	8.50	0.976	N/A	0.217	1.11	1.0	1.0	0.292		0.101
4	13.33	2.00	17.0	6.82	8.50	1.00	N/A	0.217	1.11	1.0	1.0	0.292		0.103
3	15.33	2.00	17.0	6.82	8.50	1.00	N/A	0.217	1.11	1.0	1.0	0.292		0.103
2	17.33	2.00	17.0	6.82	8.50	1.00	N/A	0.217	1.11	1.0	1.0	0.292		0.103
1	19.33	1.67	17.0	6.82	10.2	1.00	N/A	0.217	1.21	1.0	1.0	0.292		0.094
Base of wall	20													$\overline{T}_{max} = 0.72$

b)		Unfactored T_{max} (kips/ft) (Eq. 15-E-1)		Factored ε_{rein} (%)		Unfactored T_{max} (kips/ft) (Eq. 15-E-1)		Factored ε_{rein} (%)	
		Wall Designed for $\phi = 34^o$		**Wall Designed for $\phi = 34^o$**		**Wall Designed for $\phi = 34^o$ and c = 0.2 ksf***		**Wall Designed for $\phi = 34^o$ and c = 0.2 ksf***	
Layer Number	**z (ft)**	**Not Accounting for Cohesion (as Designed in Example 1)**	**Accounting for Cohesion, c = 0.2 ksf**	**Not Accounting for Cohesion (as Designed in Example 1)**	**Accounting for Cohesion, c = 0.2 ksf**	***Accounting for Cohesion in Final Design**	***+Cohesion Lost**	***Accounting for Cohesion in Final Design**	***+Cohesion Lost**
Column→	1	2	3	4	5	6	7	8	9
10 (top)	1.33	0.060	0.017	0.83	0.25	0.015	0.051	0.60	2.04
9	3.33	0.093	0.027	1.30	0.38	0.023	0.080	0.93	3.19
8	5.33	0.131	0.038	1.85	0.54	0.033	0.112	1.31	4.49
7	7.33	0.168	0.049	2.38	0.69	0.042	0.145	1.69	5.78
6	9.33	0.292	0.085	2.06	0.60	0.052	0.177	2.07	7.08
5	11.33	0.345	0.101	2.44	0.71	0.061	0.209	2.45	8.38
4	13.33	0.354	0.103	2.50	0.73	0.063	0.215	2.51	8.59
3	15.33	0.354	0.103	2.50	0.73	0.063	0.215	2.51	8.59
2	17.33	0.354	0.103	2.50	0.73	0.063	0.215	2.51	8.59
1	19.33	0.323	0.094	2.28	0.67	0.057	0.196	2.29	7.85
Base of wall	20	$\overline{T}_{max} = 2.47$	$\overline{T}_{max} = 0.72$	$\varepsilon_{ave} = 2.06$	$\varepsilon_{ave} = 0.60$	$\overline{T}_{max} = 0.47$	$\overline{T}_{max} = 1.61$	$\varepsilon_{ave} = 1.89$	$\varepsilon_{ave} = 6.46$

*Using minimum creep stiffness of 3.0 kips/ft for all layers, which is well below the minimum stiffness available of 8.2 kips/ft. However, this is for illustration purposes only, and not recommended for design.

+Cohesion is lost over time due to softening of the backfill due to moisture increase, or, in the case of apparent cohesion, due to backfill moisture content changes, over time.

Figure 15-E-18 Comparison of Stiffness Method predicted factored reinforcement strains for wrapped face (flexible) wall with some soil cohesion (Example 5): (a) designed assuming cohesion is not present (i.e., ϕ = 34° and c = o; same as Example 1), (b) designed assuming cohesion is present (i.e., ϕ = 34° and c = 0.2 ksf).



15-E-10 Summary of Lessons Learned from Design Examples

The provided design examples illustrate the use of the Stiffness Method for several geosynthetic wall design scenarios. These scenarios include flexible and stiff facings, and for the stiff faced walls, mechanical (i.e., structural) and friction dominated facing/reinforcement connections, coverage ratios ranging from 0.9 to 1.0, and cohesionless and cohesive soil backfills. Designs are carried out using both the Stiffness Method and the Simplified Method. Designs were carried out for internal stability (soil failure, reinforcement and connection rupture, and pullout) considering Service, Strength, and Extreme Event I (seismic) limit states.

Lessons learned from these examples are as follows:

- In all cases, for the Stiffness Method designs, the Soil Failure Limit controlled the amount and strength of reinforcements needed. However, for Example 4 (i.e., the block faced wall with frictional facing reinforcement connections), since it represented a hypothetical proprietary wall system, the minimum strength geogrid available for the system was stronger than the strength required to meet the soil failure limit state.
- For Example 1 (i.e., the flexible faced wall), the difference between the Stiffness and Simplified method designs was the least of all the examples (i.e., total T_{al} for wall section of 10.6 kips/ft for the Stiffness Method and 11.9 kips/ft for the Simplified Method). The Stiffness Method, however, required less reinforcement in the lower half of the wall and more reinforcement in the upper half of the wall relative to the Simplified Method distribution of reinforcement strength. Example 4 (block faced wall with primarily frictional facing/reinforcement connections) had the largest difference in the Stiffness and Simplified method designs regarding the total reinforcement strength T_{al} needed (i.e., total T_{al} needed for wall section of 16.1 kips/ft for the Stiffness Method and 30.1 kips/ft for the Simplified Method).
- The main reason for the larger difference in total T_{al} needed between the methods for Example 4 was due to the connection strength design for the Simplified Method, especially for seismic loading. This was mainly due to the fact that the Stiffness Method predicts a significantly lower reinforcement load (i.e., T_{max} and T_0) than does the Simplified Method due to the greater prediction accuracy of the Stiffness Method.
- Comparison of examples 1 and 2 can be used to assess the effect of facing stiffness on the magnitude of the total T_{al} needed for the wall design using the Stiffness Method. For the flexible faced wall (Example 1), the total T_{al} needed was 10.6 kips/ft, whereas for the comparable stiff (i.e., block) faced wall (Example 2), the total T_{al} needed was 9.4 kips/ft. The main reason for the difference was the reduction in T_{max} resulting from the facing stiffness and its effect on the strength required to meet the soil failure limit state requirements. Had the connection strength controlled the Stiffness Method design, the difference between the flexible and stiff faced wall examples would have varied depending on the efficiency of the connection.

- Example 5 was used to demonstrate the effect soil cohesion can have on the predicted reinforcement loads and strains when using the Stiffness Method. Even a small amount of cohesion (i.e., 0.2 ksf) can have a big effect on the predicted reinforcement load, and the amount of soil reinforcement needed. In general, backfill soil with clayey fines should be avoided. Based on this example, provided the backfill cohesion is small, ignoring the contribution of the soil cohesion to the soil shear strength used for design will result in a wall design with minimal risk of poor performance provided the fines content is not too high and good drainage is provided. However, it must be recognized that the reinforcement loads at end of wall construction will be reduced due to the cohesion whether or not the cohesion is ignored for the wall design. In this case, the effect of potential loss of soil cohesion due to longer term soil moisture content changes on reinforcement load and wall face deformation changes after wall construction could be investigated using the Stiffness Method. However, the final wall design should not be conducted taking advantage of the reduced reinforcement loads due to soil cohesion, as post-construction changes in the reinforcement loads and wall face deformation are likely to be unacceptable.

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Appendix 15-F Description of Typical Temporary Shoring Systems and Selection Considerations

15-F-1 Fill Applications

While most temporary retaining systems are used in cut applications, some temporary retaining systems are also used in fill applications. Typical examples include the use of MSE walls to support preload fills that might otherwise encroach into a wetland or other sensitive area, the use of modular block walls or wrapped face geosynthetic walls to support temporary access road embankments or ramps, and the use of temporary wrapped face geosynthetic walls to support fills during intermediate construction stages.

MSE walls, including wrapped face geosynthetic walls, are well suited for the support of preload fills because they can be constructed quickly, are relatively inexpensive, are suitable for retaining tall fill embankments, and can tolerate significant settlements. Modular block walls without soil reinforcement (e.g., ecology block walls) are also easy to construct and relatively inexpensive; however they should only be used to support relatively short fill embankments and are less tolerant to settlement than MSE walls. Therefore, block walls are better suited to areas with firm subgrade soils where the retained fill thickness behind the walls is less than 15 feet.

15-F-2 MSE Walls

MSE walls are described briefly in Section 15.5.3, and extensively in Publication No. FHWA-NHI-00-043 (Elias, et al., 2001). In general, MSE walls consist of strips or sheets of steel or polymeric reinforcement placed as layers in backfill material and attached to a facing. Facings may consist of concrete blocks or panels, gabions, or a continuation of the reinforcement layer.

15-F-3 Prefabricated Modular Block Walls

Prefabricated modular block walls without soil reinforcement are discussed in Section 15.5.4 and should be designed as gravity retaining structures. Concrete blocks used for gravity walls typically consist of 2½- by 2½- by 5-foot solid rectangular concrete blocks designed to interlock with each other. They are typically cast from excess concrete at concrete batch plants and are relatively inexpensive. Because of their rectangular shape they can be stacked a variety of ways. Because of the tightly fitted configuration of a concrete block wall, oversized blocks will tend to fit together poorly. Occasionally, blocks from a concrete batch plant are found to vary in dimension by several inches.

15-F-4 Common Cut Applications

A wide range of temporary shoring systems are available for cut applications. Each temporary shoring system has advantages and disadvantages, conditions where the system is suitable or not suitable, and specific design considerations. The following sections provide a brief overview of many common temporary shoring systems for cut applications. The “Handbook of Temporary Structures in Construction” (Ratay, 1996) is another useful resource for information on the design and construction of temporary shoring systems.

15-F-5 Trench Boxes

Trench boxes are routinely used to protect workers during installation of utilities and other construction operations requiring access to excavations deeper than 4 feet. Trench boxes consist of two shields connected by internal braces and have a fixed width and height. The typical construction sequence consists of excavation of a trench and then setting the trench box into the excavation prior to allowing workers to gain access to the protected area within the trench box. For utility construction, the trench box is commonly pulled along the excavation by the excavator as the utility construction advances. Some trench boxes are designed such that the trench boxes can be stacked for deeper excavations.

The primary advantage of trench boxes is that they provide protection to workers for a low cost and no site specific design is generally required. Another advantage is that trench boxes are readily available and are easy to use. One disadvantage of trench boxes is that no support is provided to the soils—where existing improvements are located adjacent to the excavation, damage may result if the soils cave-in towards the trench box. Therefore, trench boxes are not suitable for soils that are too weak or soft to temporarily support themselves. Another disadvantage of trench boxes is the internal braces extend across the excavation and can impede access to the excavation. Finally, trench boxes provide no cutoff for groundwater; thus, a temporary dewatering system may be necessary for excavations that extend below the water table for trench boxes to be effective.

Trench boxes are most suitable for trenches or other excavations where the depth is greater than the width of the excavation and soil is present on both sides of the trench boxes. Trench boxes are not appropriate for excavations that are deeper than the trench box.

15-F-6 Sheet Piling

Sheet piling is a common temporary shoring system in cut applications and is particularly beneficial as the sheet piles can act as a diaphragm wall to reduce groundwater seepage into the excavation. Sheet piling typically consists of interlocking steel sheets that are much longer than they are wide. Sheets can also be constructed out of vinyl, aluminum, concrete, or wood; however, steel sheet piling is used most often due to its ability to withstand driving stresses and its ability to be removed and reused for other walls. Sheet piling is typically installed by driving with a vibratory pile driving hammer. For sheet piling in cut applications, the piling is installed first, then the soil in front of the wall is excavated or dredged to the design elevation. There are two general types of sheet pile walls: cantilever, and anchored/braced.

Sheet piling is most often used in waterfront construction; although, sheet piling can be used for many upland applications. One of the primary advantages of sheet piling is that it can provide a cutoff for groundwater flow and the piles can be installed without lowering the groundwater table. Another advantage of sheet piling is that it can be used for irregularly shaped excavations. The ability for the sheet piling to be removed makes sheet piling an attractive shoring alternative for temporary applications. The ability for sheet piling to be anchored by means of ground anchors or deadman anchors (or braced internally) allows sheet piling to be used where deeper excavations are planned or where large surcharge loading is present. One disadvantage of sheet piling is that it is installed by vibrating or driving; thus, in areas where vibration sensitive improvements or soils are present, sheet piling may not be appropriate. Another disadvantage is that where very dense soils are present or where cobbles, boulders or other obstructions are present, installation of the sheets is difficult.

15-F-7 Soldier Piles

Soldier pile walls are frequently used as temporary shoring in cut applications. The ability for soldier piles to withstand large lateral earth pressures and the proven use adjacent to sensitive infrastructure make soldier piles an attractive shoring alternative. Soldier pile walls typically consist of steel beams installed in drilled shafts; although, drilled shafts filled with steel cages and concrete or precast reinforced concrete beams can be used. Following installation of the steel beam, the shaft is filled with structural concrete, lean concrete, or a combination of the two. The soldier piles are typically spaced 6 to 8 feet on center. As the soil is excavated from in front of the soldier piles, lagging is installed to retain the soils located between adjacent soldier piles. The lagging typically consists of timber; however, reinforced concrete beams, reinforced shotcrete, or steel plates can also be used as lagging. Ground anchors, internal bracing, rakers, or deadman anchors can be incorporated in soldier pile walls where the wall height is higher than about 12 feet, or where backslopes or surcharge loading are present.

Soldier piles are an effective temporary shoring alternative for a variety of soil conditions and for a wide range of wall heights. Soldier piles are particularly effective adjacent to existing improvements that are sensitive to settlement, vibration, or lateral movement. Construction of soldier pile walls is more difficult in soils prone to caving, running sands, or where cobbles, boulders or other obstructions are present; however, construction techniques are available to deal with nearly all soil conditions. The cost of soldier pile walls is higher than some temporary shoring alternatives. In most instances, the steel soldier pile is left in place following construction. Where ground anchors or deadman anchors are used, easements may be required if the anchors extend outside the right-of-way/property boundary. Where ground anchors are used and soft soils are present below the base of the excavation, the toe of the soldier pile should be designed to prevent excessive settlements.

15-F-8 Prefabricated Modular Block Walls

In general, modular blocks (see Section 15.6.6.1.2) for cut applications require the soil deposit to have adequate standup time such that the excavation can be made and the blocks placed without excessive caving. Otherwise large temporary backcuts and subsequent backfill placement may be required. A key advantage to modular block walls is that the blocks can be removed and reused after the temporary structure is no longer needed. One disadvantage to using modular blocks in cut applications is that the blocks are placed in front of an excavation and the soils are initially not in full contact with the blocks unless the areas is backfilled. Some movement of the soil mass is required prior to load being applied to the blocks—this movement can be potentially damaging to upslope improvements.

15-F-9 Braced Cuts

Braced cuts are used in applications where a temporary excavation is required that provides support to the retained soils in order to reduce excessive settlement or lateral movement of the retained soils. Braced cuts are generally used for trenches or other excavations where soil is present on both sides of the excavation and construction activities are not affected by the presence of struts extending across the excavation. A variety of techniques are available for constructing braced cuts; however, most include a vertical element, such as a sheet pile, metal plate, or a soldier pile, that is braced across the excavation by means of struts. Many of the considerations discussed below for soldier pile walls and sheet piling apply to braced cuts.

15-F-10 Soil Nail Walls

The soil nail wall system consists of drilling and grouting rows of steel bars or "nails" behind the excavation face as it is excavated and then covering the face with reinforced shotcrete. The placement of soil nails reinforces the soils located behind the excavation face and increases the soil's ability to resist a mass of soil from sliding into the excavation. Soil nail walls are typically used in dense to very dense granular soils or stiff to hard, low plasticity, fine-grained soils. Soil nail walls are less cost effective in loose to medium dense sands or soft to medium stiff/high plasticity fine-grained soils.

The soils typically are required to have an adequate standup time (to allow placement of the steel wire mesh and/or reinforcing bars to be installed and the shotcrete to be placed). Soils that have short standup times are problematic for soil nailing. Many techniques are available for mitigating short standup time, such as installation of vertical elements (vertical soil nails or light steel beams set in vertical drilled shafts placed several feet on center along the perimeter of the excavation), drilling soil nails through soil berms, use of slot cuts, and flash-coating with shotcrete. Easements may be required if the soil nails extend outside the right-of-way/property boundary.

15-F-11 Uncommon Shoring Systems for Cut Applications

The following shoring systems require special, very detailed, expert implementation:

15-F-11.1 Diaphragm/Slurry Walls

Diaphragm/slurry walls are constructed by excavating a deep trench around the proposed excavation. The trench is filled with a weighted slurry that keeps the excavation open. The width of the trench is at least as wide as the concrete wall to be constructed. The slurry trench is completed by installing steel reinforcement cages and backfilling the trench with tremied structural concrete that displaces the slurry. The net result is a continuous wall that significantly reduces horizontal ground water flow. Once the concrete cures, the soil is excavated from in front of the slurry wall. Internal bracing and/or ground anchors can be incorporated into slurry walls. Diaphragm/slurry walls can be incorporated into a structure as permanent walls.

Diaphragm/slurry walls are most often used where groundwater is present above the base of the excavation. Slurry walls are also effective where contaminated groundwater is to be contained. Slurry walls can be constructed in dense soils where the use of sheet piling is difficult. Other advantages of slurry walls include the ability to withstand significant vertical and lateral loads, low construction vibrations, and the ability to construct slurry walls in low-headroom conditions. Slurry walls are particularly effective in soils where high groundwater and loose soils are present, and dewatering could lead to settlement related damage of adjacent improvements, assuming that the soils are not so loose or soft that the slurry is inadequate to prevent squeezing of the very soft soil.

In addition to detailed geotechnical design information, diaphragm/slurry walls require jobsite planning, preparation and control of the slurry, and contractors experienced in construction of slurry walls. For watertight applications, special design and construction considerations are required at the joints between each panel of the slurry wall.

15-F-11.2 Secant Pile Walls

Secant pile walls are another type of diaphragm wall that consist of interconnected drilled shafts. First, every other drilled shaft is drilled and backfilled with low strength concrete without steel reinforcement. Next, structural drilled shafts are installed between the low strength shafts in a manner that the structural shafts overlap the low strength shafts. The structural shafts are typically backfilled with structural concrete and steel reinforcement. The net result is a continuous wall that significantly reduces horizontal ground water flow while retaining soils behind the wall.

Secant pile walls are typically more expensive than many types of cut application temporary shoring alternatives; thus, the use of secant pile walls is limited to situations where secant pile walls are better suited to the site conditions than other shoring alternatives. Conditions where secant pile walls may be more favorable include high groundwater, the need to prevent migration of contaminated groundwater, sites where dewatering may induce settlements below adjacent improvements, sites with soils containing obstructions, and sites where vibrations need to be minimized.

15-F-11.3 Cellular Cofferdams

Sheet pile cellular cofferdams can be used for applications where internal bracing is not desirable due to interference with construction activities within the excavation. Cellular cofferdams are typically used where a dewatered work area or excavation is necessary in open water or where large dewatered heads are required. Cellular cofferdams consist of interlocking steel sheet piles constructed in a circle, or cell. The individual cells are constructed some distance apart along the length of the excavation or area to be dewatered. Each individual cell is joined to adjacent cells by arcs of sheet piles, thus providing a continuous structure. The cells are then filled with soil fill, typically granular fill that can be densified. The resulting structure is a gravity wall that can resist the hydrostatic and lateral earth pressures once the area within the cellular cofferdam is dewatered or excavated. As a gravity structure, cellular cofferdams need adequate bearing; therefore, sites where the cellular cofferdam can be founded on rock or dense soil are most suitable for these structures.

Cellular cofferdams are difficult to construct and require accurate placement of the interlocking sheet piles. Sites that require installation of sheet piles through difficult soils, such as through cobbles or boulders are problematic for cellular cofferdams and can result in driving the sheets out of interlock.

15-F-11.4 Frozen Soil Walls (Ground Freezing)

Frozen soil walls can be used for a variety of temporary shoring applications including construction of deep vertical shafts and tunneling. Frozen soil walls are typically used where conventional shoring alternatives are not feasible or have not been successful. Frozen soil walls can be constructed as gravity structures or as compressive rings. Ground freezing also provides an effective means of cutting of groundwater flows. Frozen soil has compressive strengths similar to concrete. Installation of a frozen soil wall can be completed with little vibration and can be completed around existing utilities or other infrastructure. Ground freezing is typically completed by installing rows of steel freeze pipes along the perimeter of the planned excavation. Refrigerated fluid is then circulated through the pipes at temperatures typically around -20°C to -30°C. Frozen soil forms around each freeze pipe until a continuous mass of frozen soil is present. Once the frozen soil reaches the design thickness, excavation can commence within the frozen soil.

Frozen soil walls can be completed in difficult soil and groundwater conditions where other shoring alternatives are not feasible. Frozen soil walls can provide an effective cutoff for groundwater and are well suited for containment of contaminated groundwater. Frozen soil walls are problematic in soils with rapid groundwater flows, such as coarse sands or gravels, due to the difficulty in freezing the soil. Flooding is also problematic to frozen soil walls where the flood waters come in contact with the frozen soil—a condition which can lead to failure of the shoring. Special care is required where penetrations are planned through frozen soil walls to prevent groundwater flows from flooding the excavation. Accurate installation of freeze pipes is required for deeper excavations to prevent windows of unfrozen soil. Furthermore, ground freezing can result in significant subsidence as the frozen ground thaws. If settlement sensitive structures are below or adjacent to ground that is to be frozen, alternative shoring means should be selected.

15-F-11.5 Deep Soil Mixing

Deep soil mixing (DSM) is an in-situ soil improvement technique used to improve the strength characteristics of panels or columns of native soils. DSM utilizes mixing shafts suspended from a crane to mix cement into the native soils. The result is soil mixed panels or columns of improved soils. Two types of DSM walls can be constructed: gravity walls and diaphragm-type walls. Gravity type DSM walls consist of columns or panels of improved soils configured in a pattern capable of resisting movement of soil into the excavation. Diaphragm-type DSM walls are constructed by improving the soil along the perimeter of the excavation and inserting vertical reinforcement into the improved soil immediately after mixing cement into the soil. The result is a low permeability structural wall that can be anchored with tiebacks, similar to a soldier pile wall, where the improved soil acts as the lagging.

Advantages with deep soil mixing gravity walls include the use of the native soils as part of the shoring system and reduced or no reinforcement. However, a significant volume of the native soils needs to be improved over a wide area to enable the improved soil to act as a gravity structure. Advantages with soil mixed diaphragm walls include the ability to control groundwater seepage, construction of the wall facing simultaneously with placement of steel soldier piles, and a thinner zone of improved soils compared to gravity DSM walls.

DSM walls can be installed top-down by wet methods where mechanical mixing systems combine soil with a cementitious slurry or through bottom up dry soil mixing where mechanical mixing systems mix pre-sheared soil with pneumatically injected cement or lime. DSM is generally appropriate for any soil that is free of boulders or other obstructions; although, it may not be appropriate for highly organic soils. DSM can be completed in very soft to stiff cohesive soils and very loose to medium dense granular soils.

15-F-11.6 Permeation Grouting

Permeation grouting involves the pressurized injection of a fluid grout to improve the strength of the in-situ soils and to reduce the soil's permeability. A variety of grouts are available—micro-fine cement grout and sodium silicate grout are two of the more frequently used types in permeation grouting. To be effective, the grout must be able to penetrate the soil; therefore, permeation grouting is not applicable in cohesive soils or granular soils with more than about 20 percent fines. Disadvantages of permeation grouting is the expense of the process and the high risk of difficulties. Permeation grouting, like ground freezing or jet grouting, can be used to create gravity retaining walls consisting of improved soils or can be used to create compression rings for access shafts or other circular excavations.

In addition to characterizing the soils gradation and stratigraphy, it is important to characterize the permeability of the soils to evaluate the suitability of permeation grouting.

15-F-11.7 Jet Grouting

Jet grouting is a ground improvement technique that can be used to construct temporary shoring walls and groundwater cutoff walls. Jet grouting can also be used to form a seal or strut at the base of an excavation. Jet grouting is an erosion based technology where high velocity fluids are injected into the soil formation to break down the soil structure and to mix the soil with a cementitious slurry to form columns of improved soil. Jet grouting can be used to construct diaphragm walls to cutoff groundwater flow and can be configured to construct gravity type shoring systems or compressive rings for circular shafts. Jet grouting is applicable to most soil conditions; however, high plasticity clays or stiff to hard cohesive soils are problematic for jet grouting.

Advantages with jet grouting include the ability to use of the native soils as part of the shoring system. A significant volume of the native soils needs to be improved over a wide area to enable the improved soil to act as a gravity structure. The width of the improved soil column is difficult to control, thus the final face of a temporary shoring wall may be irregular or protrude into the excavation.

15-F-12 Factors Influencing Choice of Temporary Shoring

A multitude of factors will influence the choice of temporary shoring systems for a particular application. The most common considerations are cost, subsurface constraints (i.e. difficult driving conditions, the need to cutoff groundwater seepage, etc.), site constraints (i.e. limited access, impacts to adjacent infrastructure, etc.), and local practice. The sections below, while not all-inclusive, provide a brief discussion of several of the factors that influence selection of temporary shoring systems.

15-F-12.1 Application

The first screening criteria for alternative temporary shoring options will be the purpose of the shoring—will it retain an excavation or support a fill.

15-F-12.2 Cut/fill Height

Some retaining systems are more suitable for supporting deep excavations/fill thicknesses than others. Temporary modular block walls are typically suitable only for relatively short fill embankments (less than 15 feet), while MSE walls can be designed to retain fills several tens of feet thick.

In cut applications, the common cantilever retaining systems (sheet piling and soldier piles) are typically most cost effective for retained soil heights of 12 to 15 feet or less. Temporary shoring walls in excess of 15 feet typically require bracing, either external (struts, rakers, etc.) or internal (ground anchors or dead-man anchors).

15-F-13 Soil Conditions

15-F-13.1 Dense Soils and Obstructions

Dense subsurface conditions, such as presented by glacial till or bedrock, result in difficult installation conditions for temporary shoring systems that are typically driven or vibrated into place (sheet piling). Cobbles, boulders and debris within the soil also often present difficult driving conditions. It is often easier to use drilling methods to install shoring in these conditions. However, oversize materials and dense conditions may also hinder conventional auger drilling, resulting in the need for specialized drilling equipment. Methods such as slurry trenches and grouting may become viable in areas with very difficult driving and drilling conditions.

15-F-13.2 Caving Conditions

Caving conditions caused by a combination of relatively loose cohesionless soils and/or groundwater seepage may result in difficult drilling conditions and the need to use casing and/or drilling slurry to keep the holes open.

15-F-13.3 Permeability

Soil permeability is based primarily on the soil grain size distribution and density. It influences how readily groundwater flows through a soil. If soils are very permeable and the excavation will be below the water level, then some sort of groundwater control will be required as part of the shoring system; this could consist of traditional dewatering methods or the use of shoring systems that also function as a barrier to seepage, such as sheet piling and slurry trench methods.

15-F-13.4 Groundwater, Bottom Heave and Piping

The groundwater level with respect to the proposed excavation depth will have a substantial influence on the temporary shoring system selected. Excavations that extend below the groundwater table and that are underlain by relatively permeable soils will require either dewatering, shoring systems that also function as a barrier to groundwater seepage, or some combination thereof. If the anticipated dewatering volumes are high, issues associated with treating and discharge of the effluent can be problematic. Likewise, large dewatering efforts can cause settlement of nearby structures if they are situated over compressible soils, or they may impact nearby contamination plumes, should they exist. Considerations for barrier systems include the depth to an aquitard to seal off groundwater flow and estimated flow velocities. If groundwater velocity is high, some barrier systems such as frozen ground and permeation grouting will not be suitable.

Bottom heave and piping can occur in soft/loose soils when the hydrostatic pressure below the base of the excavation is significantly greater than the resistance provided by the floor soils. In this case, temporary shoring systems that can be used to create a seepage barrier below the excavation, thus increasing the flow path and reducing the hydrostatic pressure below the base, may be better suited than those that do not function as a barrier. For example, sheet piling can be installed as a seepage barrier well below the base of the excavation, while soldier pile systems cannot. This is especially true if an aquitard is situated below the base of the excavation where the sheet piles can be embedded into the aquitard to seal off the groundwater flow path.

15-F-13.5 High Locked in Lateral Stresses

Glacially consolidated soils, especially fine-grained soils, often have high locked in lateral stresses because of the overconsolidation process (i.e. Ko can be much greater than a typical normally consolidated soil deposit). The Seattle Clay is an example of this type of soil, and much has been written about the performance of cuts into this material made to construct Interstate 5 (Peck, 1963; Sherif, 1966; Andrews, et al., 1966; and Strazer, et al., 1974). When cuts are made into soils with high locked in lateral stresses, they tend to rebound upon the stress relief, which can open up joints and fractures. Hydrostatic pressure buildup in the joints and fractures can function as a hydraulic jack and move blocks of soil, and movement can quickly degrade the shear strength of the soil. Therefore, for excavations into virgin material suspected of having high locked in lateral stresses, temporary shoring methods that limit the initial elastic rebound are required. For example, anchored shoring systems that are loaded and locked-off before the excavation will likely perform better than passive systems that allow the soil move, such as soil nails.

15-F-13.6 Compressible Soils

Compressible soils are more likely to impact the selection of temporary walls used to retain fills. MSE walls are typically more settlement tolerant than other fill walls, such as modular block walls.

15-F-13.7 Space Limitations

Space limitations include external constraints, such as right-of-way issues and adjacent structures, and internal constraints such as the amount of working space required. If excavations are required near existing right-of-ways, then temporary construction easements may be required to install the shoring system. Permanent easements may be required if the shoring systems include support from ground anchors or dead-man anchors that may remain after construction is complete. To minimize the need for temporary and permanent easements, cantilever walls or walls with external bracing (e.g. struts or rakers) should be considered. However, if the work space in front of the excavation needs to be clear, then shoring systems with external support may not be appropriate.

Existing infrastructure, such as underground utilities that cannot be relocated, may have the same impact on the choice of temporary shoring system as nearby right-of-ways.

15-F-13.8 Adjacent Infrastructure

The location of infrastructure adjacent to the site and the sensitivity of the infrastructure to settlement and/or vibrations will influence the selection of temporary shoring. For example, it may be necessary to limit dewatering or incorporate recharge wells if the site soils are susceptible to consolidation if the water table is lowered. If the adjacent infrastructure is brittle or supported above potentially liquefiable soils, it may be necessary to limit vibrations, which may exclude the selection of temporary shoring systems that are driven or vibrated into place, such as sheet piling.

The shoring system itself could also be sensitive to adjacent soil improvement or foundation installation activities. For example, soil improvement activities such as the installation of stone columns in loose to medium dense sands immediately in front of a shoring structure could cause subsidence of the loose sands and movement, or even failure, of the shoring wall. In such cases, the shoring wall shall be designed assuming that the soil immediately in front of the wall could displace significantly, requiring that the wall embedment be deepened and ground anchors be added.

15-F-14 References

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Appendix 15-G Testing and Acceptance Protocols for Tiebacks in Clay

Testing and Acceptance for Tiebacks Installed in Clay

The contents for this appendix are based on Allen (2020).

For tiebacks installed in intact glacially overconsolidated clay, paleolandslide deposits derived from the glacially overconsolidated clay, or otherwise disturbed glacial clay, a project specific protocol for tieback bond zone design, testing, and acceptance shall consist of the following:

- **Sacrificial pullout and sacrificial pullout with creep tests conducted on tiebacks in each soil unit in which tieback bond zones will be installed:**
  - To be able to extrapolate the pullout test results to longer bond zones, a minimum bond zone length of 4.6 m (15 ft) should be used for the test tiebacks to minimize the effect of load transfer rate nonlinearity along the bond zone soil-grout interface.
  - The testing protocol and analysis should be consistent with the protocol used for long-term tieback testing.
  - The pullout tests should be done in pairs. The first test is used to establish the values of T_c and T_{uw} that will be used for the second pullout test, loading the tieback incrementally until pullout is achieved, if possible. The sacrificial tieback testing schedule for this testing is provided in Tables 15-G-1 and 15-G-2.
  - The loading increments should be based on the Factored Design Load (FDL), using a load increment of 0.10FDL to 0.20FDL. A load factor of 1.35, consistent with required load factors in AASHTO (2020), should be used to determine the FDL.
  - The second pullout test is also loaded incrementally until T_{uw} from the first test is achieved, at which point a 72-hour creep test is conducted. If in the second test the creep rate versus load level curve is starting to sweep upward sooner than expected, it may become necessary to use a lower value of T_{uw} for the 72-your load hold. In that case, the next increment of load increase above T_{uw} may need to start lower than shown in Table 15-G-2.
  - Once the creep test is completed for the second tieback, the tieback load is increased incrementally above T_{uw} until pullout is achieved, if possible.
  - At each load increment, the load level should be held for 60 minutes and creep measured.
- **Contractor designed tieback bond zone diameter and length:** The results from these sacrificial verification (pullout) tests described above, if they are successful, should be used to design the tieback bond zone length and diameter for the proposed production tieback installation method. To obtain the average bond zone soil-grout interface adhesion for the final design of the tieback bond zone, a resistance factor of 0.67 (i.e., the reciprocal of the safety factor, or 1/1.5 = 0.67) should be applied to T_{uw} determined from these pullout and extended creep tests. If the tiebacks are in disturbed glacially overconsolidated clay (e.g., paleolandslide or otherwise similar deposits), a resistance factor of 0.45 should be used to account for the increased variability of the deposit.

- **Production creep performance tests:** Five percent of the production tiebacks (or a minimum of 3 tiebacks per wall, whichever is greater) should be subjected to a creep performance test. The tieback testing schedule for this creep performance testing is provided Table 15-G-3.
- **Production cyclic performance tests, but with no longer-term creep testing:** Five percent of the production tiebacks (or a minimum of 3 tiebacks per wall, whichever is greater) should be subjected to a cyclic performance test. In cyclic performance tests the highest load tested is held for 60 minutes to determine the creep rate. The tieback testing schedule for this cyclic performance testing is provided Table 15-G-4.
- **Production proof tests conducted on all remaining tiebacks in each wall:** In proof tests the highest load tested is held for 60 minutes to determine the creep rate. The tieback testing schedule for proof testing is provided Table 15-G-5.

Table 15-G-1 Sacrificial pullout test schedule for tiebacks in glacial clay soil units (first test)

Load*	Hold Time (minutes)
AL	--
0.20 FDL	60
0.40 FDL	60
0.50 FDL	60
0.60 FDL	60
0.70 FDL	60
0.80 FDL	60
0.90 FDL	60
1.0 FDL	60
1.2 FDL	60
1.4 FDL	60
1.6 FDL	60
1.8 FDL	60
2.0 FDL	60

*FDL = Factored Design Load. Failure is defined as the tieback being unable to hold the load without continued movement (pullout).

Table 15-G-2 Sacrificial pullout test schedule for tiebacks in glacial clay soil units with 72-hour creep test (second test)

Load*	Hold Time (minutes)
AL	--
0.20 T_{uw}	60
0.40 T_{uw}	60
0.50 T_{uw}	60
0.60 T_{uw}	60
0.70 T_{uw}	60
0.80 T_{uw}	60
0.90 T_{uw}	60
T_{uw} from first test	4,320 (72 hrs)
1.2 FDL	60
1.4 FDL	60
1.6 FDL	60
1.8 FDL	60
2.0 FDL	60

*FDL = Factored Design Load. Failure is defined as the tieback being unable to hold the load without continued movement (pullout).

Table 15-G-3 Production tieback creep performance test schedule for tiebacks in glacial clay soil units

*Load	Hold Time (minutes)
AL	--
0.20 FDL	60
0.40 FDL	60
0.60 FDL	60
0.80 FDL	60
1.00 FDL	360 (6 hrs)

*Conduct on 5% of the production tiebacks in each wall, but no less than 3 tiebacks per wall.

Table 15-G-4 Production tieback cyclic performance test schedule for tiebacks in glacial clay soil units

[a]Load	Hold Time (minutes)
AL	--
0.25 FDL	--
AL	--
0.25 FDL	--
0.50 FDL	--
AL	--
0.25 FDL	--
0.50 FDL	--
0.75 FDL	--
AL	--
0.25 FDL	--
0.50 FDL	--
0.75 FDL	--
1.00 FDL	[b]60
AL	--
Jack to lock-off load	__--

[a]Conduct on 5% of the production tiebacks in each wall, but no less than 3 tiebacks per wall.

[b]If the anchor movement between 6 and 60 minutes exceeds 0.04 inches (0.03 inches for pressure- and post-grouted tiebacks), the maximum test load shall be held for an additional 300 minutes.

Table 15-G-5 Production tieback proof test schedule for tiebacks in glacial clay soil units

[a]Load	Hold Time (minutes)
AL	--
0.25 FDL	10
0.50 FDL	10
0.75 FDL	10
1.00 FDL	[b]60

[a]Conduct on all remaining production tiebacks in each wall.

[b]If the anchor movement between 6 and 60 minutes exceeds 0.04 inches (0.03 inches for pressure- and post-grouted tiebacks), the maximum test load shall be held for an additional 300 minutes.

Creep test measurement times and tieback acceptance criteria shall be as provided in Table 15-G-6 for tiebacks in clay.

Table 15-G-6 Creep measurement times and creep criteria for tiebacks in clay soil units

Hold Time (minutes)	Measurement Times (minutes)	Creep Criterion*
10	1, 2, 3, 4, 5, 6, and 10	0.75 mm/log cycle (0.03 inch per log cycle) of time
60	1, 2, 3, 4, 5, 6, 10, 20, 30, 40, 50, and 60	1.0 mm/log cycle (0.04 inch per log cycle) of time
360	1, 2, 3, 4, 5, 6, 10, 20, 30, 40, 50, 60, then every 30 minutes up to 360 minutes	1.0 mm/log cycle (0.04 inch per log cycle) of time
4320	1, 2, 3, 4, 5, 6, 10, 20, 30, 40, 50, 60, then every 30 minutes up to 4,320 minutes	+1.5 mm/log cycle (0.06 inch per log cycle) of time

*Adjust criterion based on test results from the sacrificial pullout tests, but no greater than shown in this table.

+Limit to 1.0 mm/log cycle (0.04 in./log cycle) of time if testing pressure- or post-grouted tiebacks.

Use slope of creep curve (i.e., such as from a log linear regression) to determine creep rate for comparison to the creep criterion.

Additional Implementation Requirements for Production Tieback Walls

Based on the results of this study, the following are recommendations that should be considered when developing tieback testing programs for production walls:

1. The special testing requirements provided in this appendix should be considered applicable to tiebacks installed in overconsolidated clays, both in an intact condition and in a disturbed condition (e.g., partially reconsolidated paleolandslide deposits such as the Vashon Unsorted), in the central Puget Sound region. This testing is especially important when, for the soil surrounding the tieback bond zones, the soil consistency index is less than 0.9 and the liquid limit is greater than 50, but should also be considered for any clayey silt, silty clay, or clay. See the report conclusions (Allen 2020) for guidance regarding the soil data requirements needed to make this assessment.
2. Two sacrificial pullout/creep test tiebacks should be installed in each clay unit; one sacrificial test is for pullout and the other is for pullout and creep testing (see Tables 15-G-1 and 15-G-2). The load zone should be in the target soil unit. The verification (pullout) tests must be performed prior to production tieback installation.
3. A minimum 15-foot-long bond length is required for the sacrificial verification test tiebacks. Additional tendon steel should be added to the test tiebacks to make sure the tieback can be loaded to at least twice the FDL and high enough to achieve pullout, if possible.
4. Except for the load-cycled performance tests, no load cycling is allowed for tiebacks in glacial clay units. No retesting is allowed for tiebacks in glacial clay units.
5. If the verification (pullout) test results indicate good creep performance at the 4,320-minute hold time, a creep criterion up to 0.06 inch per log cycle of time could be considered by the Engineer for straight shafted tiebacks. For pressure-grouted and post-grouted tiebacks, a creep criterion up to 0.04 inch per log cycle of time could be considered.

6. If a tieback fails in creep, lock off the load at 50% of the load at creep failure. Additional tiebacks may be required to achieve the wall design load resistance.
7. The sacrificial verification test load-hold periods shall start as soon as the test load is applied and the anchor movement, with respect to a fixed reference, shall be measured and recorded in accordance with Table 15-G-6.
8. The maximum test load in a cyclic performance test shall be held for 60 minutes. The load-hold period shall start as soon as the maximum test load is applied and the tieback movement, with respect to a fixed reference, shall be measured and recorded in accordance with Table 15-G-6. If the anchor movement between 6 and 60 minutes exceeds 0.04 inches (0.03 inches for pressure- and post-grouted tiebacks), the maximum test load shall be held for an additional 300 minutes. If the load-hold is extended the anchor movement shall be recorded in accordance with Table 15-G-6.
9. The maximum test load in a proof test shall be held for 60 minutes. The load-hold period shall start as soon as the maximum test load is applied and the anchor movement, with respect to a fixed reference, shall be measured and recorded in accordance with Table 15-G-6. If the anchor movement between 6 and 60 minutes exceeds 0.04 inches, the maximum test load shall be held for an additional 300 minutes. If the load-hold is extended, the tieback movement shall be recorded in accordance with the 360-minute creep measurements listed in Table 15-G-6.

AL = Alignment Load, FDL = Factored Design Load

References

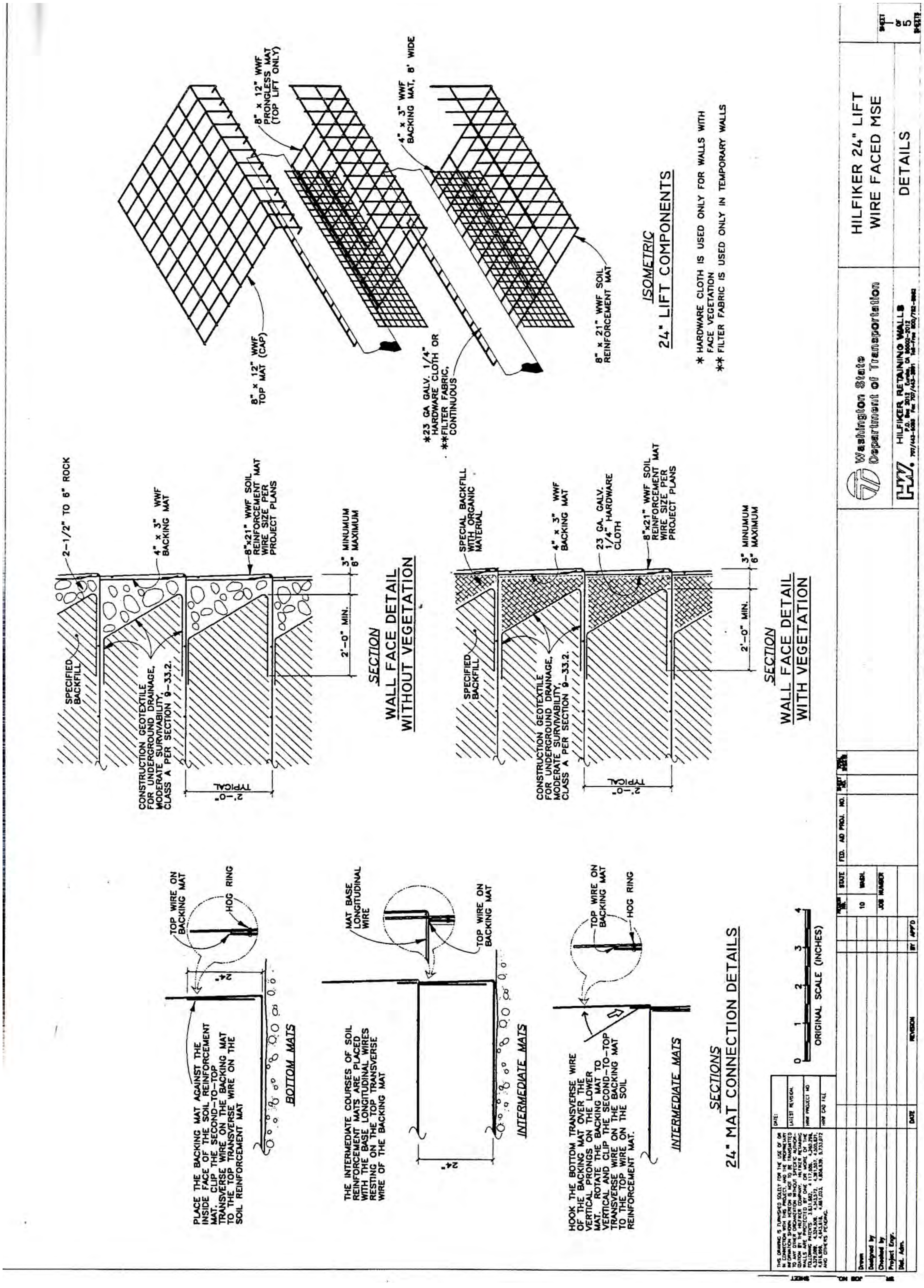
Allen, Tony M., 2020, I-90 *Demonstration Project: Long-Term Performance of Tiebacks Installed in Glacially Overridden Clays*, Washington State Department of Transportation, Research Report WA-RD 823.1, 252 pp.

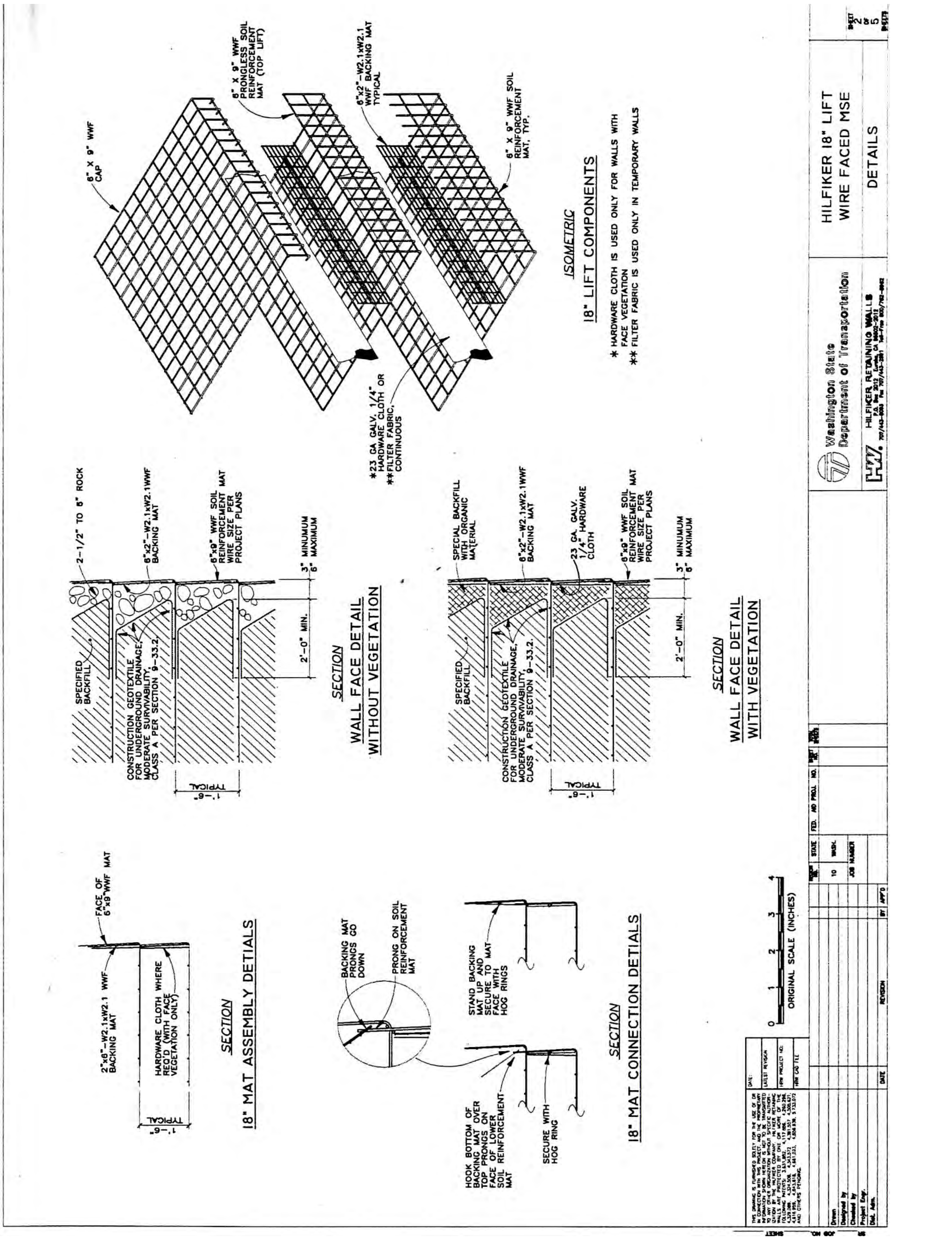
Appendix 15-H Preapproved Wall Appendix: Specific Requirements and Details for Hilfiker Welded Wire Faced Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific design requirements shall be met:

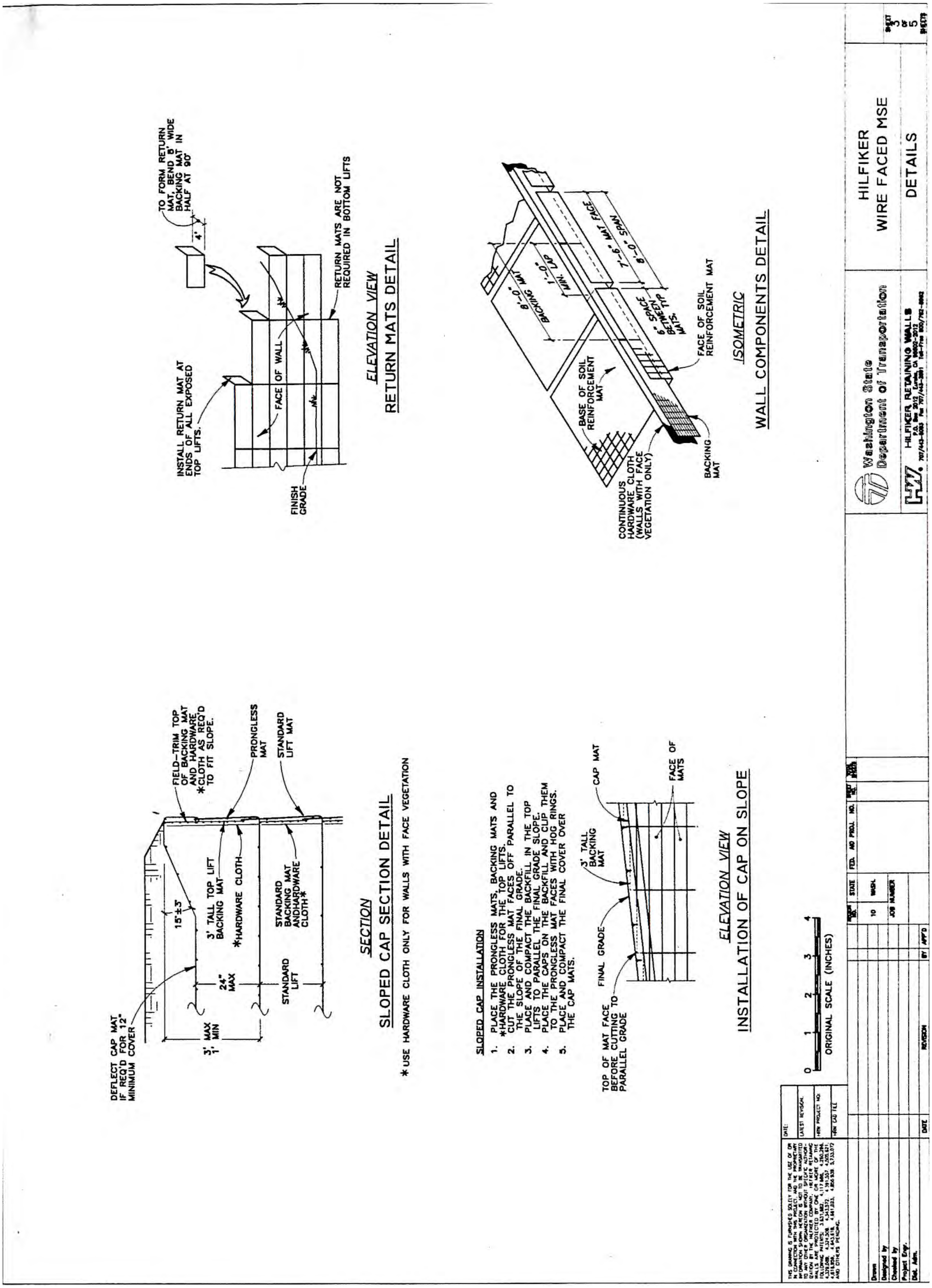
> No HITEC evaluation report is currently available for this wall system. Design procedures for specific elements of the wall system have been provided to WSDOT in a letter dated September 15, 2003. The design procedures used by Hilfiker Retaining Walls are in full conformance with the AASHTO *Standard Specifications* for Highway Bridges (2002). Interim approval is given for the continued use of the AASHTO *Standard Specifications* as the basis for design.
>
> Regarding the soil reinforcement material, the minimum wire size acceptable for permanent walls is W4.5 for the longitudinal wires. For the transverse wires, the minimum wire size shall be W3.5. For all permanent walls, the welded wire shall be galvanized in accordance with the AASHTO LRFD specifications. For temporary walls, galvanization is not required, but the life of the wire shall be designed to be adequate for the intended life.
>
> Regarding the backing mats used in the welded wire facing, the minimum clear opening dimension of the backing mat shall not exceed the minimum particle size of the wall facing backfill. The maximum particle size for the wall facing backfill shall be 6 inches.
>
> The maximum vertical spacing of soil reinforcement shall be 24 inches.
>
> The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet. Larger diameter culverts or obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers.
>
> This wall system is preapproved for a welded wire/gravel fill face for vertical to near vertical facing batter and welded wire vegetated face for wall face batters as steep as 6V:1H. This preapproval presumes that the facing tolerances in the WSDOT *Standard Specifications* Section 6-13.3(1) for welded wire faced walls are met.

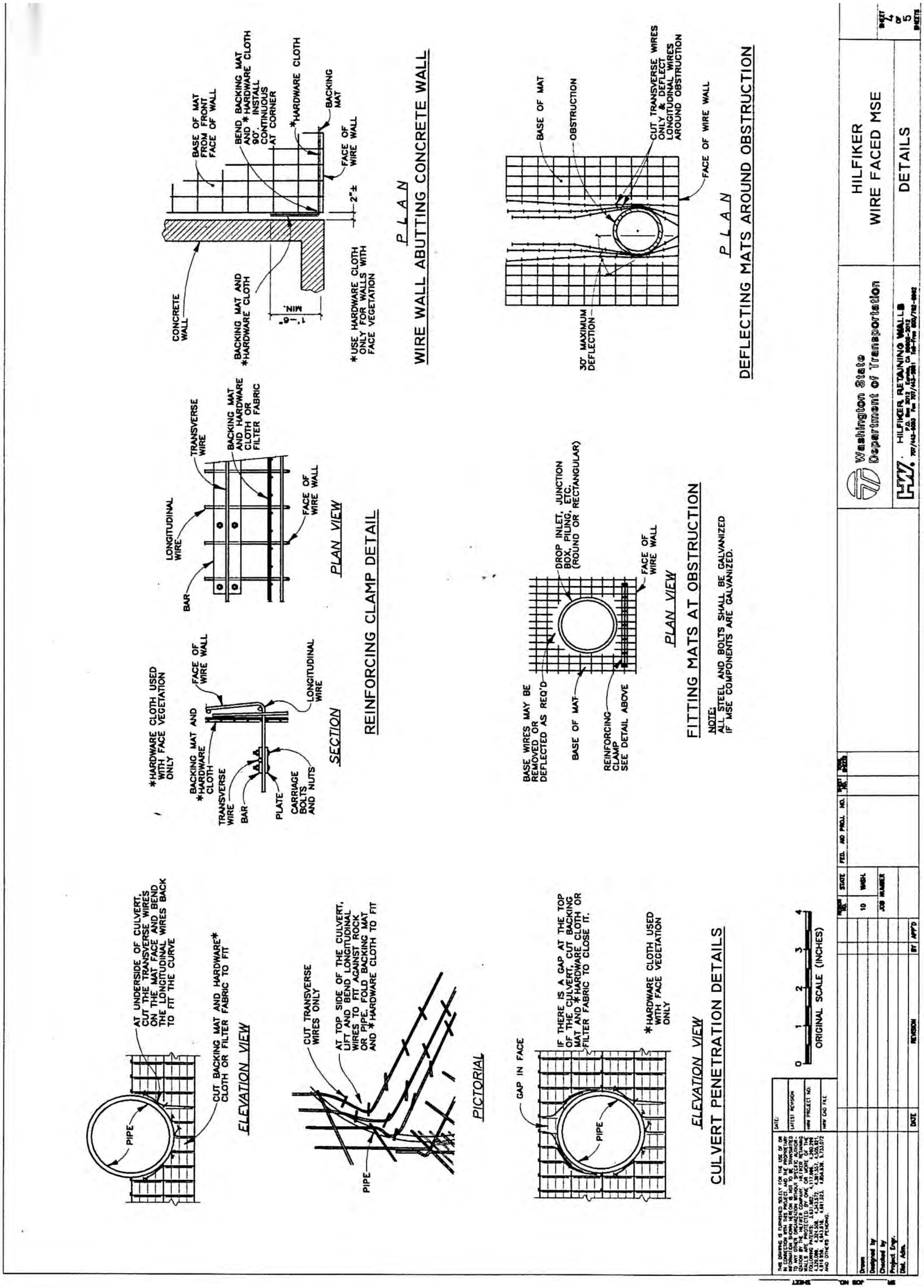
The following standard details shall be used for the Hilfiker Welded Wire Faced Wall system:


HILFIKER 24" LIFT
WIRE FACED MSE
DETAILS
Washington State Department of Transportation
HILFIKER RETAINING WALLS
ISOMETRIC
24" LIFT COMPONENTS
8" x 12" WWF TOP MAT (CAP)
8" x 12" WWF PRONGLESS MAT (TOP LIFT ONLY)
4" x 3" WWF BACKING MAT, 8' WIDE
*23 GA GALV. 1/4" HARDWARE CLOTH OR **FILTER FABRIC, CONTINUOUS
8" x 21" WWF SOIL REINFORCEMENT MAT
* HARDWARE CLOTH IS USED ONLY FOR WALLS WITH FACE VEGETATION
** FILTER FABRIC IS USED ONLY IN TEMPORARY WALLS
SECTION
WALL FACE DETAIL WITHOUT VEGETATION
2-1/2" TO 6" ROCK
4" x 3" WWF BACKING MAT
8"x21" WWF SOIL REINFORCEMENT MAT WIRE SIZE PER PROJECT PLANS
SPECIFIED BACKFILL
CONSTRUCTION GEOTEXTILE FOR UNDERGROUND DRAINAGE, MODERATE SURVIVABILITY, CLASS A PER SECTION 9-33.2.
2'-0" MIN.
3" MINUMUM
6" MAXIMUM
2'-0" TYPICAL
SECTION
WALL FACE DETAIL WITH VEGETATION
SPECIAL BACKFILL WITH ORGANIC MATERIAL
23 GA. GALV. 1/4" HARDWARE CLOTH
SECTIONS
24" MAT CONNECTION DETAILS
BOTTOM MATS
INTERMEDIATE MATS
TOP WIRE ON BACKING MAT
HOG RING
MAT BASE LONGITUDINAL WIRE
24"
PLACE THE BACKING MAT AGAINST THE INSIDE FACE OF THE SOIL REINFORCEMENT MAT. CLIP THE SECOND-TO-TOP TRANSVERSE WIRE ON THE BACKING MAT TO THE TOP TRANSVERSE WIRE ON THE SOIL REINFORCEMENT MAT
THE INTERMEDIATE COURSES OF SOIL REINFORCEMENT MATS ARE PLACED WITH THE BASE LONGITUDINAL WIRES RESTING ON THE TOP TRANSVERSE WIRE OF THE BACKING MAT
HOOK THE BOTTOM TRANSVERSE WIRE OF THE BACKING MAT OVER THE VERTICAL PRONGS ON THE LOWER MAT. ROTATE THE BACKING MAT TO VERTICAL AND CLIP THE SECOND-TO-TOP TRANSVERSE WIRE ON THE BACKING MAT TO THE TOP WIRE ON THE SOIL REINFORCEMENT MAT.
ORIGINAL SCALE (INCHES)
SHEET 1 OF 5 SHEETS

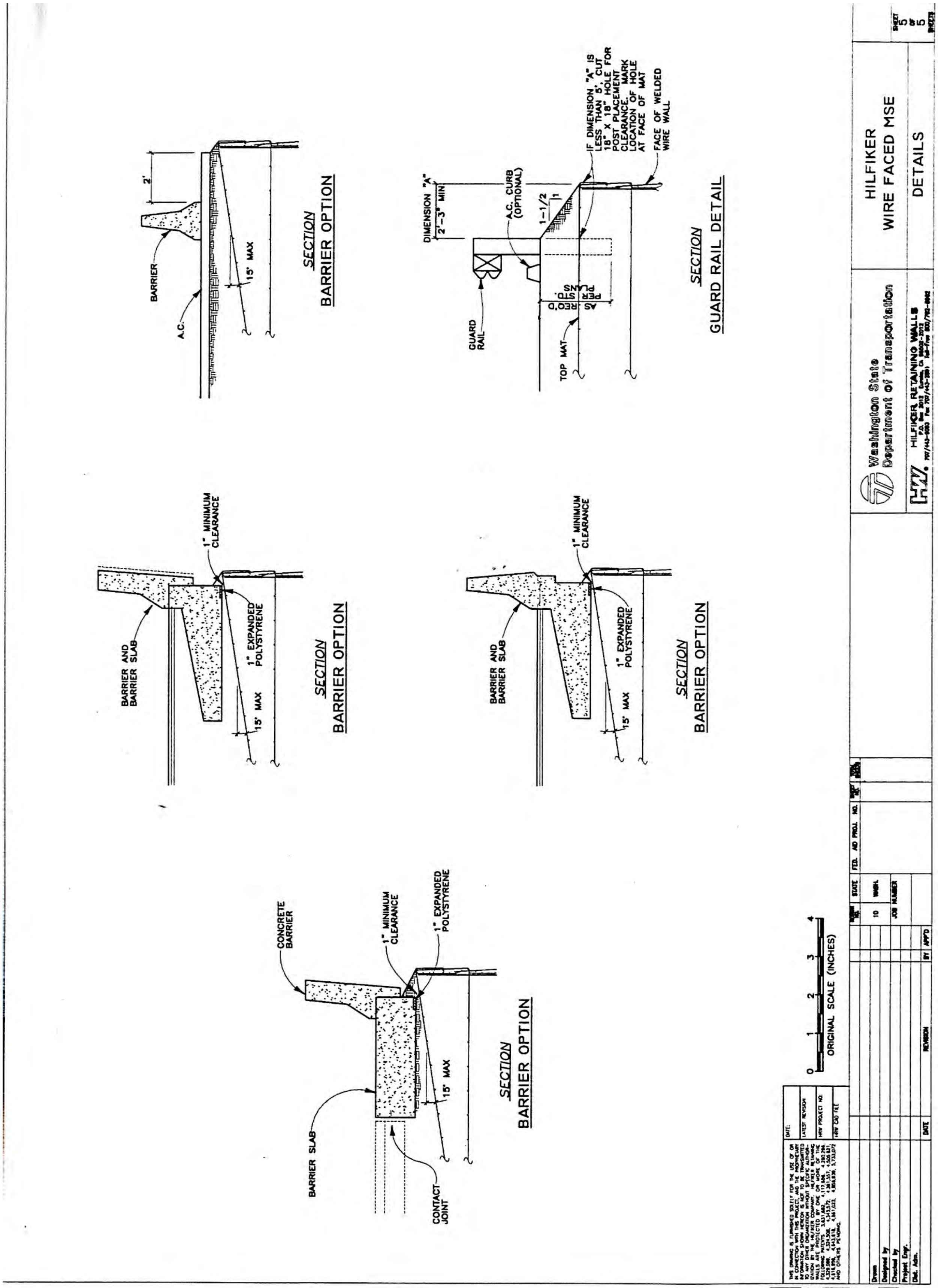


6" X 9" WWF CAP
6" X 9" WWF PRONGLESS SOIL REINFORCEMENT MAT (TOP LIFT)
6"x2"-W2.1xW2.1 WWF BACKING MAT TYPICAL
6" X 9" WWF SOIL REINFORCEMENT MAT, TYP.
*23 GA GALV. 1/4" HARDWARE CLOTH OR **FILTER FABRIC, CONTINUOUS
ISOMETRIC
18" LIFT COMPONENTS
* HARDWARE CLOTH IS USED ONLY FOR WALLS WITH FACE VEGETATION
** FILTER FABRIC IS USED ONLY IN TEMPORARY WALLS
2-1/2" TO 8" ROCK
6"x2"-W2.1xW2.1WWF BACKING MAT
6"x9" WWF SOIL REINFORCEMENT MAT WIRE SIZE PER PROJECT PLANS
3" MINUMUM
6" MAXIMUM
SPECIFIED BACKFILL
CONSTRUCTION GEOTEXTILE FOR UNDERGROUND DRAINAGE, MODERATE SURVIVABILITY, CLASS A PER SECTION 9-33.2
2'-0" MIN.
1'-6" TYPICAL
SECTION
WALL FACE DETAIL WITHOUT VEGETATION
SPECIAL BACKFILL WITH ORGANIC MATERIAL
23 GA. GALV. 1/4" HARDWARE CLOTH
SECTION
WALL FACE DETAIL WITH VEGETATION
FACE OF 6"x9"WWF MAT
2"x6"-W2.1xW2.1 WWF BACKING MAT
HARDWARE CLOTH WHERE REQ'D (WITH FACE VEGETATION ONLY)
SECTION
18" MAT ASSEMBLY DETIALS
BACKING MAT PRONGS GO DOWN
PRONG ON SOIL REINFORCEMENT MAT
STAND BACKING MAT UP AND SECURE TO MAT FACE WITH HOG RINGS
HOOK BOTTOM OF BACKING MAT OVER TOP PRONGS ON FACE OF LOWER SOIL REINFORCEMENT MAT
SECURE WITH HOG RING
SECTION
18" MAT CONNECTION DETIALS
0 1 2 3 4
ORIGINAL SCALE (INCHES)
Washington State Department of Transportation
HILFIKER RETAINING WALLS
HILFIKER 18" LIFT WIRE FACED MSE
DETAILS
SHEET 2 OF 5 SHEETS


DEFLECT CAP MAT IF REQ'D FOR 12" MINIMUM COVER
15'±3'
FIELD-TRIM TOP OF BACKING MAT AND HARDWARE *CLOTH AS REQ'D TO FIT SLOPE.
PRONGLESS MAT
STANDARD LIFT MAT
3' TALL TOP LIFT BACKING MAT
*HARDWARE CLOTH
STANDARD BACKING MAT ANDHARDWARE CLOTH*
24" MAX
3' MAX 1' MIN
STANDARD LIFT
SECTION
SLOPED CAP SECTION DETAIL
* USE HARDWARE CLOTH ONLY FOR WALLS WITH FACE VEGETATION
SLOPED CAP INSTALLATION
1. PLACE THE PRONGLESS MATS, BACKING MATS AND *HARDWARE CLOTH FOR THE TOP LIFTS.
2. CUT THE PRONGLESS MAT FACES OFF PARALLEL TO THE SLOPE OF THE FINAL GRADE.
3. PLACE AND COMPACT THE BACKFILL IN THE TOP LIFTS TO PARALLEL THE FINAL GRADE SLOPE.
4. PLACE THE CAPS ON THE BACKFILL AND CLIP THEM TO THE PRONGLESS MAT FACES WITH HOG RINGS.
5. PLACE AND COMPACT THE FINAL COVER OVER THE CAP MATS.
TOP OF MAT FACE BEFORE CUTTING TO PARALLEL GRADE
FINAL GRADE
3' TALL BACKING MAT
CAP MAT
FACE OF MATS
ELEVATION VIEW
INSTALLATION OF CAP ON SLOPE
INSTALL RETURN MAT AT ENDS OF ALL EXPOSED TOP LIFTS.
TO FORM RETURN MAT, BEND 8' WIDE BACKING MAT IN HALF AT 90°
4'
FACE OF WALL
FINISH GRADE
RETURN MATS ARE NOT REQUIRED IN BOTTOM LIFTS
ELEVATION VIEW
RETURN MATS DETAIL
8'-0" MAT
BACKING MAT
1'-0" MIN. LAP
7'-6" MAT FACE
8'-0" SPAN
6" SPACE BETWEEN MATS, TYP
FACE OF SOIL REINFORCEMENT MAT
BASE OF SOIL REINFORCEMENT MAT
CONTINUOUS HARDWARE CLOTH (WALLS WITH FACE VEGETATION ONLY)
BACKING MAT
ISOMETRIC
WALL COMPONENTS DETAIL
ORIGINAL SCALE (INCHES)
Washington State Department of Transportation
HILFIKER RETAINING WALLS
HILFIKER WIRE FACED MSE DETAILS
SHEET 3 OF 5 SHEETS



WIRE WALL ABUTTING CONCRETE WALL
PLAN
CONCRETE WALL
BACKING MAT AND *HARDWARE CLOTH
1'-6" MIN.
*USE HARDWARE CLOTH ONLY FOR WALLS WITH FACE VEGETATION
BASE OF MAT FROM FRONT FACE OF WALL
BEND BACKING MAT AND *HARDWARE CLOTH 90°. INSTALL CONTINUOUS AT CORNER
*HARDWARE CLOTH
BACKING MAT
FACE OF WIRE WALL
2"±
DEFLECTING MATS AROUND OBSTRUCTION
30° MAXIMUM DEFLECTION
BASE OF MAT
OBSTRUCTION
CUT TRANSVERSE WIRES ONLY & DEFLECT LONGITUDINAL WIRES AROUND OBSTRUCTION
FACE OF WIRE WALL
REINFORCING CLAMP DETAIL
SECTION
PLAN VIEW
*HARDWARE CLOTH USED WITH FACE VEGETATION ONLY
BACKING MAT AND *HARDWARE CLOTH
TRANSVERSE WIRE
BAR
PLATE
CARRIAGE BOLTS AND NUTS
FACE OF WIRE WALL
LONGITUDINAL WIRE
BACKING MAT AND HARDWARE CLOTH OR FILTER FABRIC
FITTING MATS AT OBSTRUCTION
BASE WIRES MAY BE REMOVED OR DEFLECTED AS REQ'D
BASE OF MAT
REINFORCING CLAMP SEE DETAIL ABOVE
DROP INLET, JUNCTION BOX, PILING, ETC. (ROUND OR RECTANGULAR)
NOTE: ALL STEEL AND BOLTS SHALL BE GALVANIZED IF MSE COMPONENTS ARE GALVANIZED.
CULVERT PENETRATION DETAILS
ELEVATION VIEW
PIPE
AT UNDERSIDE OF CULVERT, CUT THE TRANSVERSE WIRES ON THE MAT FACE AND BEND THE LONGITUDINAL WIRES BACK TO FIT THE CURVE
CUT BACKING MAT AND HARDWARE* CLOTH OR FILTER FABRIC TO FIT
PICTORIAL
CUT TRANSVERSE WIRES ONLY
AT TOP SIDE OF THE CULVERT, LIFT AND BEND LONGITUDINAL WIRES TO FIT AGAINST ROCK OR PIPE. FOLD BACKING MAT AND *HARDWARE CLOTH TO FIT
GAP IN FACE
IF THERE IS A GAP AT THE TOP OF THE CULVERT, CUT BACKING MAT AND *HARDWARE CLOTH OR FILTER FABRIC TO CLOSE IT.
*HARDWARE CLOTH USED WITH FACE VEGETATION ONLY
ORIGINAL SCALE (INCHES)
Washington State Department of Transportation
HILFIKER RETAINING WALLS
HILFIKER WIRE FACED MSE DETAILS


SECTION
BARRIER OPTION
SECTION
GUARD RAIL DETAIL
BARRIER
A.C.
15° MAX
2'
DIMENSION "A" 2'-3" MIN
A.C. CURB (OPTIONAL)
GUARD RAIL
TOP MAT
AS REQ'D PER STD. PLANS
IF DIMENSION "A" IS LESS THAN 5', CUT 18" X 18" HOLE FOR POST PLACEMENT CLEARANCE. MARK LOCATION OF HOLE AT FACE OF MAT
FACE OF WELDED WIRE WALL
BARRIER AND BARRIER SLAB
1" MINIMUM CLEARANCE
1" EXPANDED POLYSTYRENE
CONCRETE BARRIER
BARRIER SLAB
CONTACT JOINT
ORIGINAL SCALE (INCHES)
Washington State Department of Transportation
HILFIKER RETAINING WALLS
HILFIKER WIRE FACED MSE
DETAILS

Appendix 15-I Preapproved Wall Appendix: Specific Requirements and Details for Eureka Reinforced Soil Concrete Panel Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply to the design of the Hilfiker Eureka Reinforced Soil concrete 5 feet × 5 feet panel faced retaining wall:

No HITEC evaluation report is currently available for this wall system. The design procedures used by Hilfiker Retaining Walls are based on the AASHTO Standard Specifications for Highway Bridges (2002). Therefore, for internal stability of the wall, the AASHTO Simplified Method shall be used. Interim approval is given for the continued use of the AASHTO Standard Specifications as the basis for design.

Note the connector shall be designed to have adequate life considering corrosion loss.

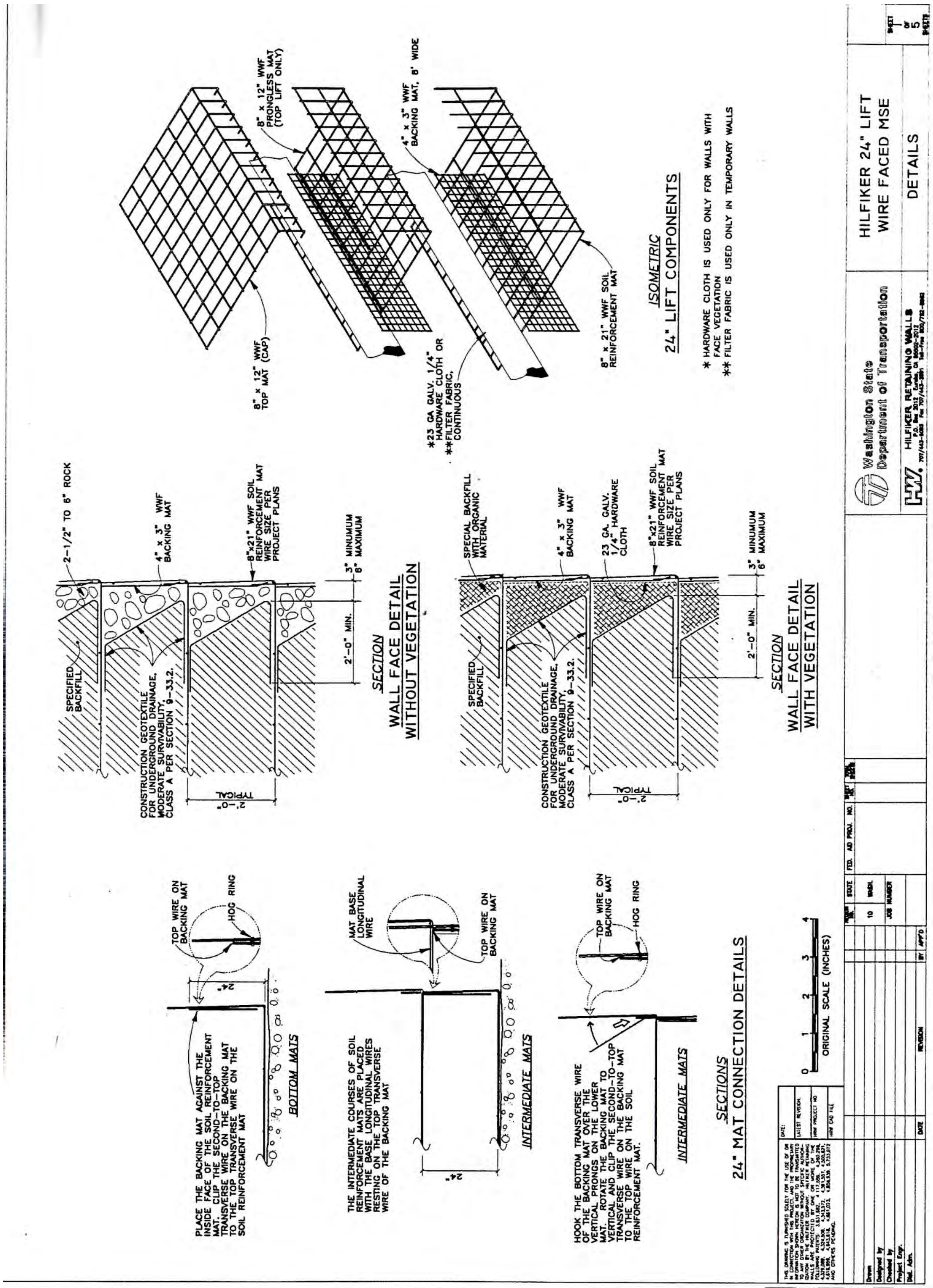
Furthermore, the connector loops embedded in the facing panels shall be lined up such that the steel grid reinforcement cross bar at the connection is uniformly loaded.

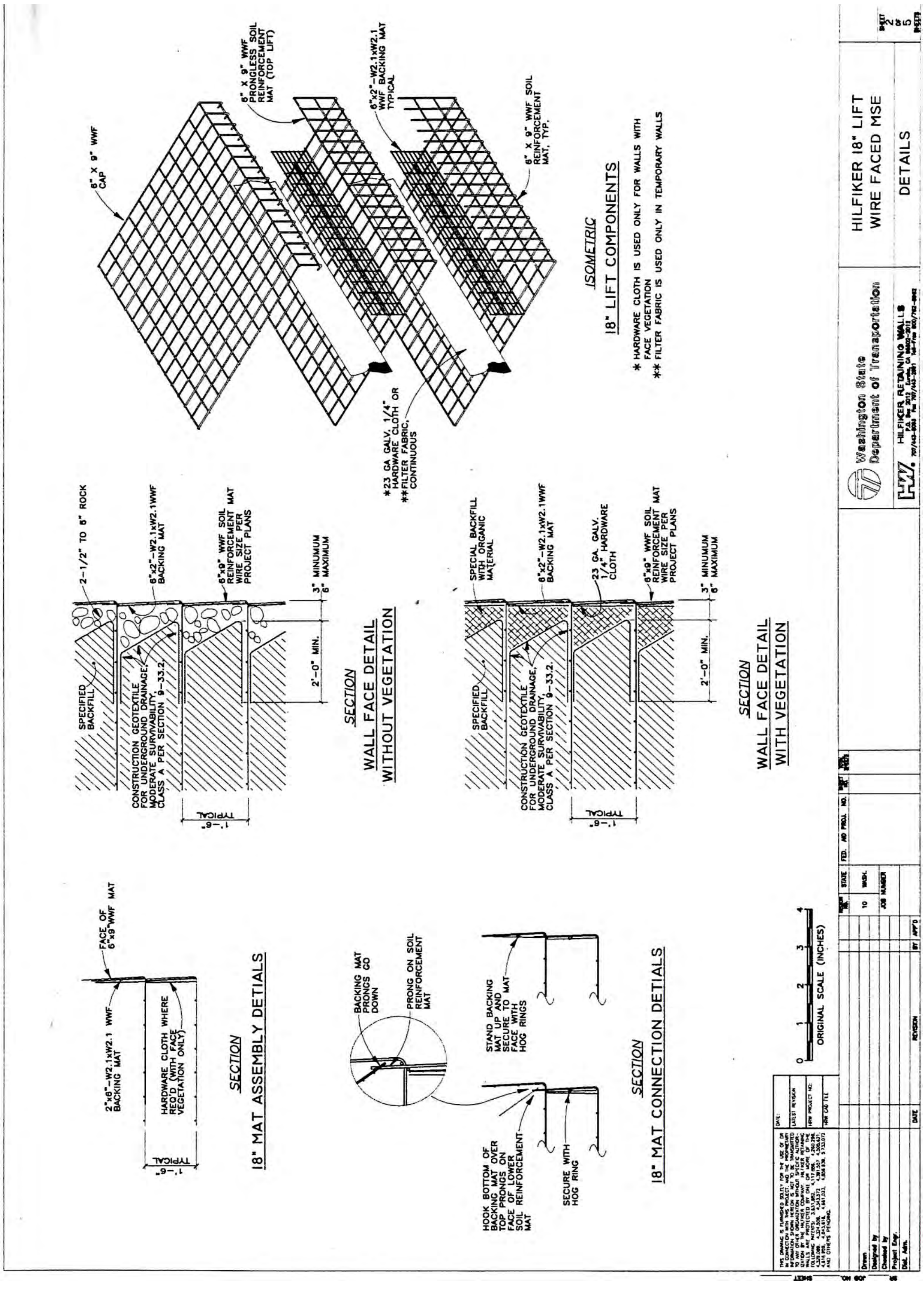
Therefore, regarding the alignment of the bearing surfaces of the embedded anchors, once the steel welded wire grid is inserted into the loops, no loop shall have a gap between the loop and the steel welded wire grid cross bar of more than 0.125 inch.

Reinforcement pullout shall be calculated based on the default values for steel grid reinforcement provided in the AASHTO Specifications. If, at some future time product and soil specific pullout data is provided to support use of non-default pullout interaction coefficients, it should be noted that LRFD pullout resistance design using these product and soil specific interaction coefficients has not been calibrated using product specific data statistics and reliability theory. Therefore, the specified resistance factors in the GDM and AASHTO LRFD Specifications should not be considered applicable to product specific pullout interaction coefficients.

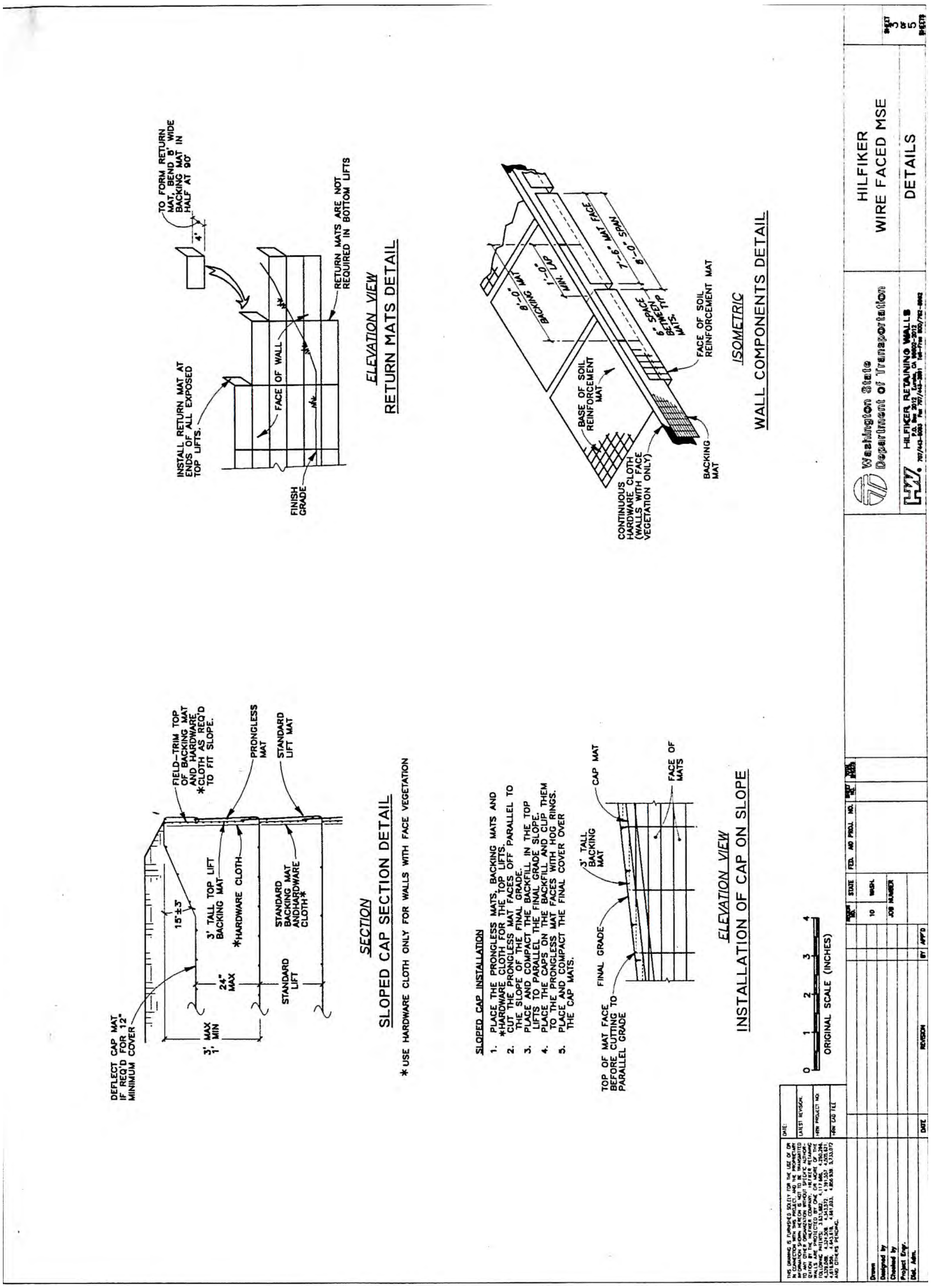
Approved details for the Hilfiker Eureka Reinforced Soil concrete 5 feet × 5 feet panel faced retaining wall system are provided in the following plan sheets. Exceptions and additional requirements regarding the approved details are as follows:

- Regarding the filter fabric shown, WSDOT reserves the right to require the use *Standard Specification* materials as specified in *Standard Specification* Section 9-33 that are similar to those specified in this plan sheet.
- No culvert penetration and obstruction avoidance details for this wall system, as well as traffic barrier details, were provided. However, the obstruction avoidance details, as well as traffic barrier details provided for the Hilfiker welded wire wall system (Chapter 15 App – Hilfiker WW Wall) are acceptable to apply to the Hilfiker Eureka RS Concrete panel Wall, up to a maximum obstruction diameter of 4 feet. This wall system is not preapproved for culvert penetration of the face, as no details for this situation have been provided.


ISOMETRIC
24" LIFT COMPONENTS
SECTION
WALL FACE DETAIL
WITHOUT VEGETATION
SECTION
WALL FACE DETAIL
WITH VEGETATION
SECTIONS
24" MAT CONNECTION DETAILS
BOTTOM MATS
INTERMEDIATE MATS
INTERMEDIATE MATS
ORIGINAL SCALE (INCHES)
HILFIKER 24" LIFT
WIRE FACED MSE
DETAILS
Washington State
Department of Transportation



2"x6"-W2.1xW2.1 WWF BACKING MAT
FACE OF 6"x9"WWF MAT
HARDWARE CLOTH WHERE REQ'D (WITH FACE VEGETATION ONLY)
1'-6" TYPICAL
SECTION
18" MAT ASSEMBLY DETIALS
BACKING MAT PRONGS GO DOWN
PRONG ON SOIL REINFORCEMENT MAT
HOOK BOTTOM OF BACKING MAT OVER TOP PRONGS ON FACE OF LOWER SOIL REINFORCEMENT MAT
STAND BACKING MAT UP AND SECURE TO MAT FACE WITH HOG RINGS
SECURE WITH HOG RING
SECTION
18" MAT CONNECTION DETIALS
SPECIFIED BACKFILL
2-1/2" TO 6" ROCK
CONSTRUCTION GEOTEXTILE FOR UNDERGROUND DRAINAGE, MODERATE SURVIVABILITY, CLASS A PER SECTION 9-33.2.
6"x2"-W2.1xW2.1WWF BACKING MAT
6"x9" WWF SOIL REINFORCEMENT MAT WIRE SIZE PER PROJECT PLANS
2'-0" MIN.
3" MINUMUM 6" MAXIMUM
SECTION
WALL FACE DETAIL WITHOUT VEGETATION
SPECIAL BACKFILL WITH ORGANIC MATERIAL
23 GA. GALV. 1/4" HARDWARE CLOTH
SECTION
WALL FACE DETAIL WITH VEGETATION
6" X 9" WWF CAP
6" X 9" WWF PRONGLESS SOIL REINFORCEMENT MAT (TOP LIFT)
6"x2"-W2.1xW2.1 WWF BACKING MAT TYPICAL
*23 GA GALV. 1/4" HARDWARE CLOTH OR **FILTER FABRIC, CONTINUOUS
6" X 9" WWF SOIL REINFORCEMENT MAT, TYP.
ISOMETRIC
18" LIFT COMPONENTS
* HARDWARE CLOTH IS USED ONLY FOR WALLS WITH FACE VEGETATION
** FILTER FABRIC IS USED ONLY IN TEMPORARY WALLS
ORIGINAL SCALE (INCHES)
Washington State Department of Transportation
HILFIKER RETAINING WALLS
HILFIKER 18" LIFT WIRE FACED MSE
DETAILS



TO FORM RETURN MAT, BEND 8' WIDE BACKING MAT IN HALF AT 90°
INSTALL RETURN MAT AT ENDS OF ALL EXPOSED TOP LIFTS.
FACE OF WALL
FINISH GRADE
RETURN MATS ARE NOT REQUIRED IN BOTTOM LIFTS
ELEVATION VIEW
RETURN MATS DETAIL
BACKING MAT
BASE OF SOIL REINFORCEMENT MAT
FACE OF SOIL REINFORCEMENT MAT
CONTINUOUS HARDWARE CLOTH (WALLS WITH FACE VEGETATION ONLY)
ISOMETRIC
WALL COMPONENTS DETAIL
DEFLECT CAP MAT IF REQ'D FOR 12" MINIMUM COVER
FIELD-TRIM TOP OF BACKING MAT AND HARDWARE *CLOTH AS REQ'D TO FIT SLOPE.
PRONGLESS MAT
STANDARD LIFT MAT
3' TALL TOP LIFT BACKING MAT
*HARDWARE CLOTH
STANDARD BACKING MAT AND HARDWARE CLOTH*
STANDARD LIFT
SECTION
SLOPED CAP SECTION DETAIL
* USE HARDWARE CLOTH ONLY FOR WALLS WITH FACE VEGETATION
SLOPED CAP INSTALLATION
CAP MAT
FACE OF MATS
3' TALL BACKING MAT
FINAL GRADE
TOP OF MAT FACE BEFORE CUTTING TO PARALLEL GRADE
ELEVATION VIEW
INSTALLATION OF CAP ON SLOPE
ORIGINAL SCALE (INCHES)
Washington State Department of Transportation
HILFIKER RETAINING WALLS
HILFIKER WIRE FACED MSE
DETAILS

Appendix 15-J Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply to the design of the Reinforced Earth™ concrete 5 feet × 5 feet panel faced retaining wall:

No HITEC evaluation report is currently available for this wall system. Design procedures for specific elements of the wall system have been provided to WSDOT in a binder dated March 29, 2004. The design procedures used by RECO are based on the AASHTO Standard Specifications for Highway Bridges (2002). Internal stability is based on the use of the Coherent Gravity method per the other widely used and accepted methods clause in the AASHTO Standard Specifications. The Coherent Gravity Method should yield similar results to the AASHTO Simplified Method for this wall system. Interim approval is given for the continued use of the AASHTO

Standard Specifications and the Coherent Gravity Method as the basis for design. Note the connector between the wall face panels and the soil reinforcement strips shall be designed to have adequate life considering corrosion loss as illustrated in the March 29, 2004 binder provided to WSDOT by RECO.

Reinforcement pullout shall be calculated based on the default values for steel grid reinforcement provided in the AASHTO Specifications. If, at some future time product and soil specific pullout data is provided to support use of non-default pullout interaction coefficients, it should be noted that LRFD pullout resistance design using these product and soil specific interaction coefficients has not been calibrated using product specific data statistics and reliability theory. Therefore, the specified resistance factors in the GDM and AASHTO LRFD Specifications should not be considered applicable to product specific pullout interaction coefficients.

Approved details for the Reinforced Earth™ concrete 5 feet × 5 feet panel faced retaining wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- Several plan sheets were submitted that detail panels with dimensions other than 5 feet × 5 feet. The cruciform shaped panels are also considered preapproved for use in WSDOT projects. However, unless otherwise shown in the contract, it should always be assumed that the 5 feet × 5 feet panels are intended for WSDOT projects. Other panel sizes may be used by special design (e.g., full height panels), with the approval of the State Bridge Design Engineer and the State Geotechnical Engineer, provided a complete wall design with detailed plans are developed and included in the construction contract (i.e., walls with larger facing panels shall not be submitted as shop drawings in design-bid-build projects).

- Where filter cloth or geotextile fabric is shown, WSDOT reserves the right to require the use *Standard Specification* materials as specified in *Standard Specification* Section 9-33 that are similar to those specified in this plan sheet.
- Where steel strips are skewed to avoid a backfill obstruction, the maximum skew angle shall be 15 degrees.
- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet. Larger diameter culverts or obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers.

The Reinforced Earth Company

8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175

STANDARD PRECAST PANEL SHOP DRAWINGS – SQUARE PANELS

PANEL GENERAL NOTES

NOTES:

1. REINFORCEMENT SHOWN SHALL COMPLY TO THE ASTM A615, GRADE 60 REBAR LAYOUT WILL BE DETAILED AND SHOWN ON PANEL SHOP DRAWINGS.
2. 1/2" x 1/2" CHAMFER SHALL BE PROVIDED ON ALL EXPOSED EDGES (FRONT FACE ONLY).
3. ALL PANEL TYPES AND OTHER RELATED ELEMENTS WILL BE DETAILED ON SHOP DRAWINGS.
4. PANEL DESIGN STRUCTURAL THICKNESS IS 5 1/2" MINIMUM, THIS THICKNESS MUST INCREASE TO ACCOMMODATE ANY ARCHITECTURAL SCULPTURED FINISH.
5. ACTUAL LOCATION OF REBARS WILL BE ADJUSTED TO ACCOMMODATE PANEL CASTING.
6. PANEL REINFORCEMENT SHALL BE PLACED WITH A MINIMUM 1 3/16" CLEARANCE FROM THE TIE STRIPS. IF MESH REINFORCEMENT IS USED, THE TIE STRIP LOCATION SHALL BE ADJUSTED TO PROVIDE THE MINIMUM REQUIRED CLEARANCE OF 1 3/16".
7. CONCRETE FOR PANELS SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH AFTER 28 DAYS OF 4000 p.s.i.
8. TIE STRIPS SHALL BE STEEL ASTM A570 GRADE 50 (GALVANIZED PER ASTM A-123).
9. VERTICAL REINFORCING BARS SHALL BE PLACED 2" MIN. CLEAR FROM THE BACK FACE OF THE PANEL.
10. ALL REINFORCING BARS SHALL BE STOPPED 2" CLEAR FROM ANY EDGE OF PANEL UNLESS NOTED ON INDIVIDUAL FABRICATION DRAWING.
11. ALL INDIVIDUAL FABRICATION DRAWINGS ARE SHOWN BACK FACE.
12. IN THE CASE OF CUT PANELS AND PANELS WITH HOLES, ADDITIONAL REINFORCEMENT SHOWN ON THE SHOP DRAWINGS SHALL BE PROVIDED ALONG WITH THE MESH REINFORCEMENT.
13. ALL PANELS SHALL HAVE TWO 1-TON LIFTING INSERTS, EXCEPT FOR M AND N PANELS. THE M AND N PANELS SHALL HAVE TWO 2-TON ERECTION HEAD ANCHORS WITH 2-TON SHEAR BARS.

AL-6-R5-4.75'

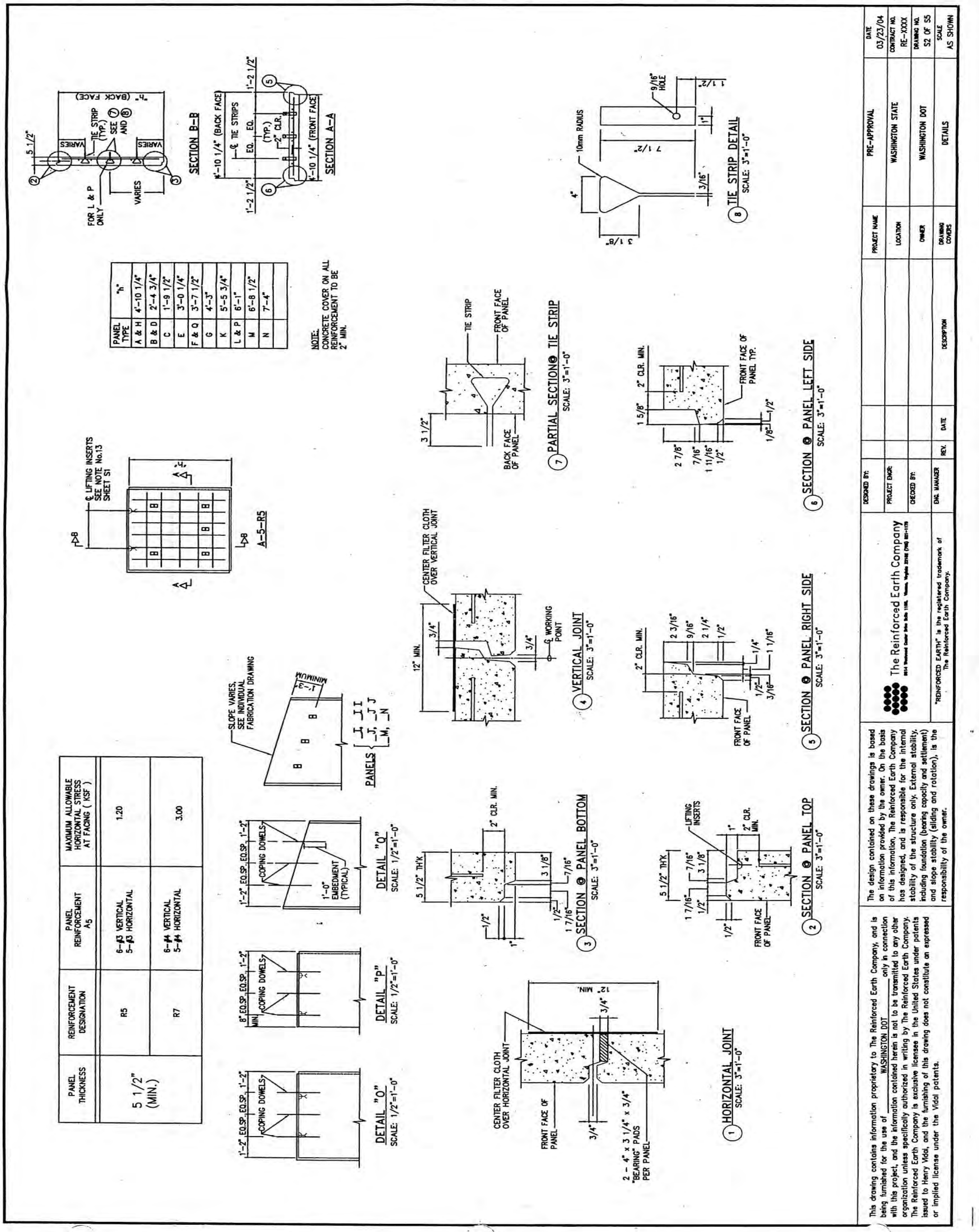
PANEL TYPE
MODIFICATION (ONE OR MORE)
NUMBER OF TIE STRIPS
PANEL REINFORCEMENT TYPE
MODIFICATION DIMENSION (WIDTH OR DIAMETER OR BEND ANGLE)

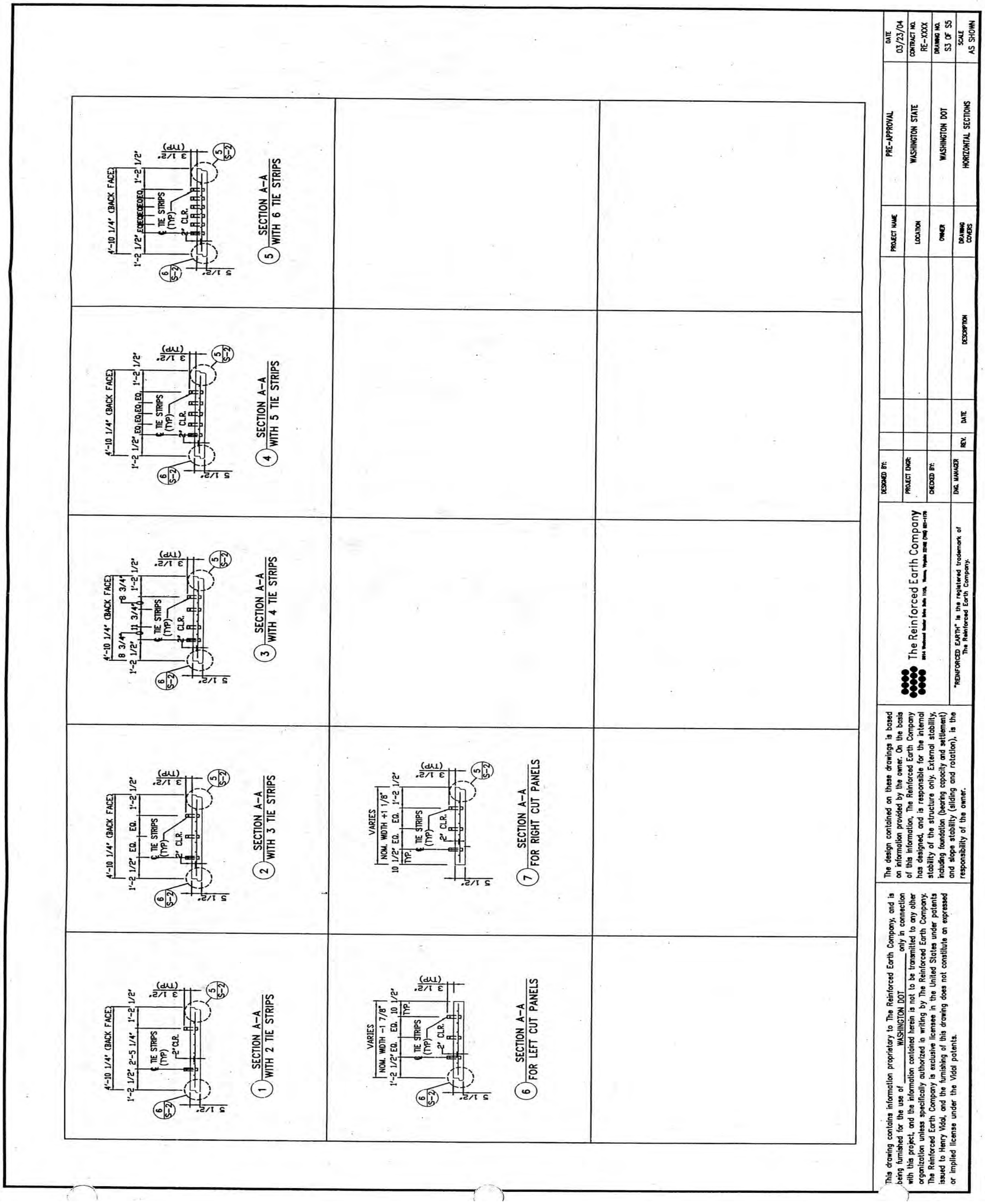
The design contained on these drawings is based on information provided by the owner. On the basis of this information, The Reinforced Earth Company has designed, and is responsible for the internal stability of the structure only. External stability, including foundation (bearing capacity and settlement) and slope stability (sliding and rotation), is the responsibility of the owner.

The Reinforced Earth Company

DESIGNED BY:	PROJECT NAME	PRE-APPROVAL	DATE	03/23/04
PROJECT ENGR:	LOCATION	WASHINGTON STATE	CONTRACT NO.	RE-XXXX
CHECKED BY:	OWNER	WASHINGTON DOT	DRAWING NO.	S1 OF S5
ENG. MANAGER	DRAWING COVERS	GENERAL NOTES	SCALE	AS SHOWN

REV.	DATE	DESCRIPTION


SECTION B-B
SECTION A-A
A-5-R5
PARTIAL SECTION @ TIE STRIP
TIE STRIP DETAIL
SECTION @ PANEL LEFT SIDE
SECTION @ PANEL RIGHT SIDE
VERTICAL JOINT
SECTION @ PANEL BOTTOM
SECTION @ PANEL TOP
HORIZONTAL JOINT
DETAIL "O"
DETAIL "P"
DETAIL "Q"
NOTE: CONCRETE COVER ON ALL REINFORCEMENT TO BE 2" MIN.
The Reinforced Earth Company
PRE-APPROVAL
WASHINGTON STATE
WASHINGTON DOT
DETAILS
03/23/04
RE-XXXX
S2 OF S5
AS SHOWN



SECTION A–A WITH 2 TIE STRIPS
SECTION A–A WITH 3 TIE STRIPS
SECTION A–A WITH 4 TIE STRIPS
SECTION A–A WITH 5 TIE STRIPS
SECTION A–A WITH 6 TIE STRIPS
SECTION A–A FOR LEFT CUT PANELS
SECTION A–A FOR RIGHT CUT PANELS
4'-10 1/4" (BACK FACE)
℄ TIE STRIPS (TYP)
2" CLR.
VARIES
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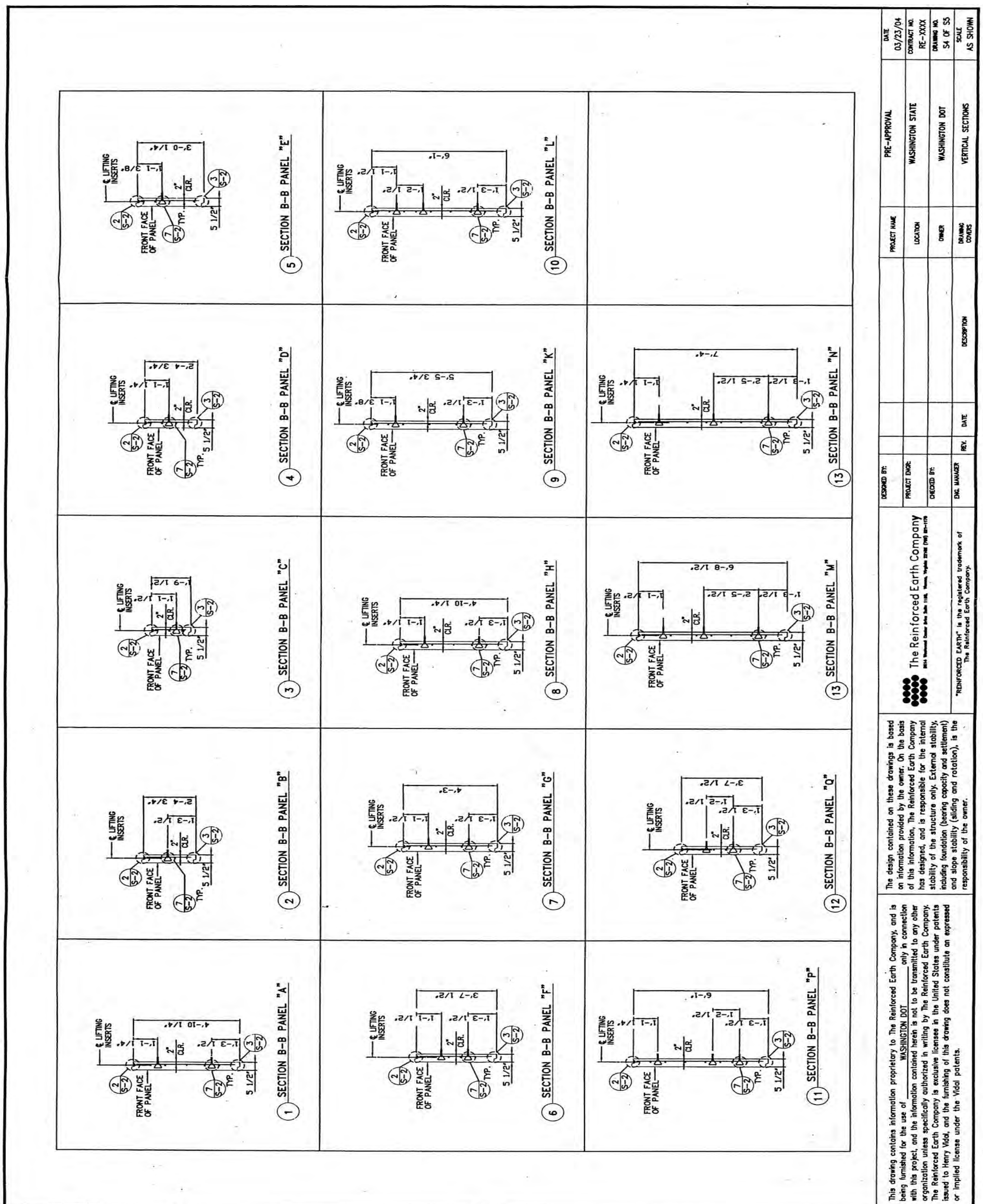
WASHINGTON STATE
WASHINGTON DOT
HORIZONTAL SECTIONS
DATE 03/23/04
CONTRACT NO. RE-XXXX
DRAWING NO. S3 OF S5
SCALE AS SHOWN


SECTION B-B PANEL "A"
SECTION B-B PANEL "B"
SECTION B-B PANEL "C"
SECTION B-B PANEL "D"
SECTION B-B PANEL "E"
SECTION B-B PANEL "F"
SECTION B-B PANEL "G"
SECTION B-B PANEL "H"
SECTION B-B PANEL "K"
SECTION B-B PANEL "L"
SECTION B-B PANEL "P"
SECTION B-B PANEL "Q"
SECTION B-B PANEL "M"
SECTION B-B PANEL "N"
FRONT FACE OF PANEL
℄ LIFTING INSERTS
TYP.
CLR.
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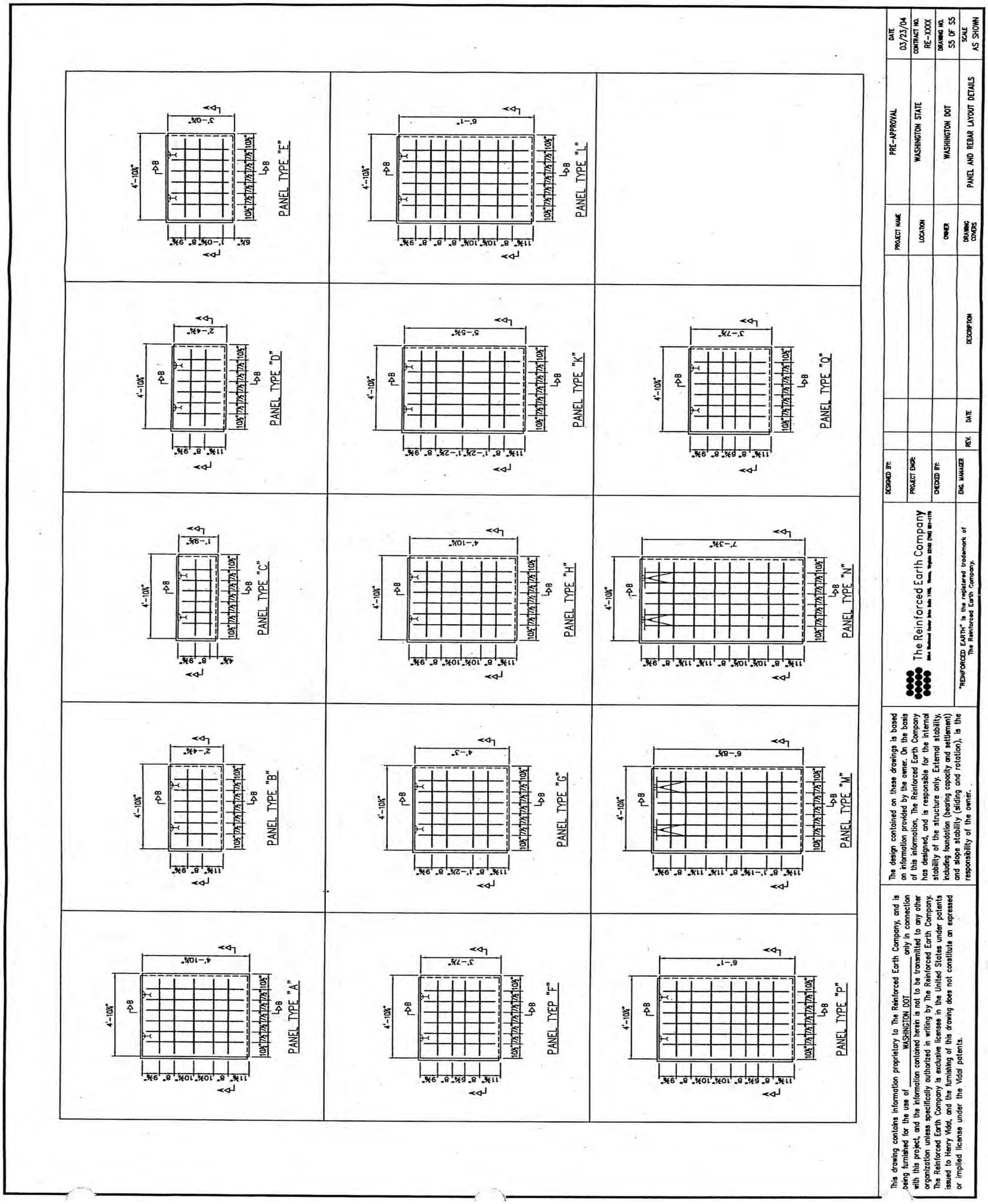
DESIGNED BY:
PROJECT ENGR:
CHECKED BY:
ENG. MANAGER
REV.
DATE
DESCRIPTION
PROJECT NAME
PRE-APPROVAL
LOCATION
WASHINGTON STATE
OWNER
WASHINGTON DOT
DRAWING COVERS
VERTICAL SECTIONS
DATE
03/23/04
CONTRACT NO.
RE-XXXX
DRAWING NO.
S4 OF S5
SCALE
AS SHOWN


PANEL TYPE "A"
PANEL TYPE "B"
PANEL TYPE "C"
PANEL TYPE "D"
PANEL TYPE "E"
PANEL TYEP "F"
PANEL TYPE "G"
PANEL TYPE "H"
PANEL TYPE "K"
PANEL TYPE "L"
PANEL TYPE "M"
PANEL TYPE "N"
PANEL TYPE "P"
PANEL TYPE "Q"
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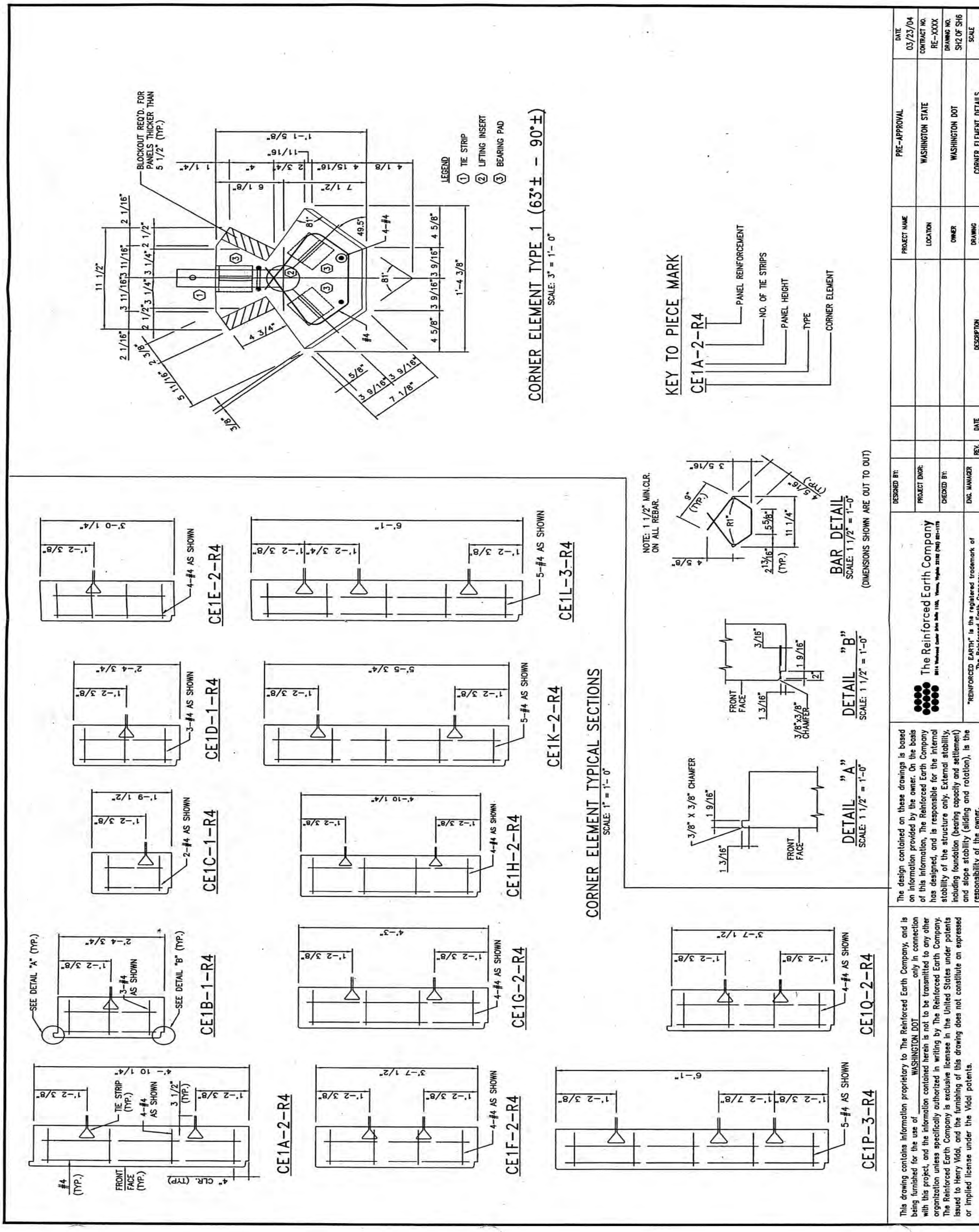
PROJECT ENGR:
CHECKED BY:
ENG. MANAGER
REV.
DATE
DESCRIPTION
PROJECT NAME
PRE-APPROVAL
LOCATION
WASHINGTON STATE
OWNER
WASHINGTON DOT
DRAWING COVERS
PANEL AND REBAR LAYOUT DETAILS
DATE
03/23/04
CONTRACT NO.
RE-XXXX
DRAWING NO.
S5 OF S5
SCALE
AS SHOWN


CORNER ELEMENT TYPE 1 (63°± - 90°±)
SCALE: 3" = 1'- 0"
BLOCKOUT REQ'D. FOR PANELS THICKER THAN 5 1/2" (TYP.)
LEGEND
① TIE STRIP
② LIFTING INSERT
③ BEARING PAD
KEY TO PIECE MARK
CE1A-2-R4
PANEL REINFORCEMENT
NO. OF TIE STRIPS
PANEL HEIGHT
TYPE
CORNER ELEMENT
CORNER ELEMENT TYPICAL SECTIONS
SCALE: 1" = 1'- 0"
CE1A-2-R4
CE1B-1-R4
CE1C-1-R4
CE1D-1-R4
CE1E-2-R4
CE1F-2-R4
CE1G-2-R4
CE1H-2-R4
CE1K-2-R4
CE1L-3-R4
CE1O-2-R4
CE1P-3-R4
SEE DETAIL "A" (TYP.)
SEE DETAIL "B" (TYP.)
TIE STRIP (TYP.)
FRONT FACE (TYP.)
4" CLR. (TYP)
DETAIL "A"
SCALE: 1 1/2" = 1'-0"
DETAIL "B"
SCALE: 1 1/2" = 1'-0"
3/8" X 3/8" CHAMFER
FRONT FACE
BAR DETAIL
SCALE: 1 1/2" = 1'-0"
(DIMENSIONS SHOWN ARE OUT TO OUT)
NOTE: 1 1/2" MIN.CLR. ON ALL REBAR.
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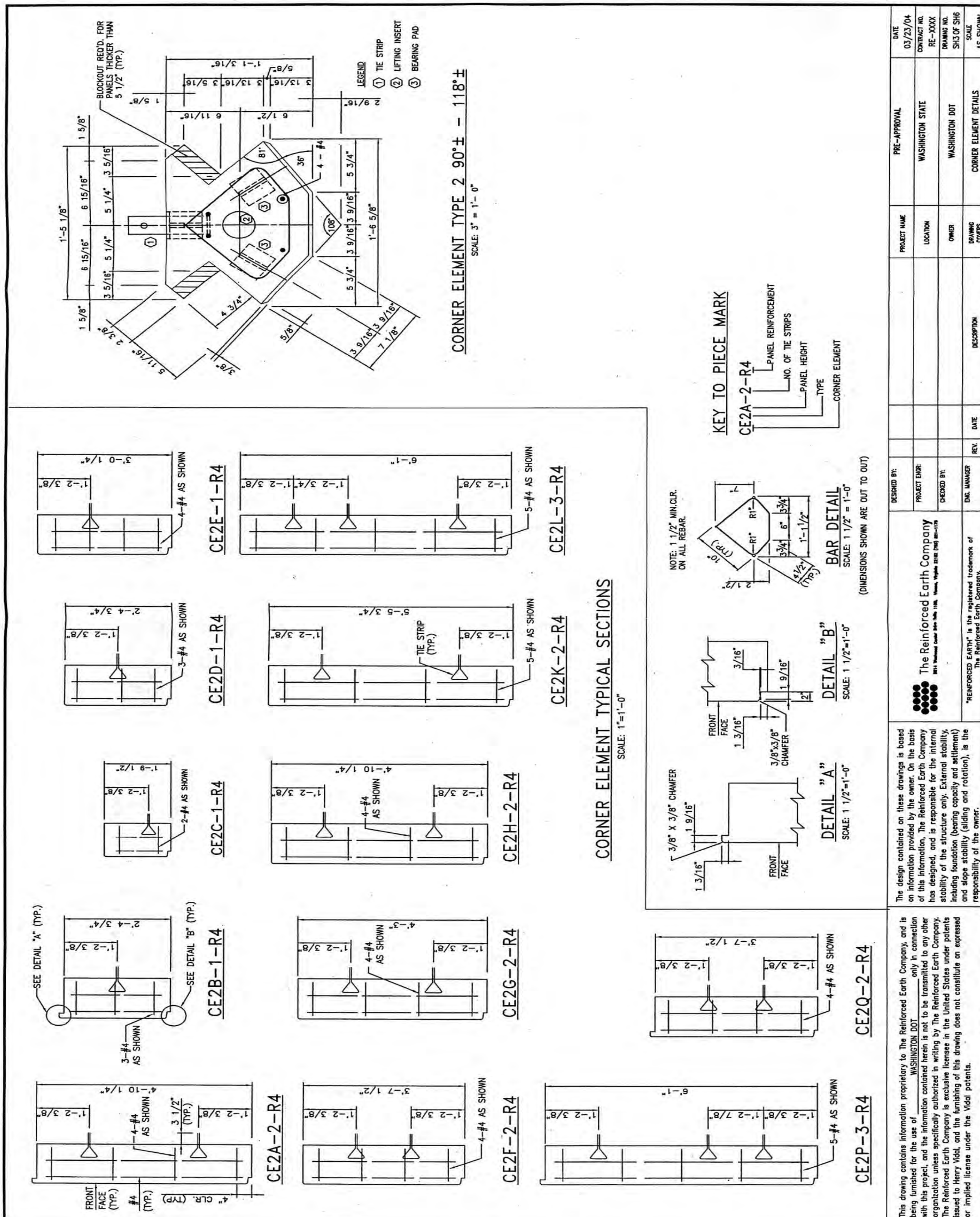
WASHINGTON STATE
WASHINGTON DOT
CORNER ELEMENT DETAILS
DATE 03/23/04
CONTRACT NO. RE-XXXX
DRAWING NO. SH2 OF SH6
SCALE AS SHOWN



CORNER ELEMENT TYPE 2 90°± - 118°±
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LEGEND
1 TIE STRIP
2 LIFTING INSERT
3 BEARING PAD
BLOCKOUT REQ'D. FOR PANELS THICKER THAN 5 1/2" (TYP.)
KEY TO PIECE MARK
CE2A-2-R4
PANEL REINFORCEMENT
NO. OF TIE STRIPS
PANEL HEIGHT
TYPE
CORNER ELEMENT
CORNER ELEMENT TYPICAL SECTIONS
SCALE: 1"=1'-0"
CE2A-2-R4
CE2B-1-R4
CE2C-1-R4
CE2D-1-R4
CE2E-1-R4
CE2F-2-R4
CE2G-2-R4
CE2H-2-R4
CE2K-2-R4
CE2L-3-R4
CE2P-3-R4
CE2Q-2-R4
DETAIL "A"
SCALE: 1 1/2"=1'-0"
DETAIL "B"
SCALE: 1 1/2"=1'-0"
BAR DETAIL
SCALE: 1 1/2" = 1'-0"
(DIMENSIONS SHOWN ARE OUT TO OUT)
NOTE: 1 1/2" MIN.CLR. ON ALL REBAR.
The Reinforced Earth Company
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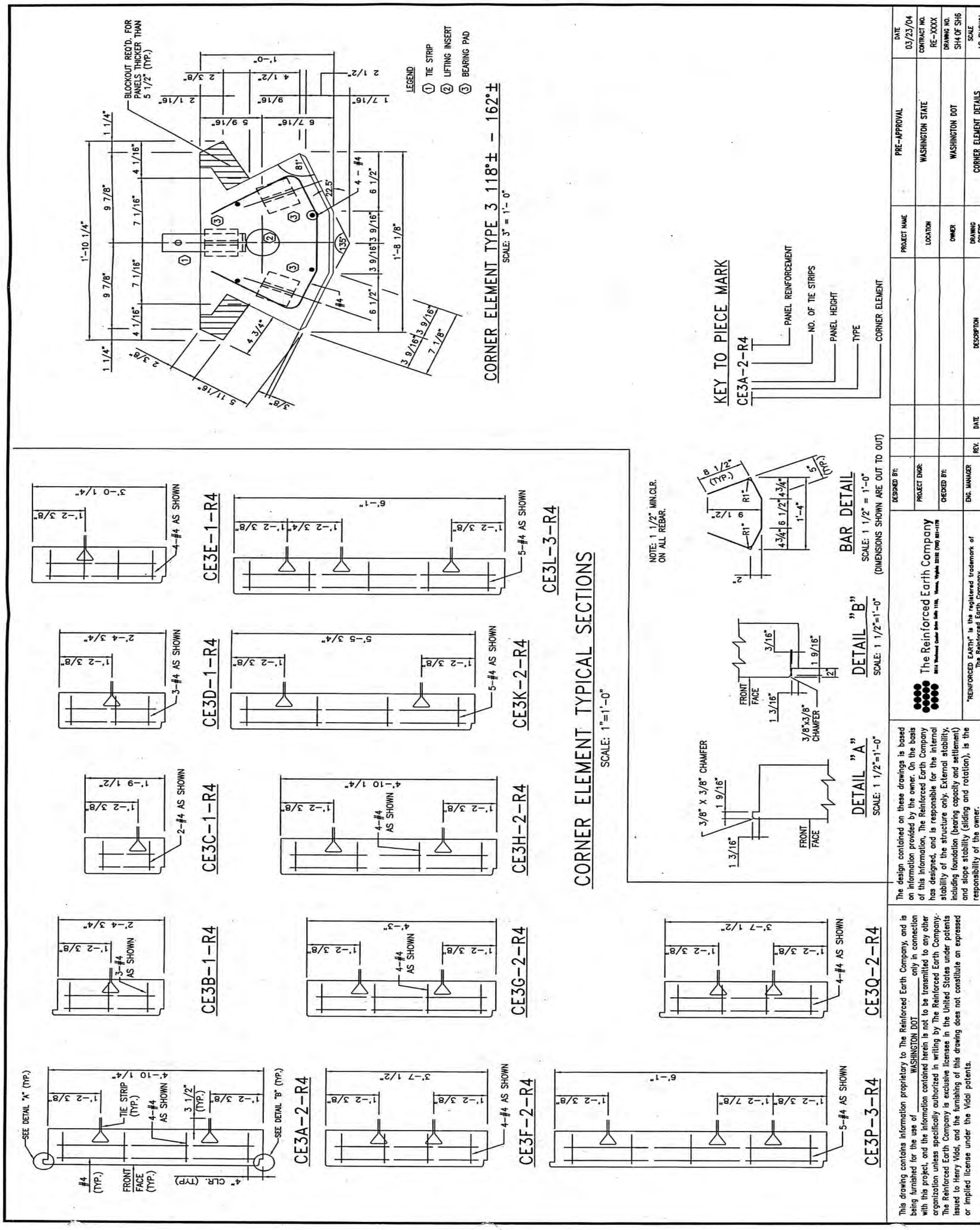
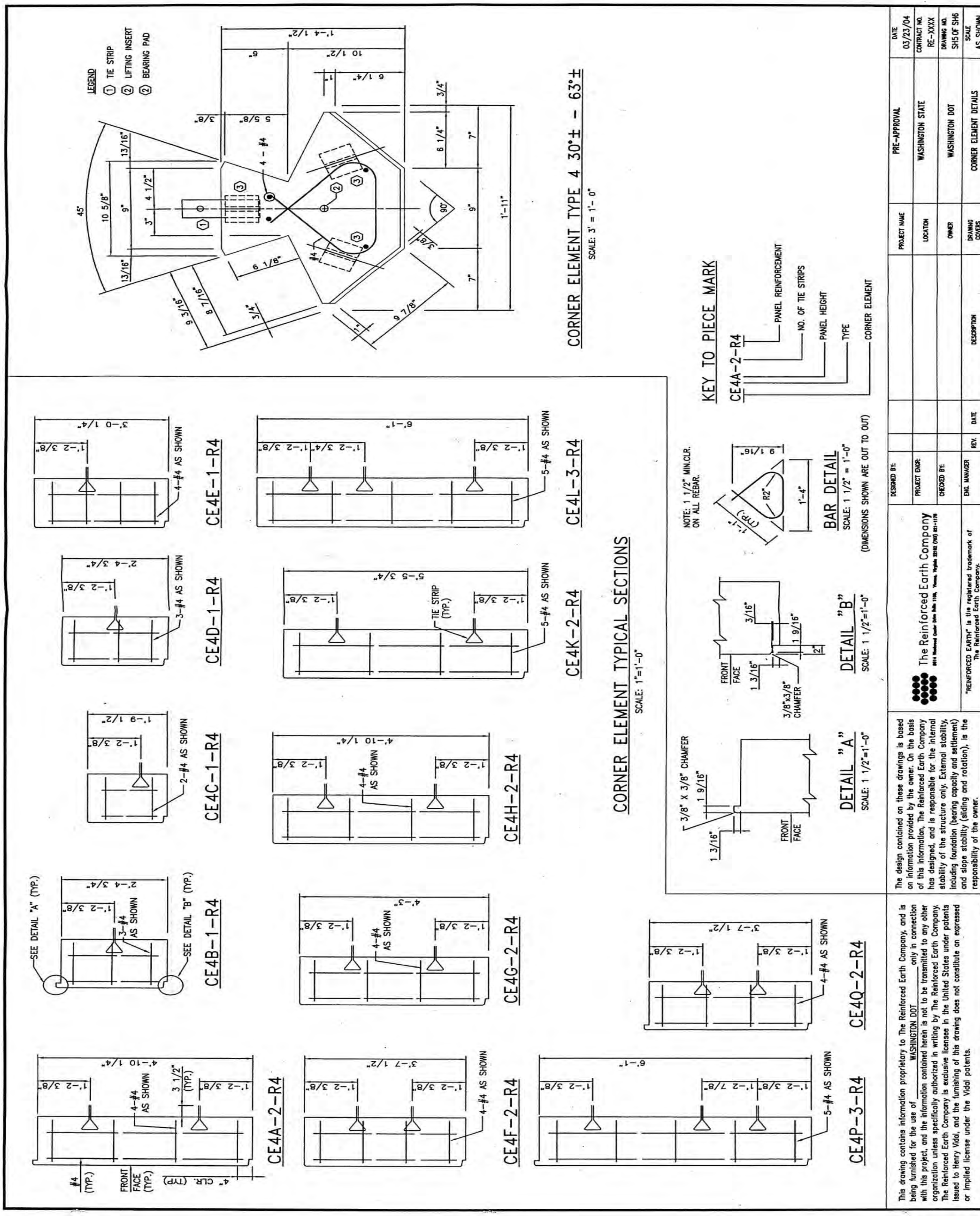
WASHINGTON STATE
WASHINGTON DOT
CORNER ELEMENT DETAILS
DATE 03/23/04
CONTRACT NO. RE-XXXX
DRAWING NO. SH3 OF SH6
SCALE AS SHOWN


CORNER ELEMENT TYPE 3 118°± – 162°±
SCALE: 3" = 1'-0"
LEGEND
1 TIE STRIP
2 LIFTING INSERT
3 BEARING PAD
BLOCKOUT REQ'D. FOR PANELS THICKER THAN 5 1/2" (TYP.)
KEY TO PIECE MARK
CE3A-2-R4
PANEL REINFORCEMENT
NO. OF TIE STRIPS
PANEL HEIGHT
TYPE
CORNER ELEMENT
CORNER ELEMENT TYPICAL SECTIONS
SCALE: 1"=1'-0"
CE3A-2-R4
CE3B-1-R4
CE3C-1-R4
CE3D-1-R4
CE3E-1-R4
CE3F-2-R4
CE3G-2-R4
CE3H-2-R4
CE3K-2-R4
CE3L-3-R4
CE3P-3-R4
CE3Q-2-R4
DETAIL "A"
SCALE: 1 1/2"=1'-0"
DETAIL "B"
SCALE: 1 1/2"=1'-0"
BAR DETAIL
SCALE: 1 1/2" = 1'-0"
(DIMENSIONS SHOWN ARE OUT TO OUT)
NOTE: 1 1/2" MIN.CLR. ON ALL REBAR.
The Reinforced Earth Company
PRE-APPROVAL
WASHINGTON STATE
WASHINGTON DOT
CORNER ELEMENT DETAILS
DATE 03/23/04
CONTRACT NO. RE-XXXX
DRAWING NO. SH4 OF SH6
SCALE AS SHOWN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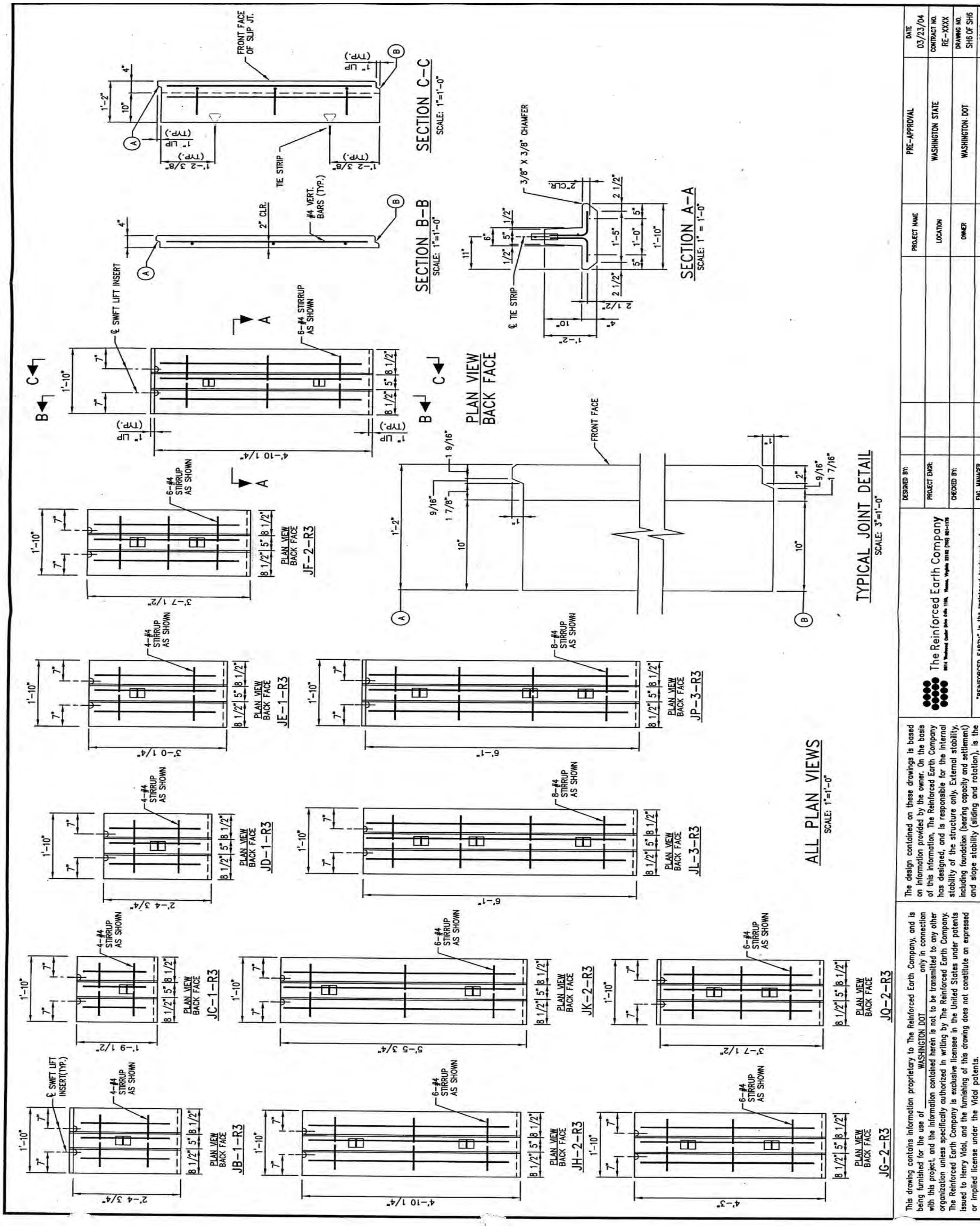
SCALE: 3" = 1'- 0"
LEGEND
① TIE STRIP
② LIFTING INSERT
③ BEARING PAD
KEY TO PIECE MARK
CE4A-2-R4
PANEL REINFORCEMENT
NO. OF TIE STRIPS
PANEL HEIGHT
TYPE
CORNER ELEMENT
CORNER ELEMENT TYPICAL SECTIONS
SCALE: 1"=1'-0"
CE4A-2-R4
CE4B-1-R4
CE4C-1-R4
CE4D-1-R4
CE4E-1-R4
CE4F-2-R4
CE4G-2-R4
CE4H-2-R4
CE4K-2-R4
CE4L-3-R4
CE4P-3-R4
CE4Q-2-R4
DETAIL "A"
SCALE: 1 1/2"=1'-0"
DETAIL "B"
SCALE: 1 1/2"=1'-0"
BAR DETAIL
SCALE: 1 1/2" = 1'-0"
NOTE: 1 1/2" MIN.CLR. ON ALL REBAR.
(DIMENSIONS SHOWN ARE OUT TO OUT)
The Reinforced Earth Company
PRE-APPROVAL
WASHINGTON STATE
WASHINGTON DOT
CORNER ELEMENT DETAILS
03/23/04
RE-XXXX
SH5 OF SH6
AS SHOWN



SECTION C-C
SCALE: 1"=1'-0"
SECTION B-B
SCALE: 1"=1'-0"
SECTION A-A
SCALE: 1" = 1'-0"
PLAN VIEW BACK FACE
TYPICAL JOINT DETAIL
SCALE: 3"=1'-0"
ALL PLAN VIEWS
SCALE: 1"=1'-0"
JB-1-R3
JH-2-R3
JG-2-R3
JC-1-R3
JK-2-R3
JQ-2-R3
JD-1-R3
JL-3-R3
JE-1-R3
JP-3-R3
JF-2-R3
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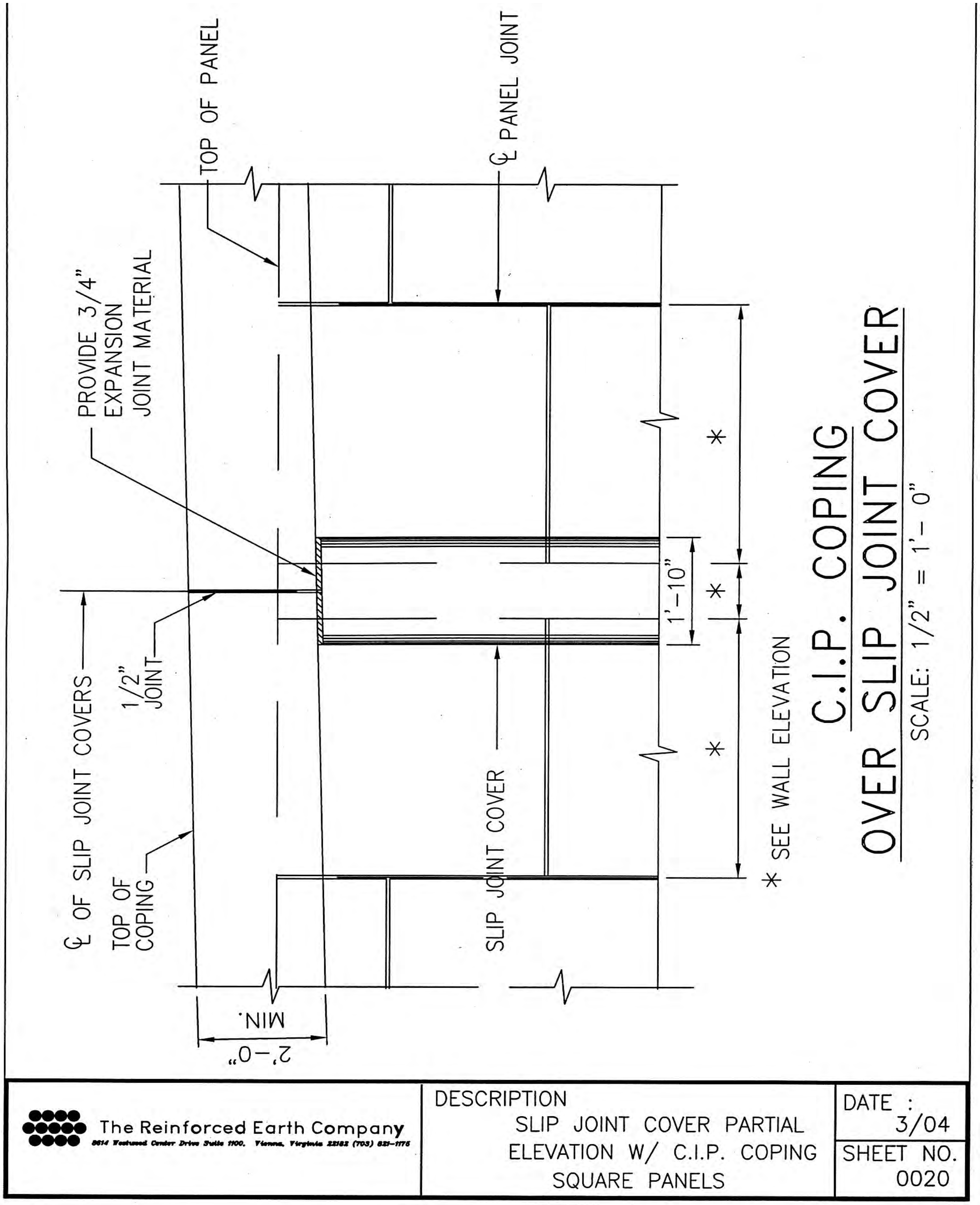
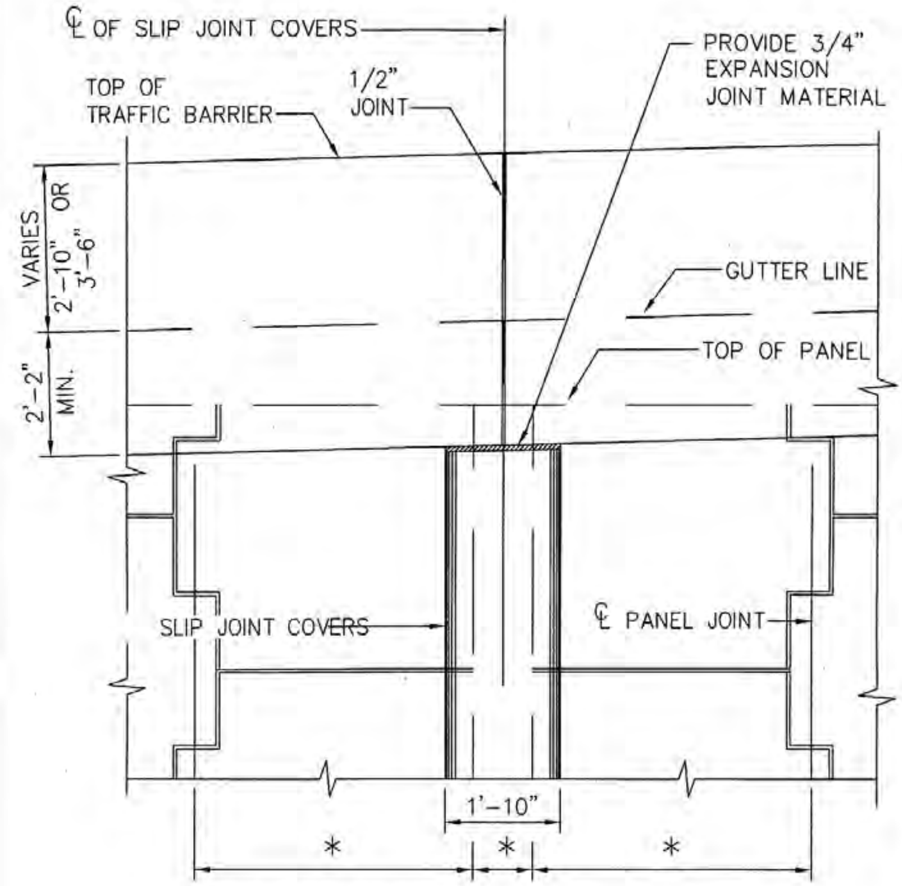
PRE-APPROVAL
WASHINGTON STATE
WASHINGTON DOT
SLIP JOINT DETAILS
03/23/04
RE-XXXX
SH6 OF SH6
AS SHOWN


TOP OF PANEL
℄ PANEL JOINT
PROVIDE 3/4" EXPANSION JOINT MATERIAL
℄ OF SLIP JOINT COVERS
1/2" JOINT
TOP OF COPING
SLIP JOINT COVER
1'-10"
2'-0" MIN.
* SEE WALL ELEVATION
C.I.P. COPING OVER SLIP JOINT COVER
SCALE: 1/2" = 1'- 0"
The Reinforced Earth Company
DESCRIPTION
SLIP JOINT COVER PARTIAL ELEVATION W/ C.I.P. COPING SQUARE PANELS
DATE : 3/04
SHEET NO. 0020

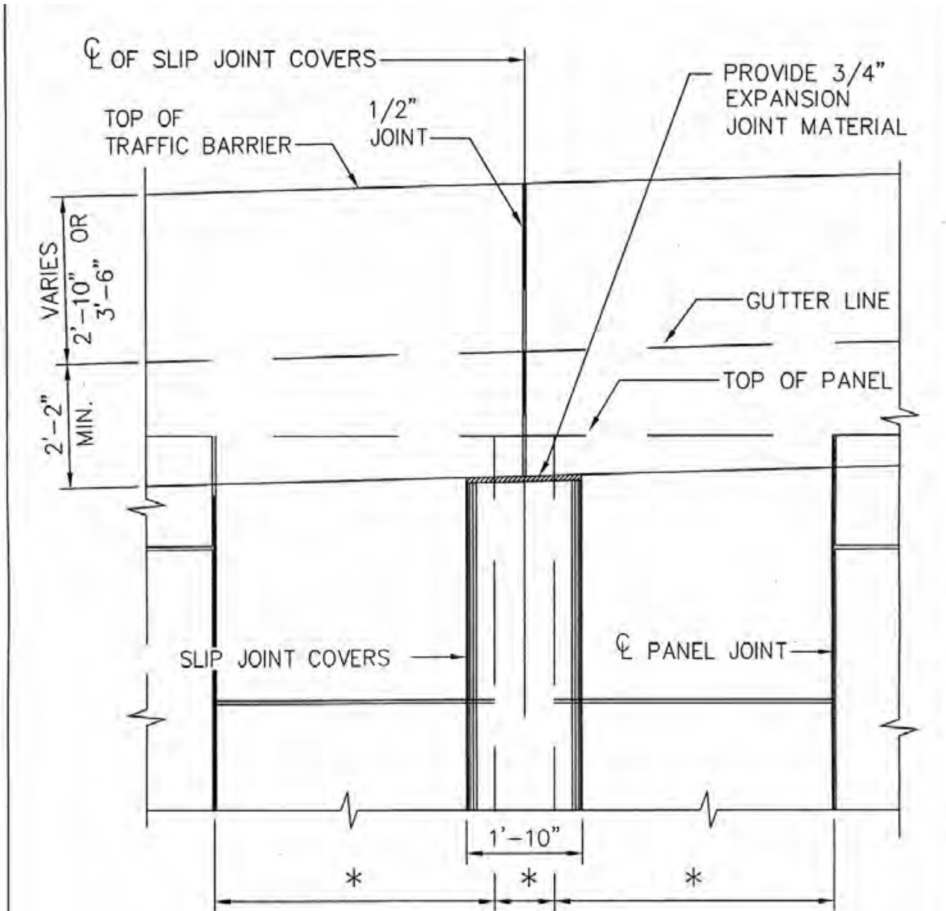


* SEE WALL ELEVATION

C.I.P. TRAFFIC BARRIER OVER SLIP JOINT COVER

SCALE: 1/2" = 1'- 0"

The Reinforced Earth Company	DESCRIPTION SLIP JOINT COVER PARTIAL ELEVATION W/ BARRIER CRUCIFORM PANELS	DATE : 3/04 SHEET NO. 0021

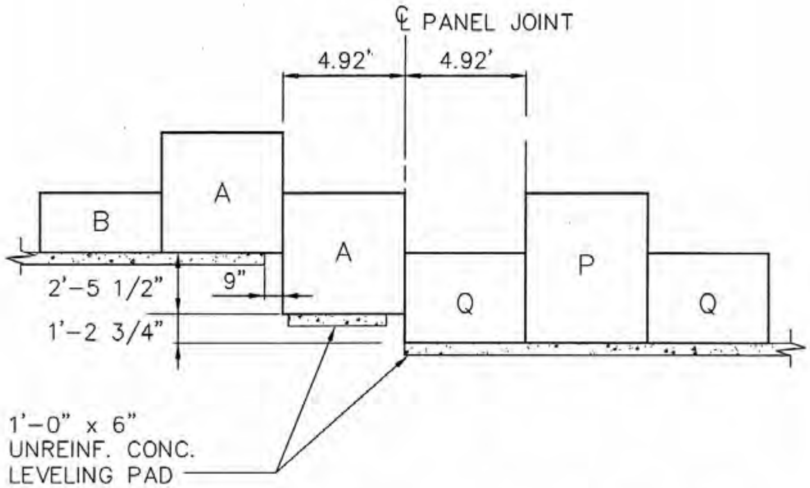


* SEE WALL ELEVATION

C.I.P. TRAFFIC BARRIER OVER SLIP JOINT COVER

SCALE: 1/2" = 1'- 0"

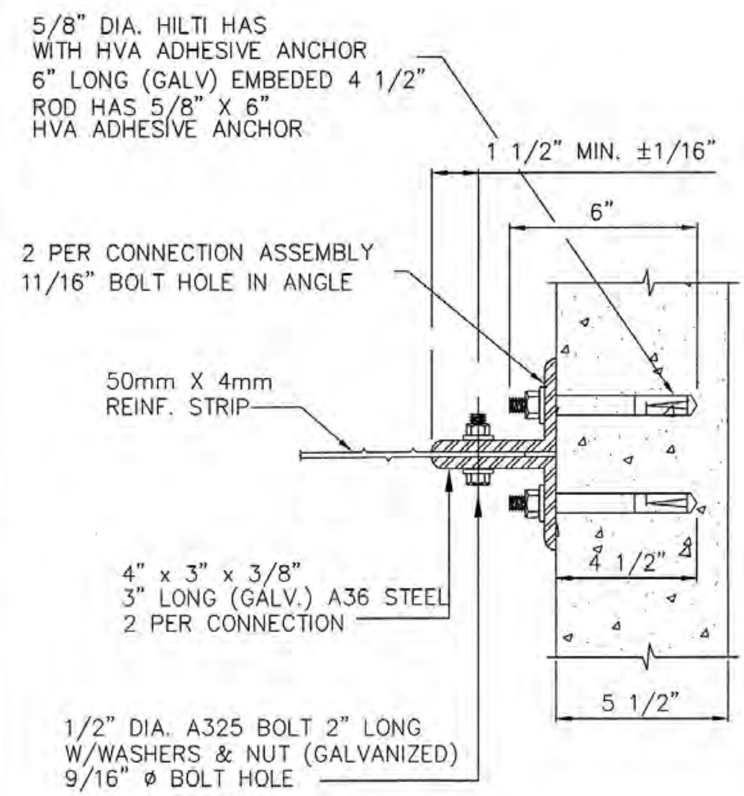
The Reinforced Earth Company	DESCRIPTION SLIP JOINT COVER PARTIAL ELEVATION W/ BARRIER SQUARE PANELS	DATE : 3/04 SHEET NO. 0022



TYPICAL LEVELING PAD STEP DETAIL

NOT TO SCALE

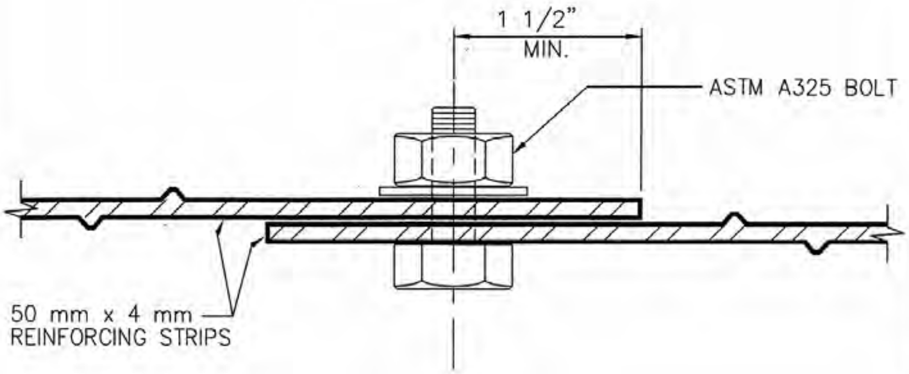
The Reinforced Earth Company	DESCRIPTION SQUARE PANELS – LEVELING PAD STEP DETAIL	DATE : 3/04 SHEET NO. 0024



CLIP ANGLE DETAIL

SCALE: 3" = 1'-0"

The Reinforced Earth Company	DESCRIPTION CLIP ANGLE DETAIL	DATE : 3/04
		SHEET NO. 0025



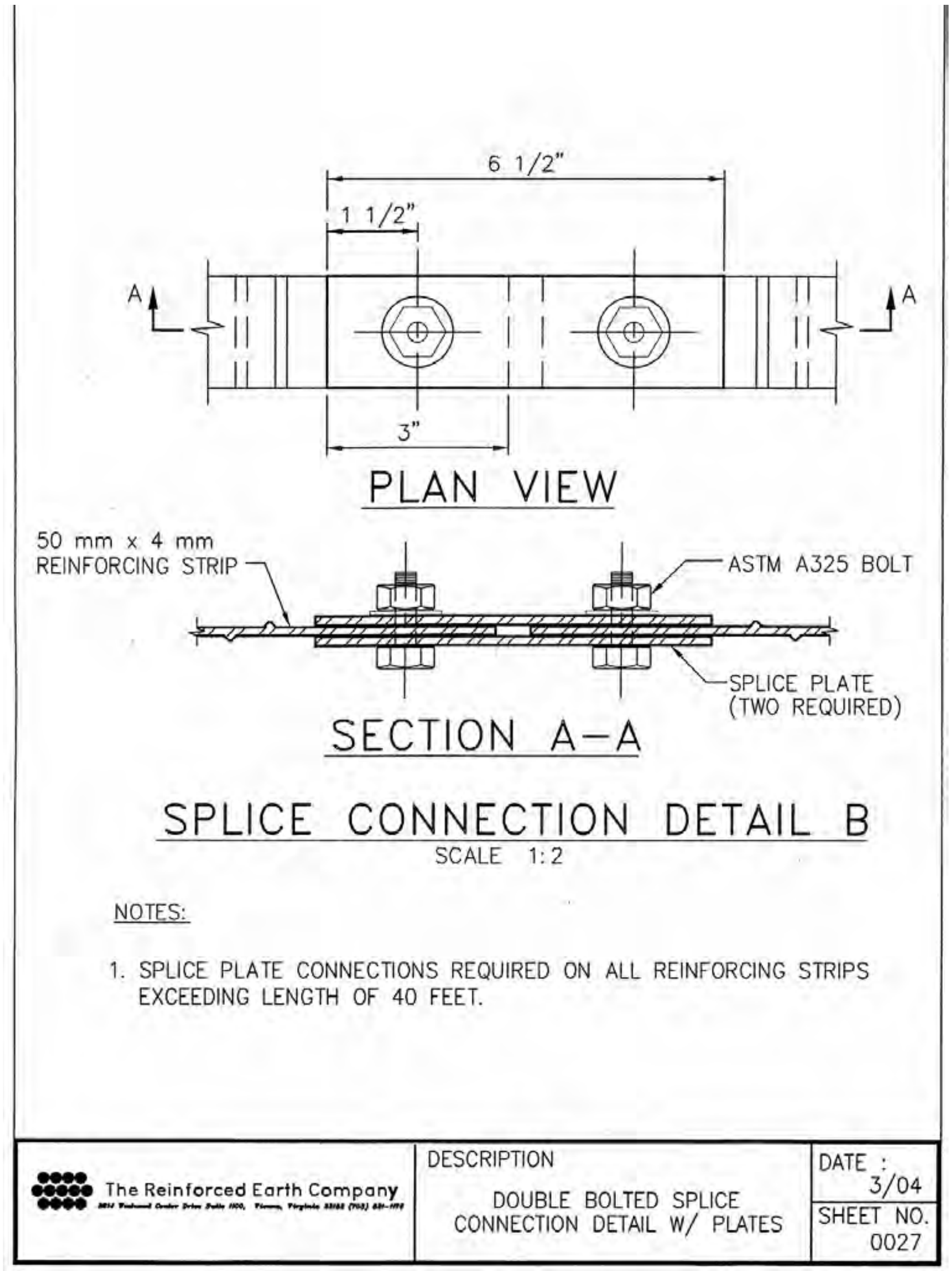
SPLICE CONNECTION DETAIL A

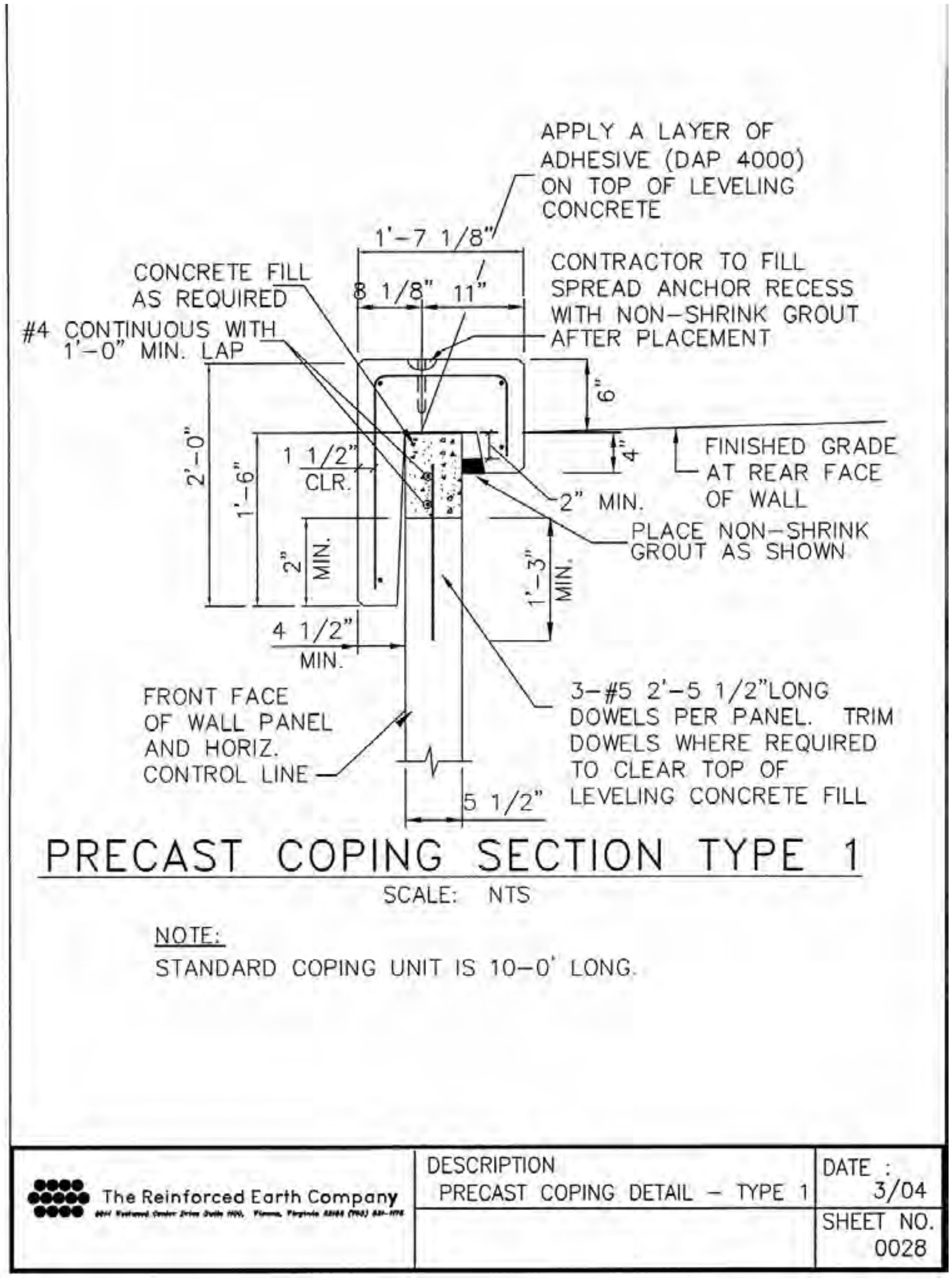
SCALE NTS

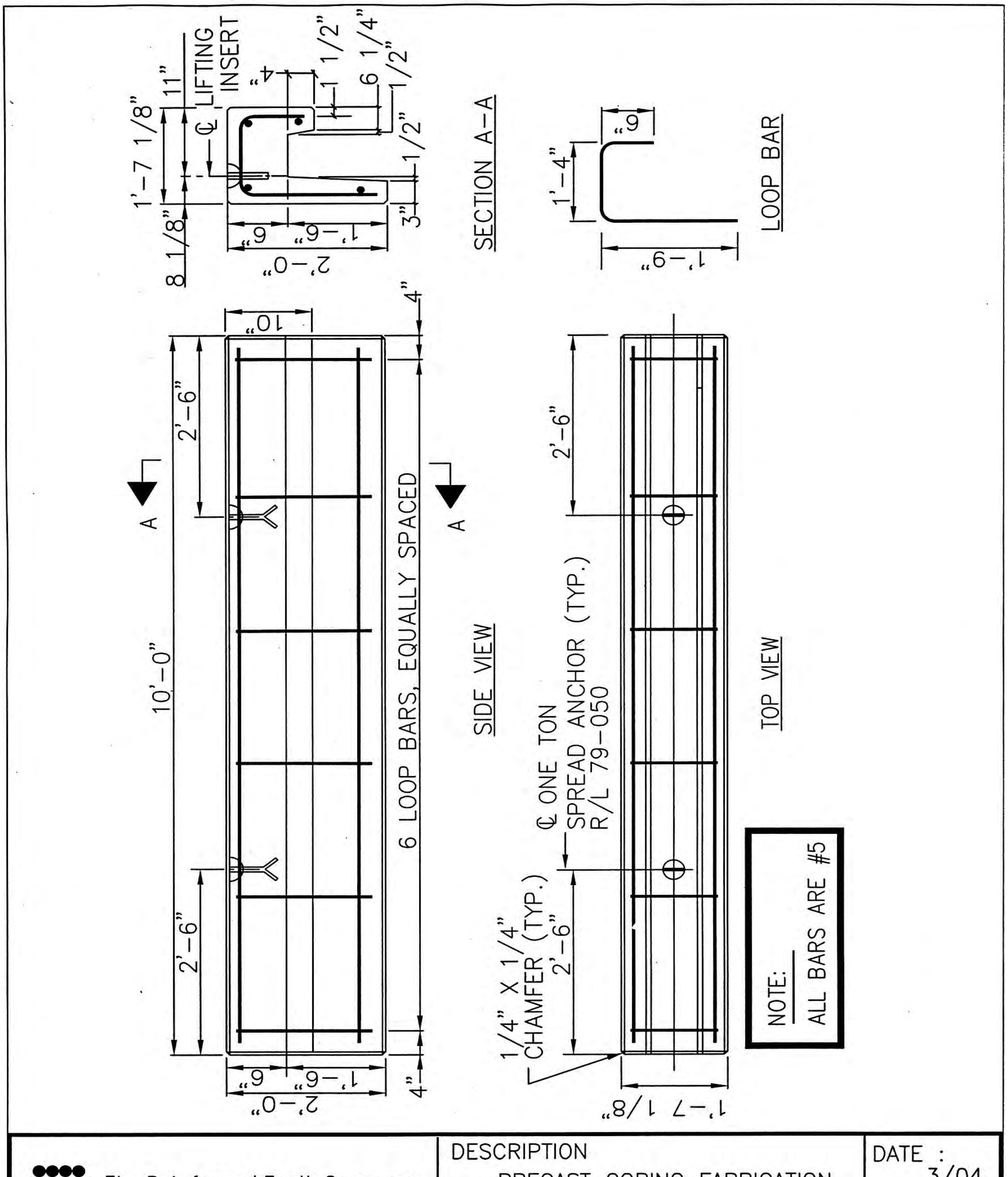
NOTES:

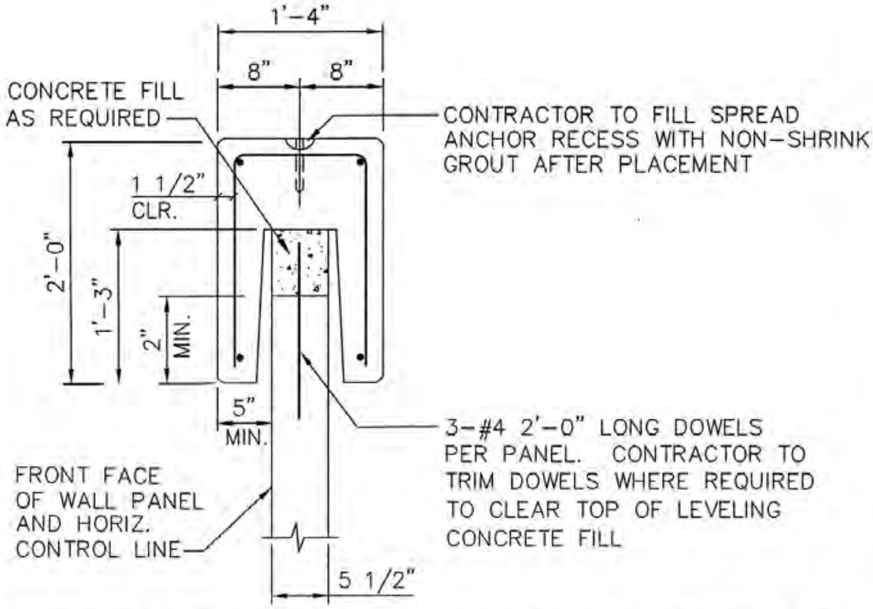
1. SPLICE PLATE CONNECTIONS REQUIRED ON ALL REINFORCING STRIPS BETWEEN LENGTH OF 32 FEET AND 40 FEET .

The Reinforced Earth Company	DESCRIPTION SINGLE BOLT SPLICE CONNECTION DETAIL	DATE : 3/04
		SHEET NO. 0026


6 1/2"
1 1/2"
A
A
3"
PLAN VIEW
50 mm x 4 mm
REINFORCING STRIP
ASTM A325 BOLT
SPLICE PLATE
(TWO REQUIRED)
SECTION A–A
SPLICE CONNECTION DETAIL B
SCALE 1:2
NOTES:
1. SPLICE PLATE CONNECTIONS REQUIRED ON ALL REINFORCING STRIPS EXCEEDING LENGTH OF 40 FEET.
The Reinforced Earth Company
DESCRIPTION
DOUBLE BOLTED SPLICE
CONNECTION DETAIL W/ PLATES
DATE :
3/04
SHEET NO.
0027


APPLY A LAYER OF ADHESIVE (DAP 4000) ON TOP OF LEVELING CONCRETE
1'-7 1/8"
CONCRETE FILL AS REQUIRED
8 1/8"
11"
CONTRACTOR TO FILL SPREAD ANCHOR RECESS WITH NON-SHRINK GROUT AFTER PLACEMENT
#4 CONTINUOUS WITH 1'-0" MIN. LAP
6"
2'-0"
1'-6"
1 1/2" CLR.
4"
FINISHED GRADE AT REAR FACE OF WALL
2" MIN.
PLACE NON-SHRINK GROUT AS SHOWN
2" MIN.
1'-3" MIN.
4 1/2" MIN.
FRONT FACE OF WALL PANEL AND HORIZ. CONTROL LINE
3-#5 2'-5 1/2" LONG DOWELS PER PANEL. TRIM DOWELS WHERE REQUIRED TO CLEAR TOP OF LEVELING CONCRETE FILL
5 1/2"
PRECAST COPING SECTION TYPE 1
SCALE: NTS
NOTE:
STANDARD COPING UNIT IS 10-0' LONG.
The Reinforced Earth Company
DESCRIPTION
PRECAST COPING DETAIL – TYPE 1
DATE : 3/04
SHEET NO. 0028


SECTION A–A
LOOP BAR
SIDE VIEW
TOP VIEW
6 LOOP BARS, EQUALLY SPACED
℄ LIFTING INSERT
℄ ONE TON SPREAD ANCHOR (TYP.) R/L 79–050
1/4" X 1/4" CHAMFER (TYP.)
NOTE:
ALL BARS ARE #5
10'–0"
2'–6"
2'–0"
1'–6"
6"
4"
10"
1'–7 1/8"
8 1/8"
11"
1'–4"
1'–9"
The Reinforced Earth Company
DESCRIPTION
PRECAST COPING FABRICATION DRAWING – TYPE 1
DATE : 3/04
SHEET NO. 0029



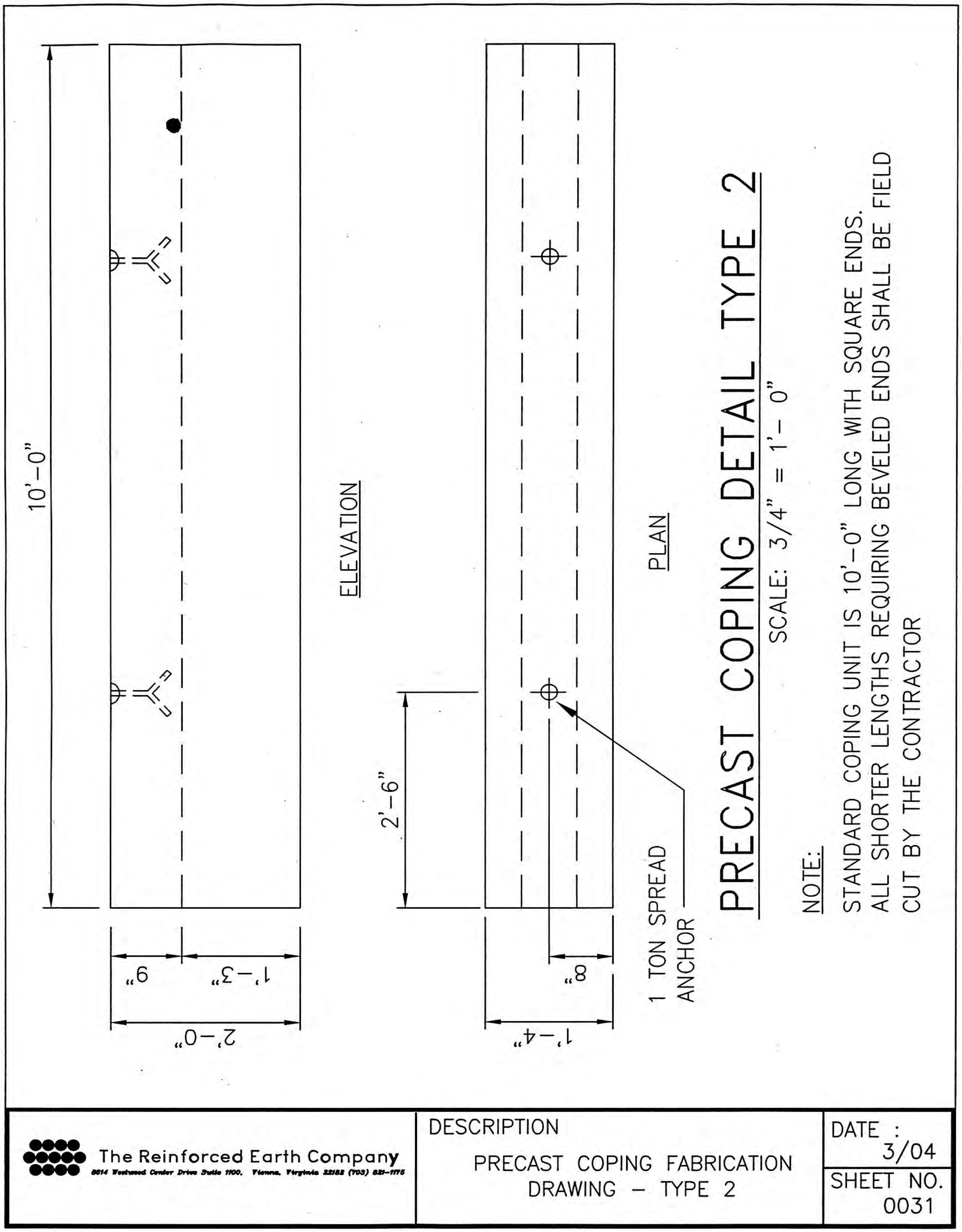
PRECAST COPING SECTION TYPE 2

SCALE: 1" = 1'- 0"

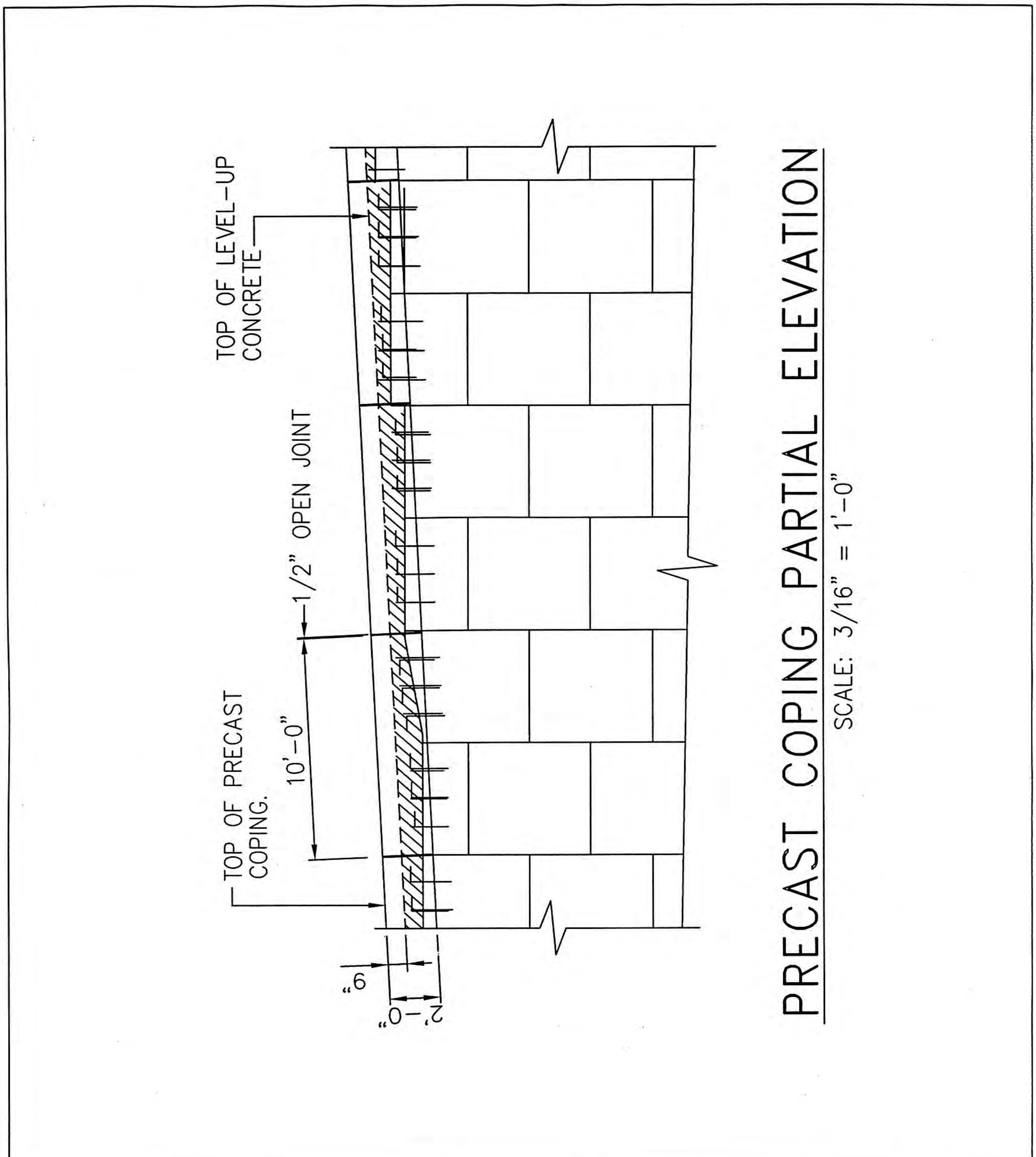
NOTE:

STANDARD COPING UNIT IS 10'-0" LONG WITH SQUARE ENDS.

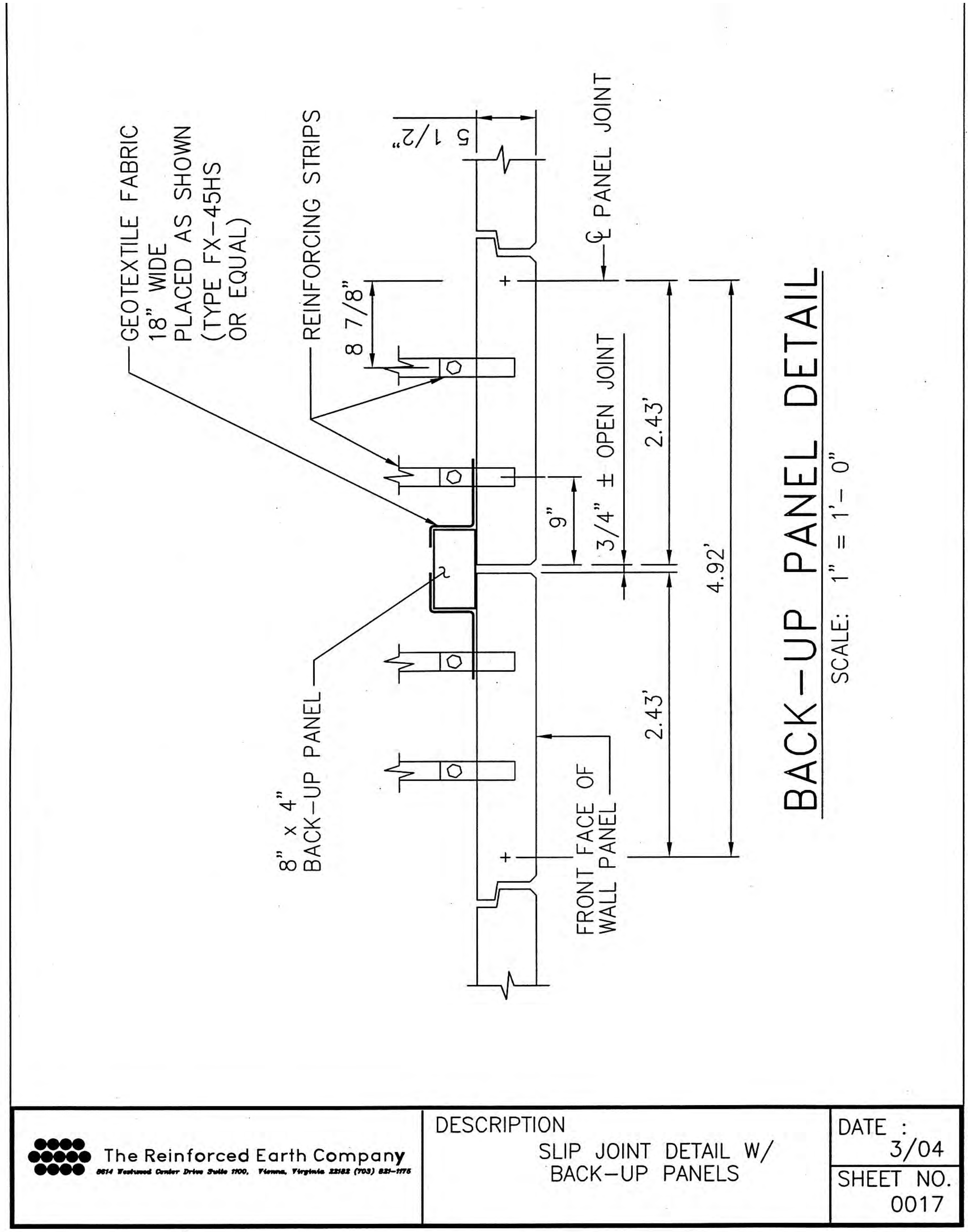
The Reinforced Earth Company	DESCRIPTION PRECAST COPING DETAIL – TYPE 2	DATE : 3/04
		SHEET NO. 0030

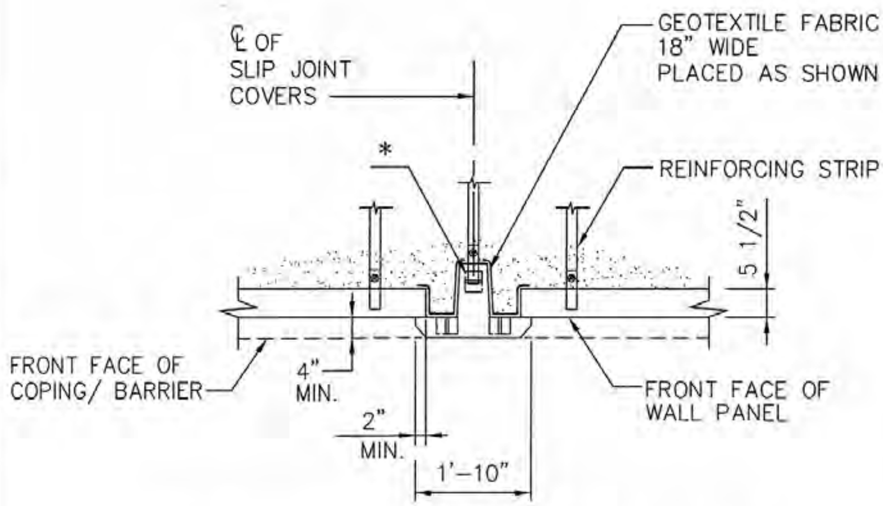


10'–0"
ELEVATION
9"
1'–3"
2'–0"
2'–6"
PLAN
8"
1'–4"
1 TON SPREAD ANCHOR
PRECAST COPING DETAIL TYPE 2
SCALE: 3/4" = 1'– 0"
NOTE:
STANDARD COPING UNIT IS 10'–0" LONG WITH SQUARE ENDS.
ALL SHORTER LENGTHS REQUIRING BEVELED ENDS SHALL BE FIELD
CUT BY THE CONTRACTOR
The Reinforced Earth Company
DESCRIPTION
PRECAST COPING FABRICATION DRAWING – TYPE 2
DATE : 3/04
SHEET NO. 0031



The Reinforced Earth Company 8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175	DESCRIPTION PRECAST COPING PARTIAL ELEVATION	DATE : 3/04
	SQUARE PANELS	SHEET NO. 0033


GEOTEXTILE FABRIC
18" WIDE
PLACED AS SHOWN
(TYPE FX-45HS
OR EQUAL)
REINFORCING STRIPS
5 1/2"
℄ PANEL JOINT
8 7/8"
9"
3/4" ± OPEN JOINT
2.43'
4.92'
2.43'
8" x 4"
BACK-UP PANEL
FRONT FACE OF
WALL PANEL
BACK-UP PANEL DETAIL
SCALE: 1" = 1'- 0"
The Reinforced Earth Company
DESCRIPTION
SLIP JOINT DETAIL W/
BACK-UP PANELS
DATE :
3/04
SHEET NO.
0017

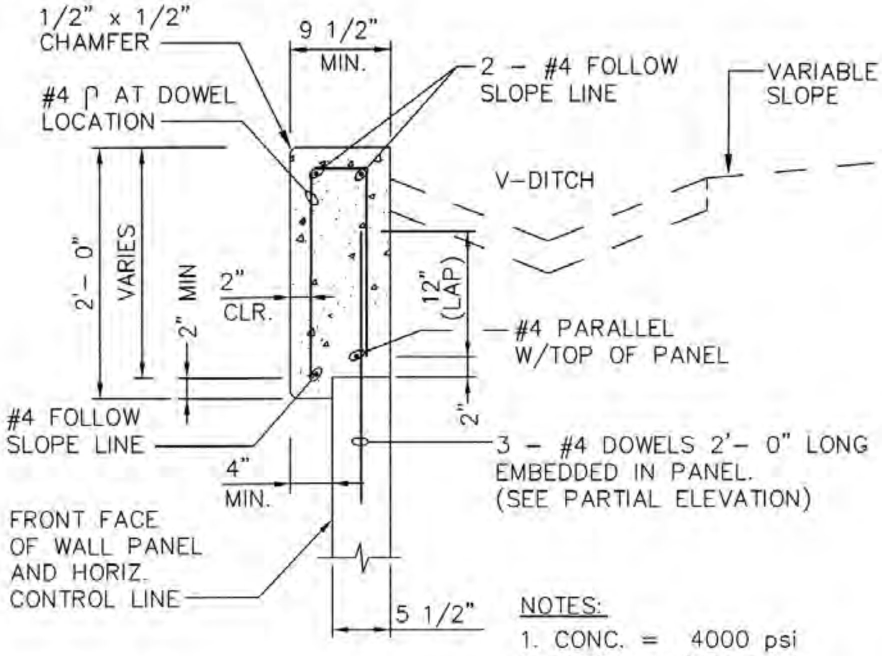


SLIP JOINT COVER DETAIL

SCALE: 1/2" = 1'– 0"

* THREE BEARING PADS PER UNIT, BASE STEM OF BEARING PAD SHALL BE FIELD CUT TO FIT FLAT ON TOP OF CORNER ELEMENT. FRONT PADS SHALL BE PLACED ON INSIDE EDGE OF LIP.

The Reinforced Earth Company	DESCRIPTION SLIP JOINT COVER DETAIL	DATE : 3/04
		SHEET NO. 0018



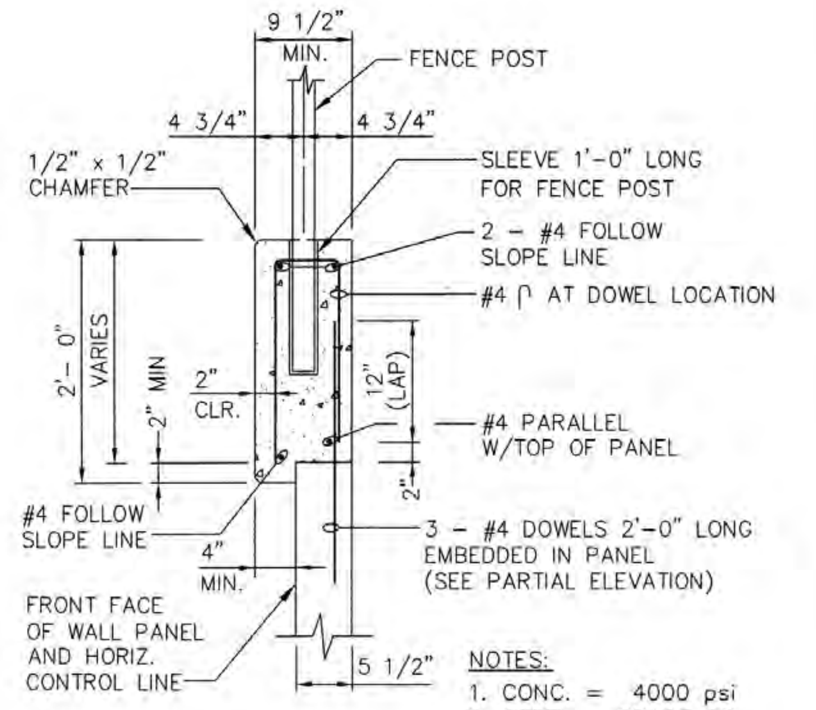
NOTES:

1. CONC. = 4000 psi
2. STEEL = GRADE 60
3. ALL LONGITUDINAL BARS ARE #4
4. EXPANSION JOINTS (1/2") EVERY 2 PANEL UNITS

C.I.P. CONC. COPING W/DITCH

SCALE: 1" = 1'– 0"

The Reinforced Earth Company	DESCRIPTION C.I.P. COPING W/ DITCH DETAIL	DATE : 3/04
		SHEET NO. 0010



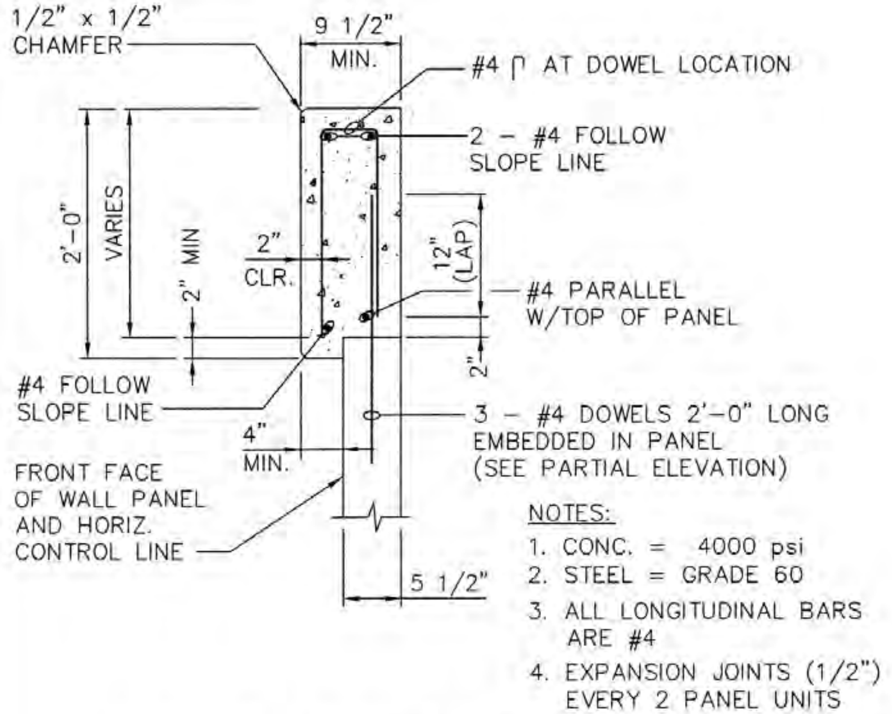
NOTES:

1. CONC. = 4000 psi
2. STEEL = GRADE 60
3. ALL LONGITUDINAL BARS ARE #4
4. EXPANSION JOINTS (1/2") EVERY 2 PANEL UNITS

C.I.P. CONC. COPING W/FENCE

SCALE: 1" = 1'- 0"

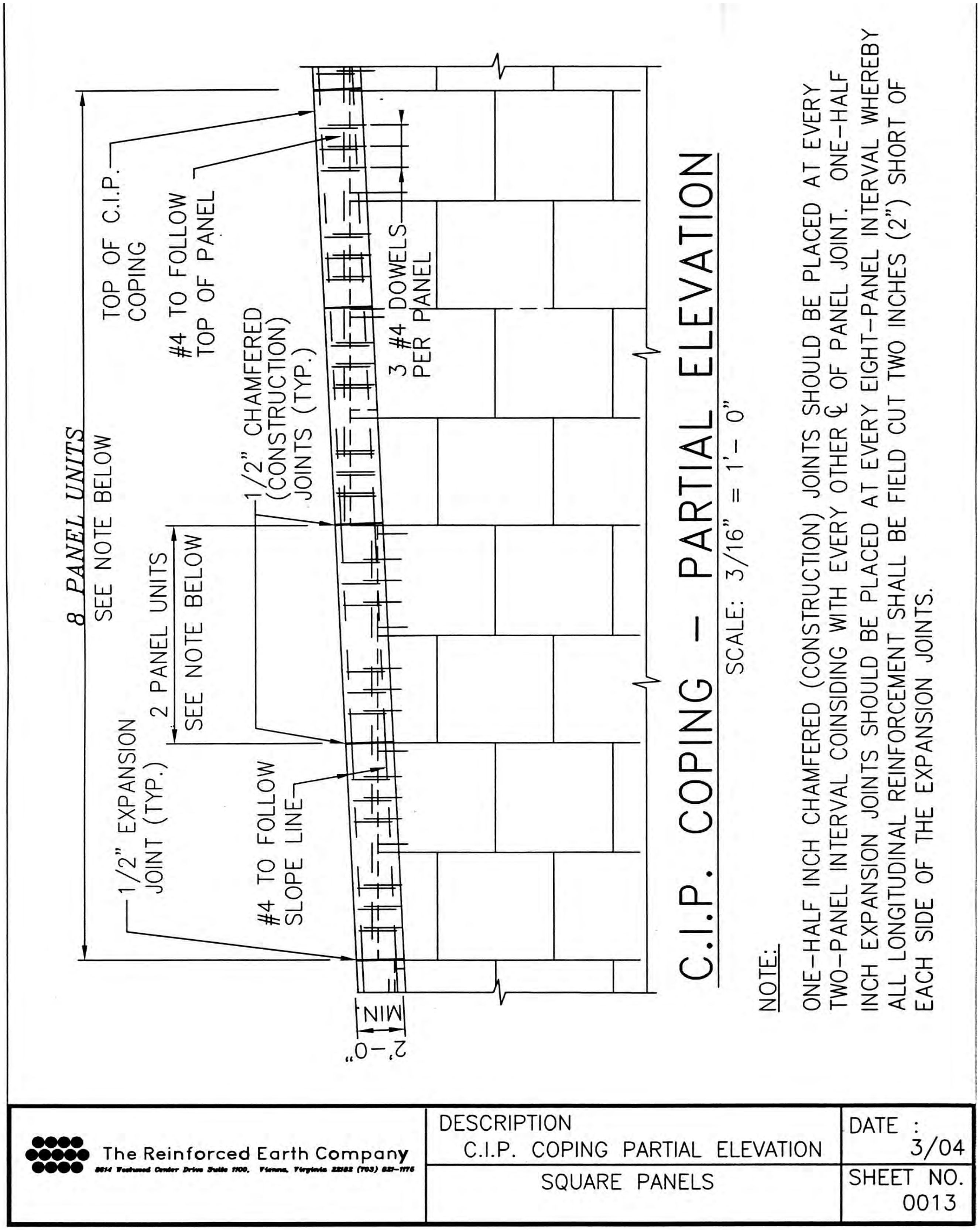
The Reinforced Earth Company	DESCRIPTION C.I.P. COPING W/ FENCE DETAIL	DATE : 3/04
		SHEET NO. 0011

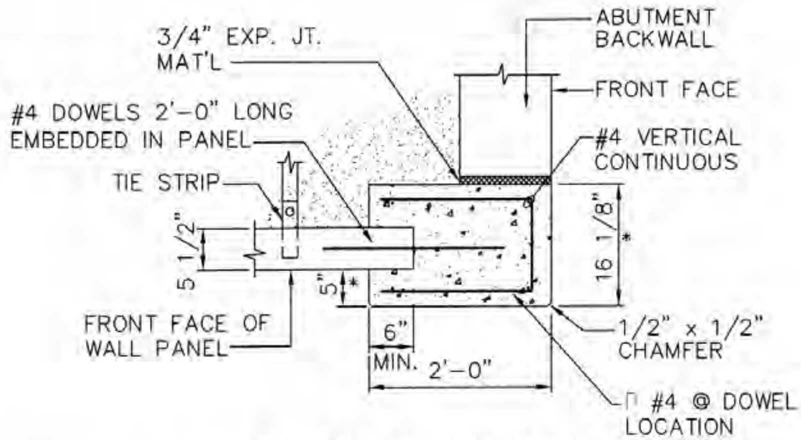


C.I.P. CONC. COPING

SCALE: 1" = 1'– 0"

The Reinforced Earth Company	DESCRIPTION C.I.P. COPING DETAIL	DATE : 3/04
		SHEET NO. 0009


8 PANEL UNITS
SEE NOTE BELOW
TOP OF C.I.P. COPING
#4 TO FOLLOW TOP OF PANEL
2 PANEL UNITS
SEE NOTE BELOW
1/2" CHAMFERED (CONSTRUCTION) JOINTS (TYP.)
3 #4 DOWELS PER PANEL
1/2" EXPANSION JOINT (TYP.)
#4 TO FOLLOW SLOPE LINE
2'–0" MIN.
C.I.P. COPING – PARTIAL ELEVATION
SCALE: 3/16" = 1'– 0"
NOTE:
ONE–HALF INCH CHAMFERED (CONSTRUCTION) JOINTS SHOULD BE PLACED AT EVERY TWO–PANEL INTERVAL COINSIDING WITH EVERY OTHER ℄ OF PANEL JOINT. ONE–HALF INCH EXPANSION JOINTS SHOULD BE PLACED AT EVERY EIGHT–PANEL INTERVAL WHEREBY ALL LONGITUDINAL REINFORCEMENT SHALL BE FIELD CUT TWO INCHES (2") SHORT OF EACH SIDE OF THE EXPANSION JOINTS.
The Reinforced Earth Company
DESCRIPTION
C.I.P. COPING PARTIAL ELEVATION
SQUARE PANELS
DATE : 3/04
SHEET NO. 0013

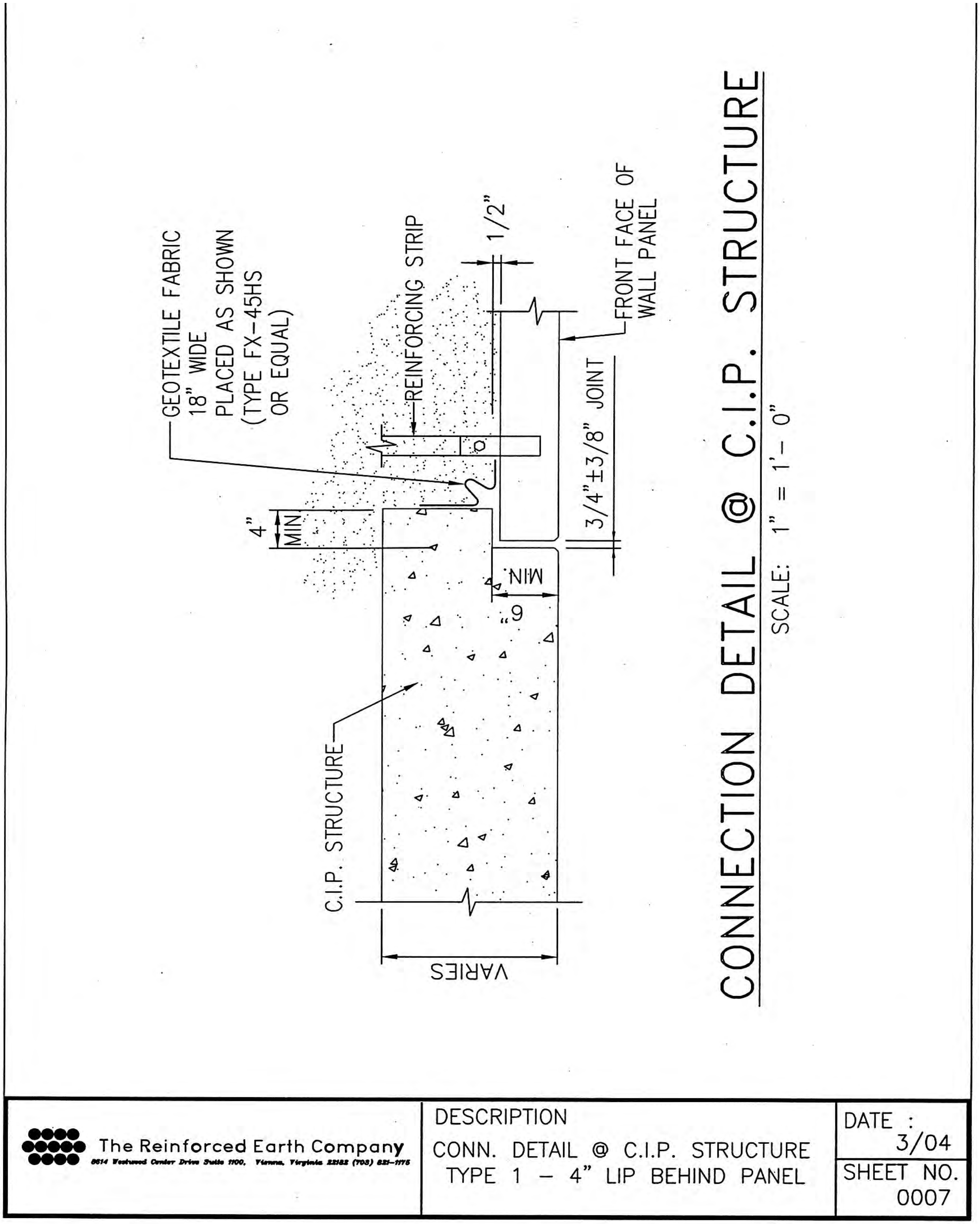


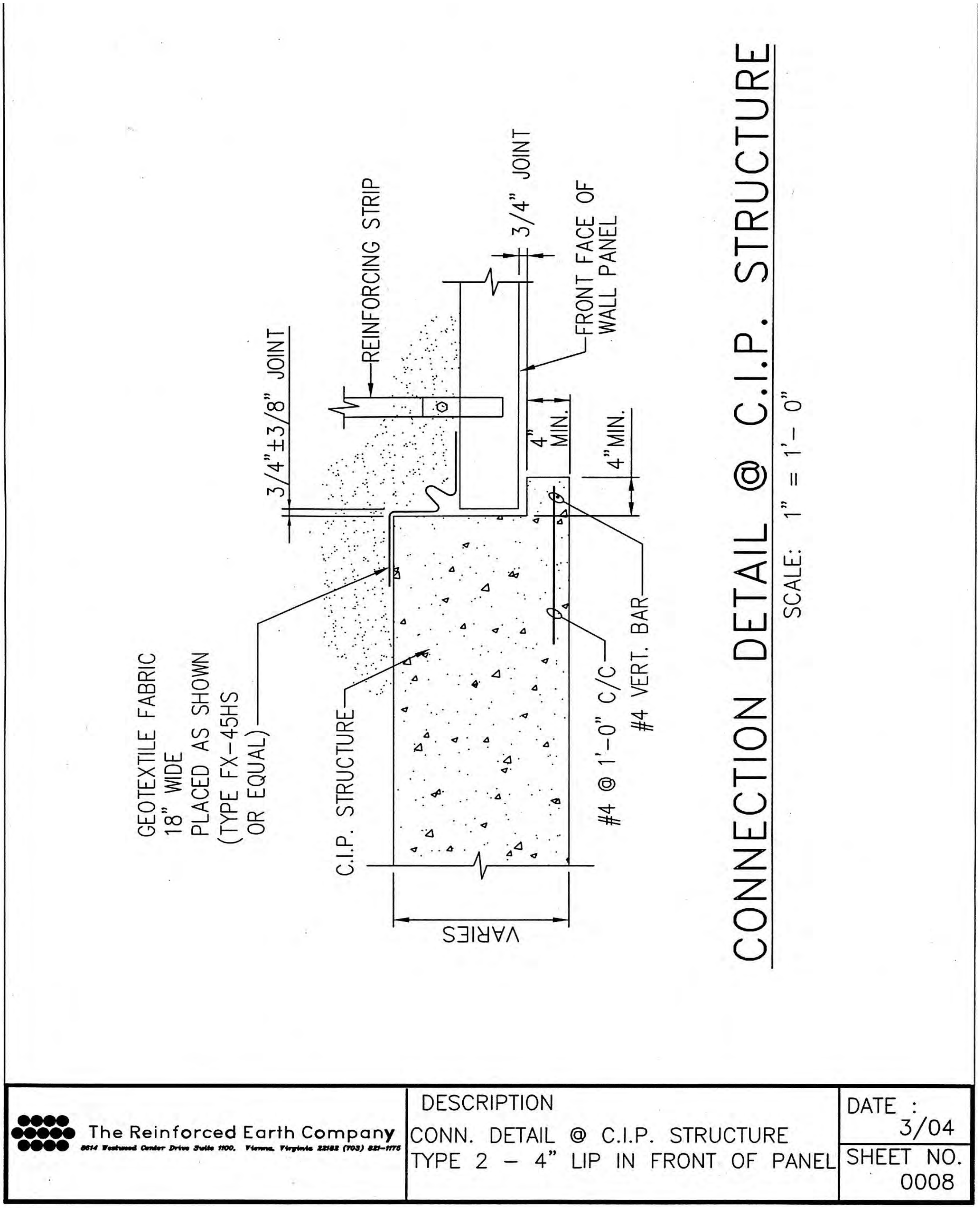
* TO MATCH PRECAST COPING

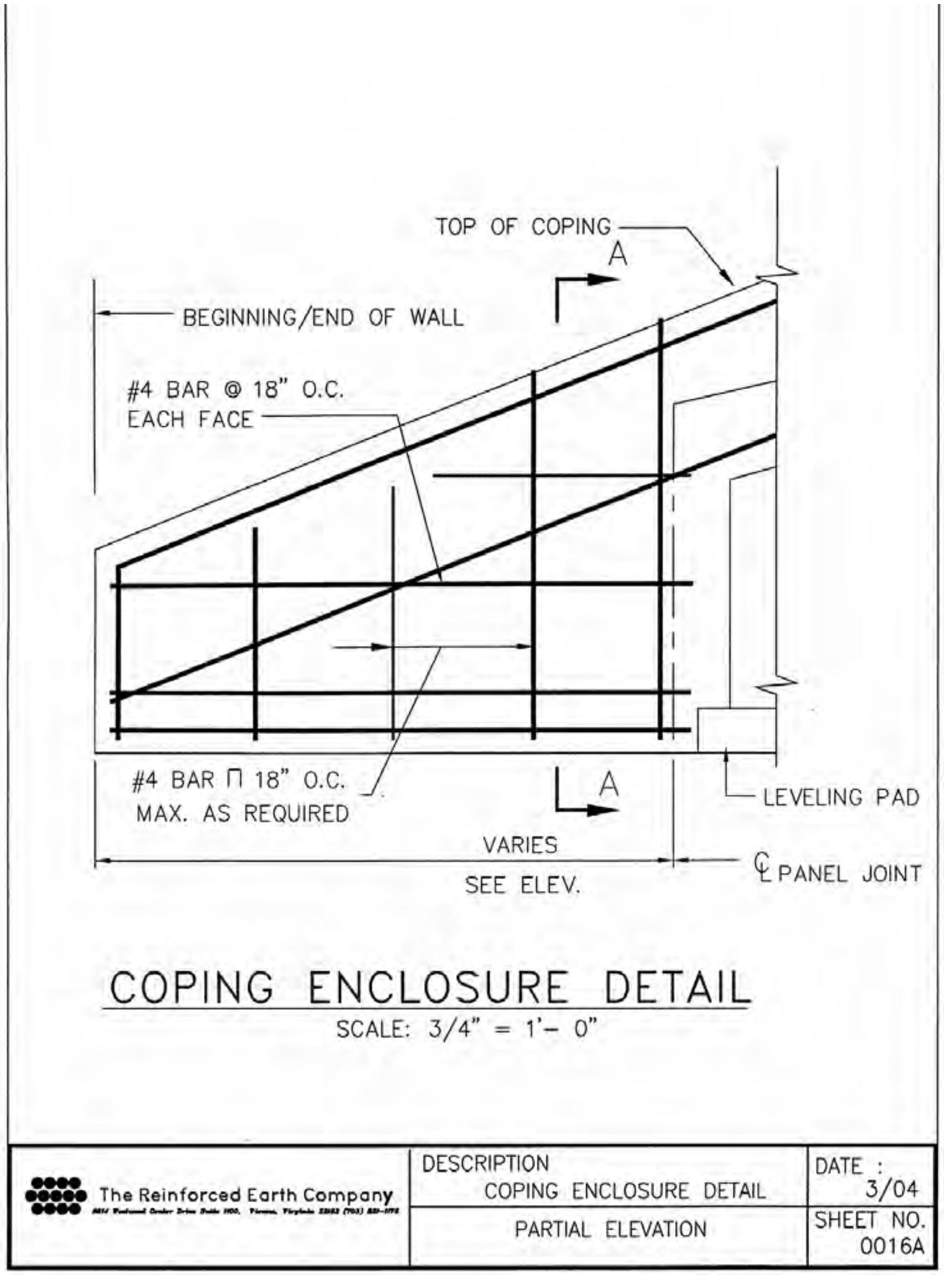
CONC VERTICAL COPING DETAIL

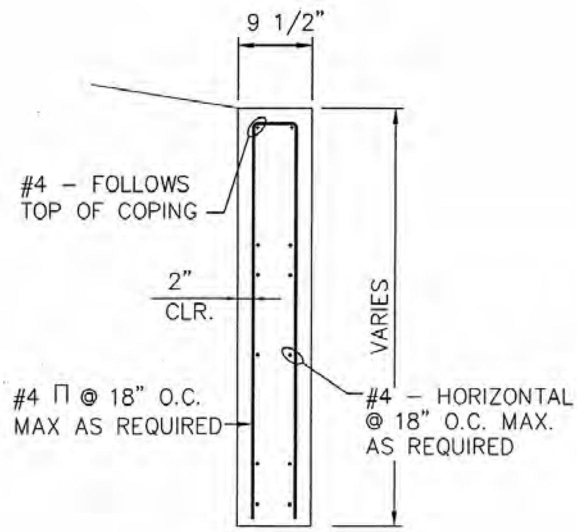
SCALE: 3/4"=1'-0"

The Reinforced Earth Company	DESCRIPTION C.I.P. VERTICAL COPING – TO MATCH PRECAST COPING	DATE : 3/04 SHEET NO. 0014


GEOTEXTILE FABRIC
18" WIDE
PLACED AS SHOWN
(TYPE FX-45HS
OR EQUAL)
REINFORCING STRIP
1/2"
FRONT FACE OF
WALL PANEL
4" MIN
6" MIN.
3/4"±3/8" JOINT
C.I.P. STRUCTURE
VARIES
CONNECTION DETAIL @ C.I.P. STRUCTURE
SCALE: 1" = 1'- 0"
The Reinforced Earth Company
8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175
DESCRIPTION
CONN. DETAIL @ C.I.P. STRUCTURE
TYPE 1 – 4" LIP BEHIND PANEL
DATE : 3/04
SHEET NO. 0007


CONNECTION DETAIL @ C.I.P. STRUCTURE
SCALE: 1" = 1'- 0"
REINFORCING STRIP
3/4" JOINT
FRONT FACE OF WALL PANEL
3/4"±3/8" JOINT
4" MIN.
4"MIN.
GEOTEXTILE FABRIC 18" WIDE PLACED AS SHOWN (TYPE FX-45HS OR EQUAL)
C.I.P. STRUCTURE
#4 @ 1'-0" C/C
#4 VERT. BAR
VARIES
The Reinforced Earth Company
DESCRIPTION
CONN. DETAIL @ C.I.P. STRUCTURE
TYPE 2 – 4" LIP IN FRONT OF PANEL
DATE : 3/04
SHEET NO. 0008

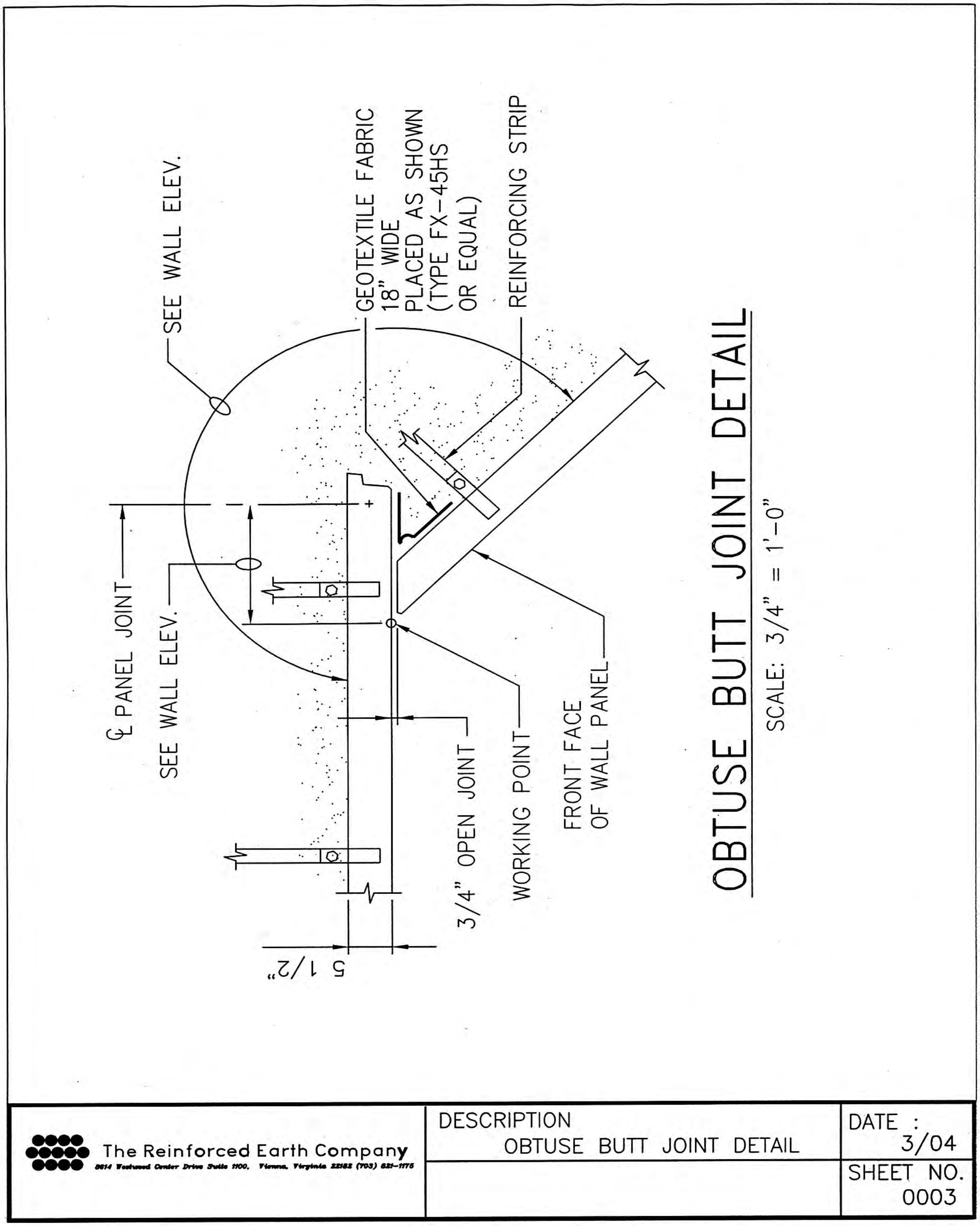

TOP OF COPING
A
BEGINNING/END OF WALL
#4 BAR @ 18" O.C.
EACH FACE
#4 BAR ⊓ 18" O.C.
MAX. AS REQUIRED
A
LEVELING PAD
VARIES
SEE ELEV.
℄ PANEL JOINT
COPING ENCLOSURE DETAIL
SCALE: 3/4" = 1'- 0"
The Reinforced Earth Company
DESCRIPTION
COPING ENCLOSURE DETAIL
PARTIAL ELEVATION
DATE :
3/04
SHEET NO.
0016A

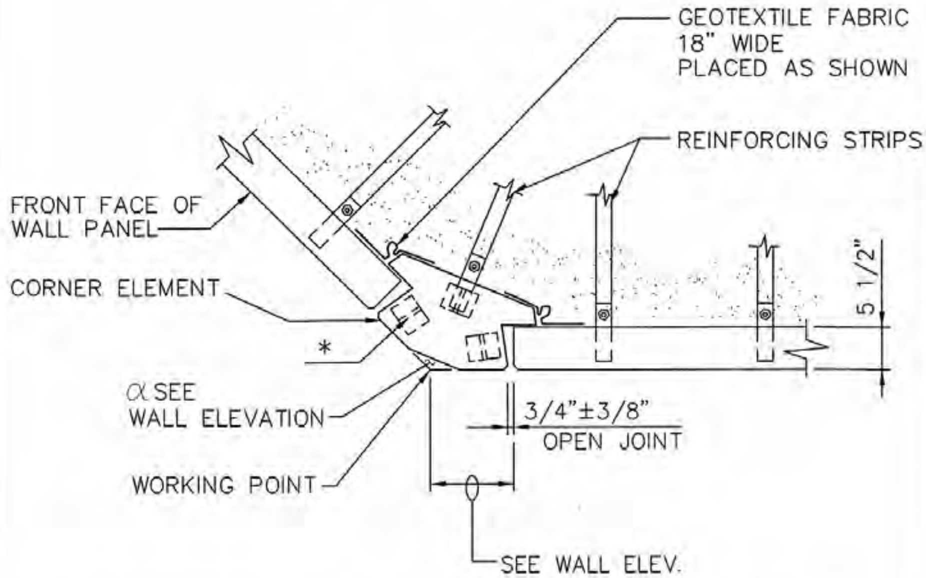


SECTION A–A

SCALE: 3/4" = 1'– 0"

The Reinforced Earth Company	DESCRIPTION COPING ENCLOSURE DETAIL	DATE : 3/04
	SECTION A–A	SHEET NO. 0016B


SEE WALL ELEV.
GEOTEXTILE FABRIC
18" WIDE
PLACED AS SHOWN
(TYPE FX-45HS
OR EQUAL)
REINFORCING STRIP
℄ PANEL JOINT
SEE WALL ELEV.
3/4" OPEN JOINT
WORKING POINT
FRONT FACE
OF WALL PANEL
5 1/2"
OBTUSE BUTT JOINT DETAIL
SCALE: 3/4" = 1'-0"
The Reinforced Earth Company
8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175
DESCRIPTION
OBTUSE BUTT JOINT DETAIL
DATE :
3/04
SHEET NO.
0003

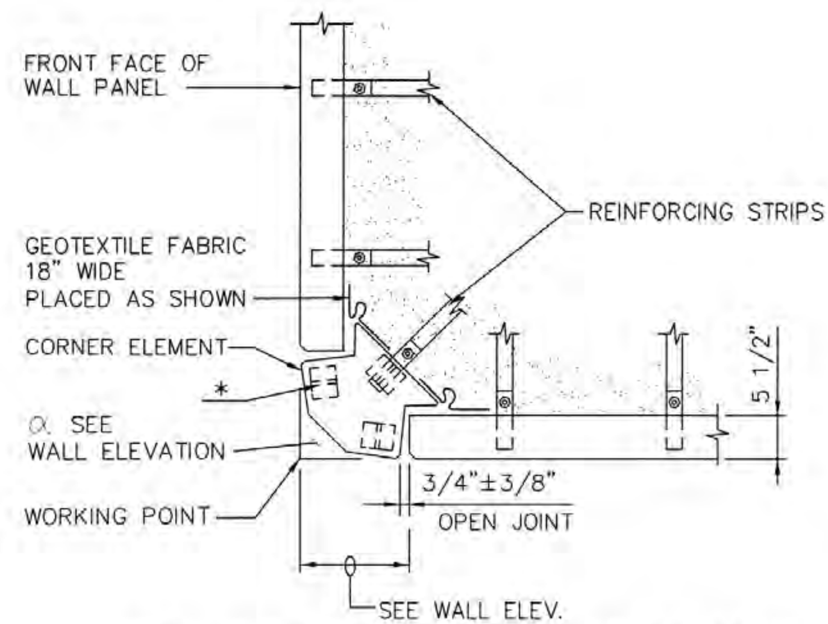


OBTUSE CORNER ELEMENT DETAIL

SCALE: 3/4" = 1'- 0"

* THREE BEARING PADS PER UNIT, BASE STEM OF BEARING PAD SHALL BE FIELD CUT TO FIT FLAT ON TOP OF CORNER ELEMENT. FRONT PADS SHALL BE PLACED ON INSIDE EDGE OF LIP.

The Reinforced Earth Company	DESCRIPTION OBTUSE CORNER ELEMENT DETAIL	DATE : 3/04
		SHEET NO. 0006

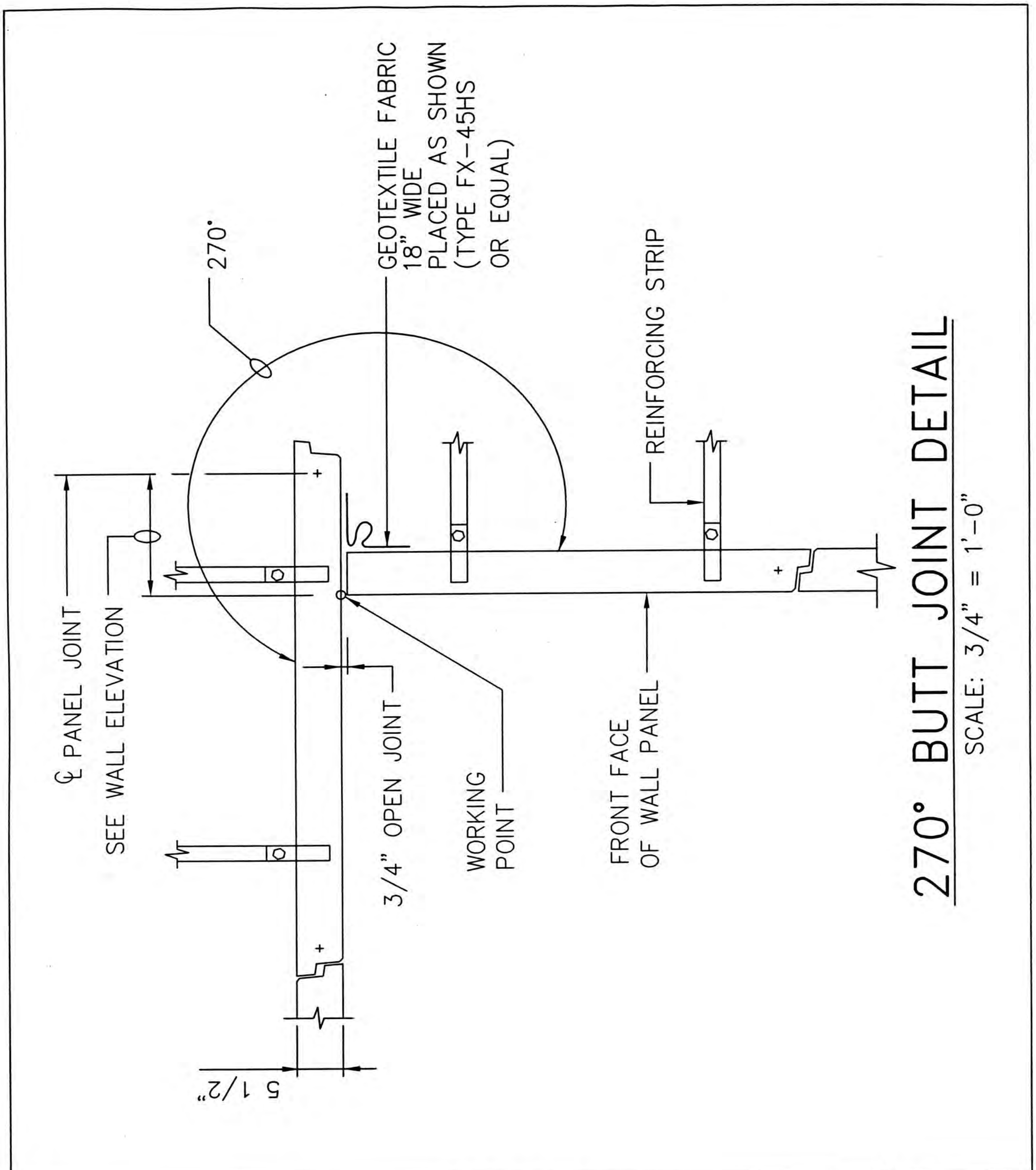


90° CORNER ELEMENT DETAIL

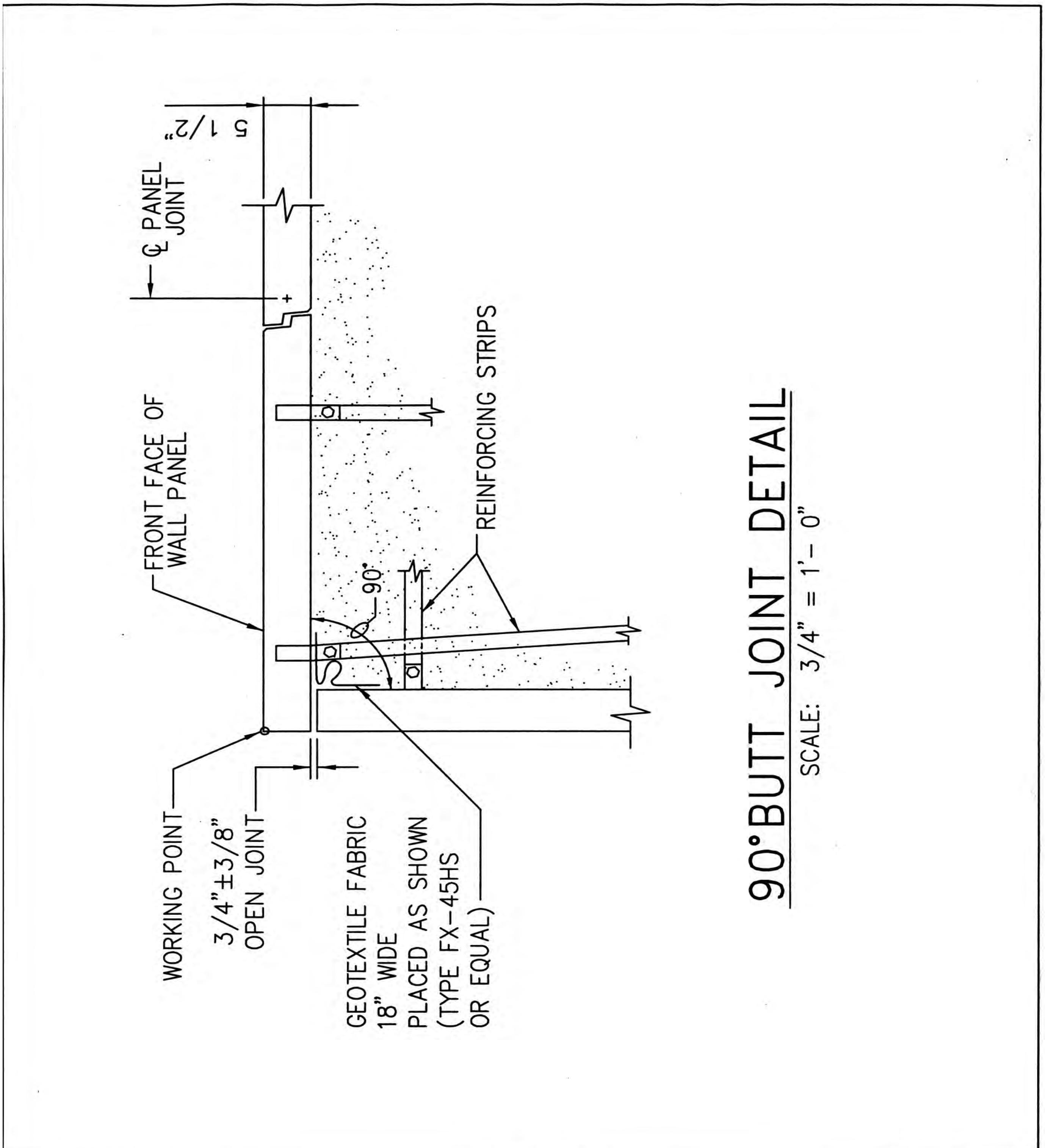
SCALE: 3/4" = 1'– 0"

* THREE BEARING PADS PER UNIT, BASE STEM OF BEARING PAD SHALL BE FIELD CUT TO FIT FLAT ON TOP OF CORNER ELEMENT. FRONT PADS SHALL BE PLACED ON INSIDE EDGE OF LIP.

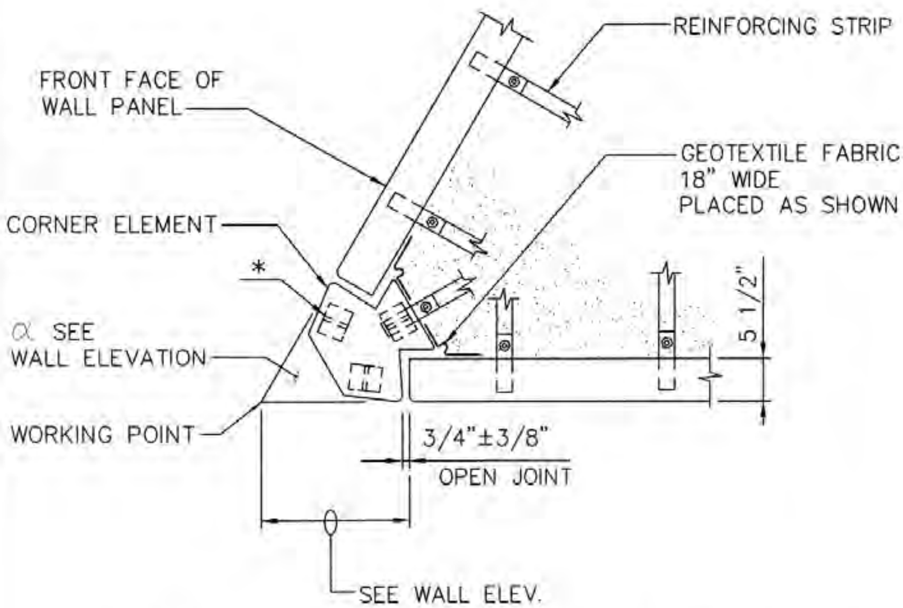
The Reinforced Earth Company	DESCRIPTION 90° CORNER ELEMENT DETAIL	DATE : 3/04
		SHEET NO. 0005



The Reinforced Earth Company 8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175	DESCRIPTION 270° BUTT JOINT DETAIL	DATE : 3/04
		SHEET NO. 0002



The Reinforced Earth Company	DESCRIPTION 90° BUTT JOINT DETAIL	DATE : 3/04
		SHEET NO. 0001

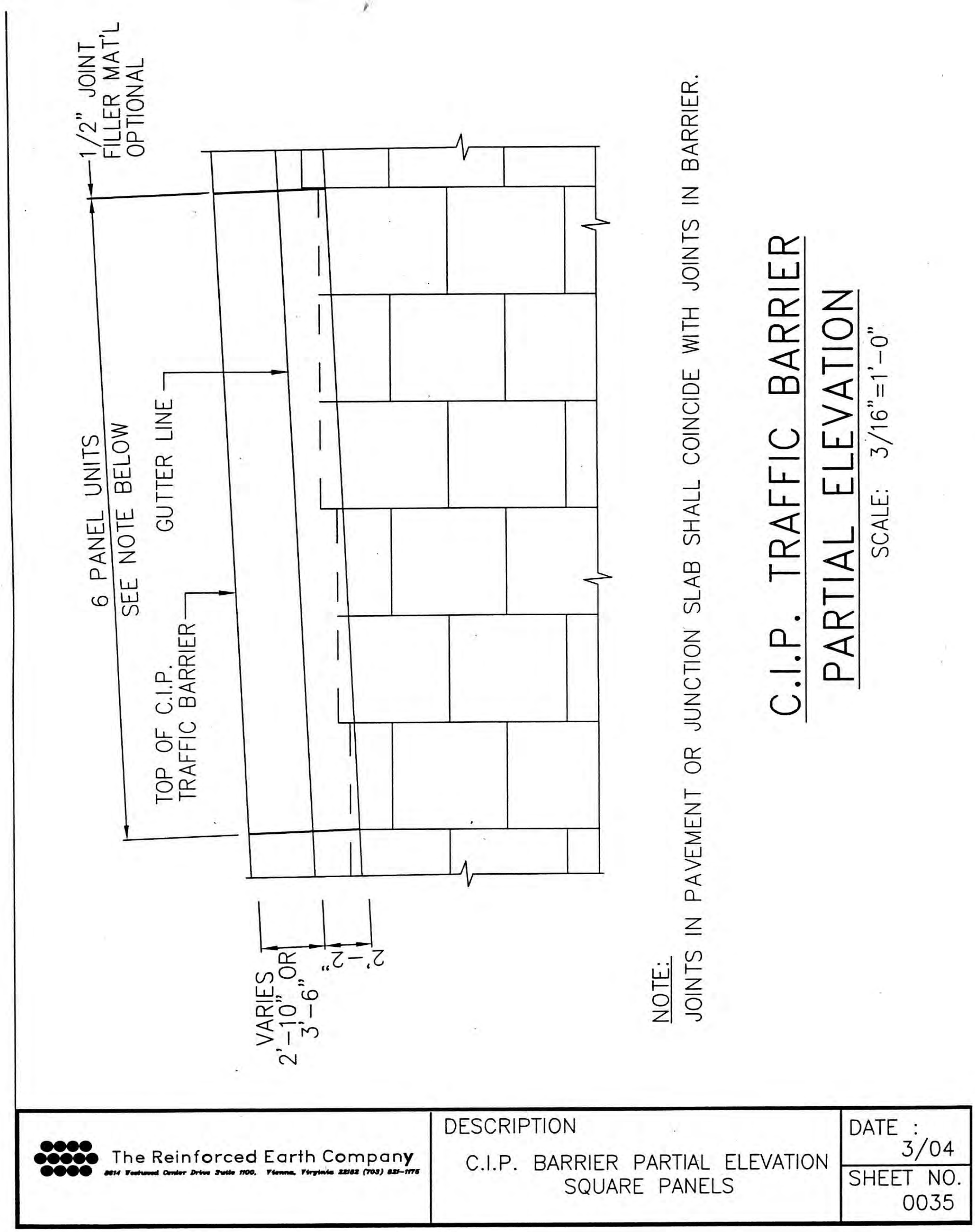


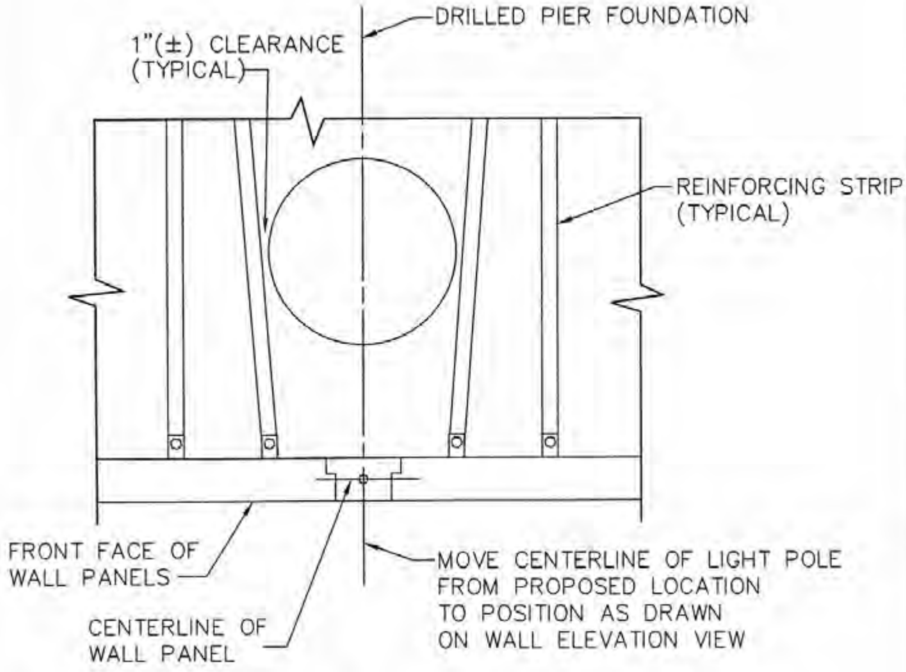
ACUTE CORNER ELEMENT DETAIL

SCALE: 3/4" = 1'– 0"

* THREE BEARING PADS PER UNIT, BASE STEM OF BEARING PAD SHALL BE FIELD CUT TO FIT FLAT ON TOP OF CORNER ELEMENT. FRONT PADS SHALL BE PLACED ON INSIDE EDGE OF LIP.

The Reinforced Earth Company	DESCRIPTION ACUTE CORNER ELEMENT DETAIL	DATE : 3/04
		SHEET NO. 0004

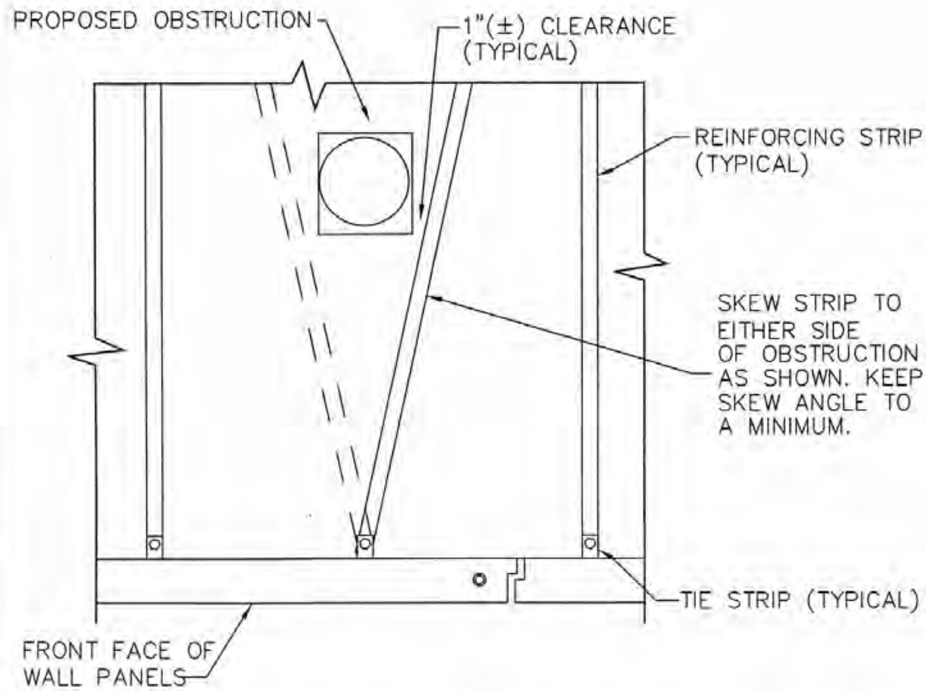

1/2" JOINT FILLER MAT'L OPTIONAL
6 PANEL UNITS
SEE NOTE BELOW
GUTTER LINE
TOP OF C.I.P. TRAFFIC BARRIER
VARIES 2'-10" OR 3'-6"
2'-2"
NOTE:
JOINTS IN PAVEMENT OR JUNCTION SLAB SHALL COINCIDE WITH JOINTS IN BARRIER.
C.I.P. TRAFFIC BARRIER
PARTIAL ELEVATION
SCALE: 3/16"=1'-0"
The Reinforced Earth Company
DESCRIPTION
C.I.P. BARRIER PARTIAL ELEVATION
SQUARE PANELS
DATE : 3/04
SHEET NO. 0035



PARTIAL WALL PLAN AT LIGHT POLE

SCALE: 3/4" = 1'-0"

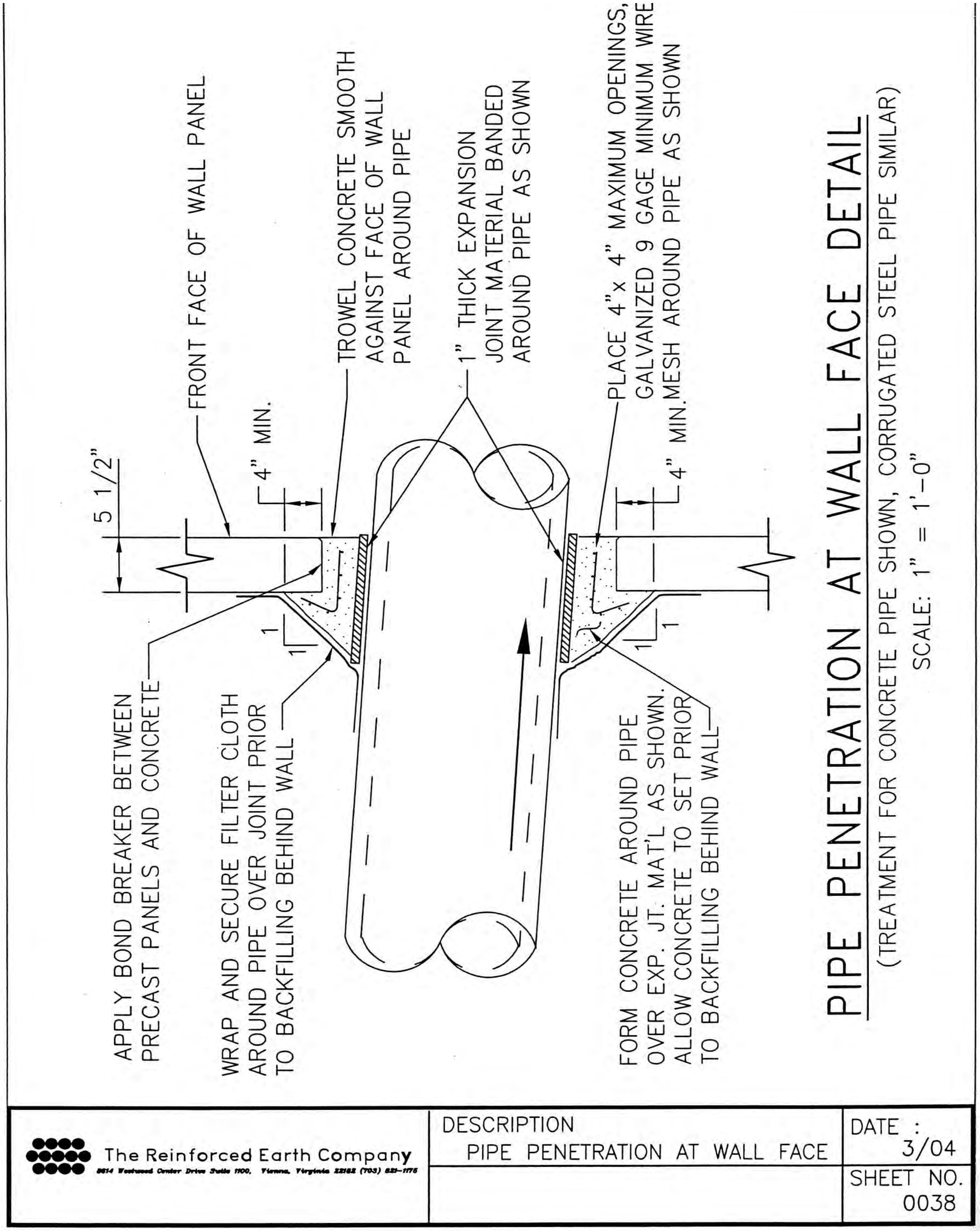
The Reinforced Earth Company	DESCRIPTION PARTIAL PLAN AT LIGHT POLE	DATE : 3/04
		SHEET NO. 0036



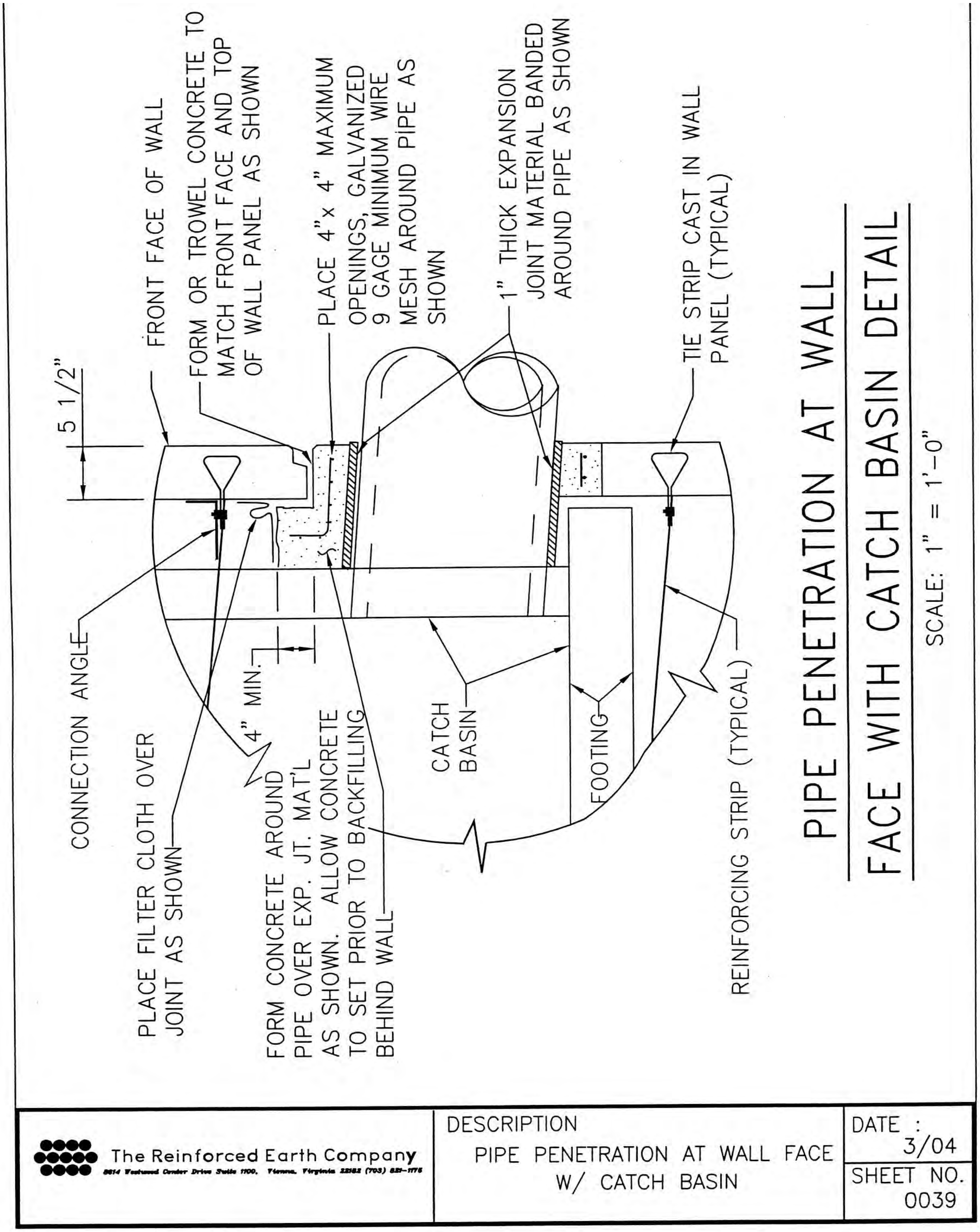
PARTIAL WALL PLAN AT OBSTRUCTION

SCALE: 3/4" = 1'–0"

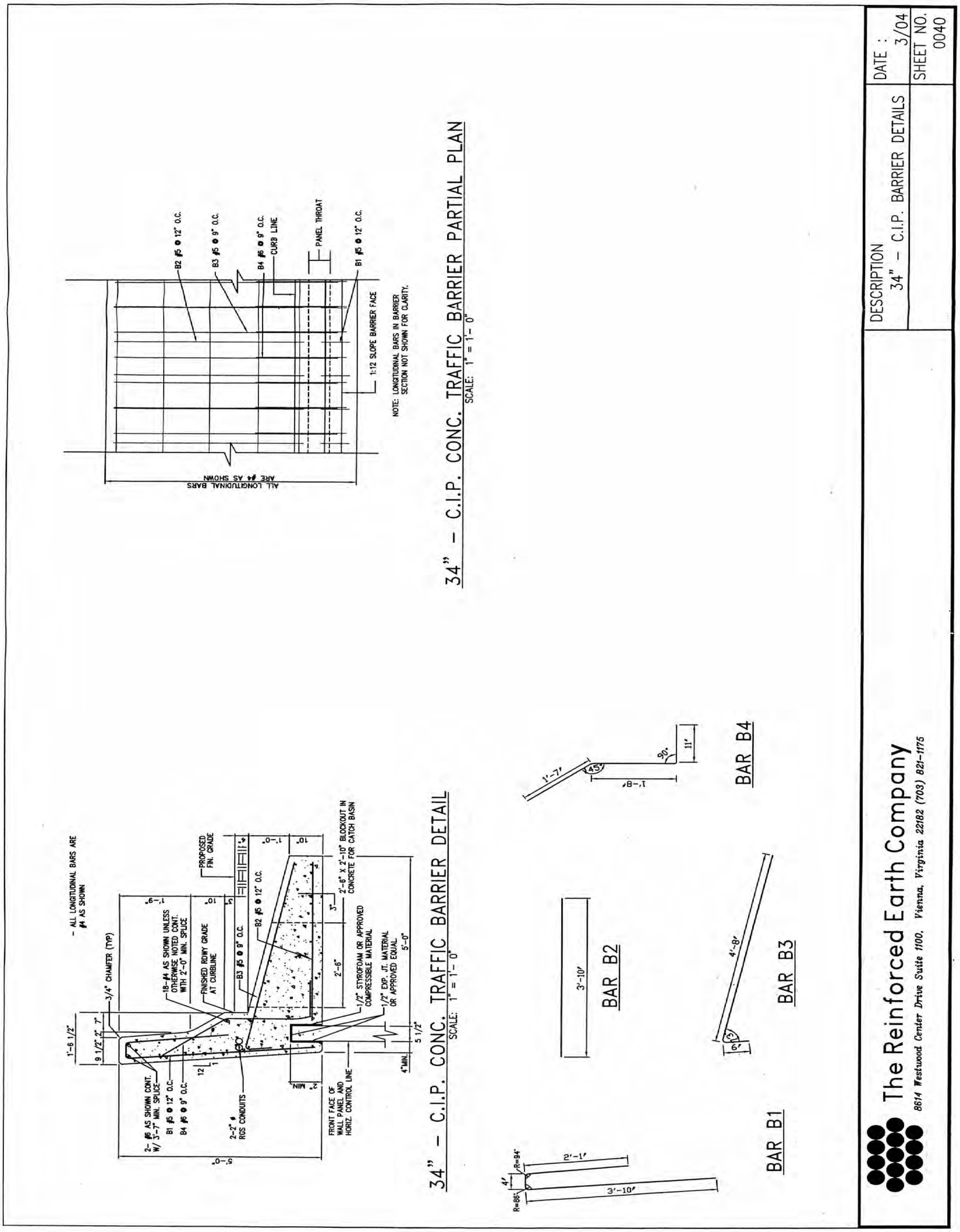
The Reinforced Earth Company	DESCRIPTION PARTIAL PLAN AT GENERAL OBSTRUCTION	DATE : 3/04 SHEET NO. 0037



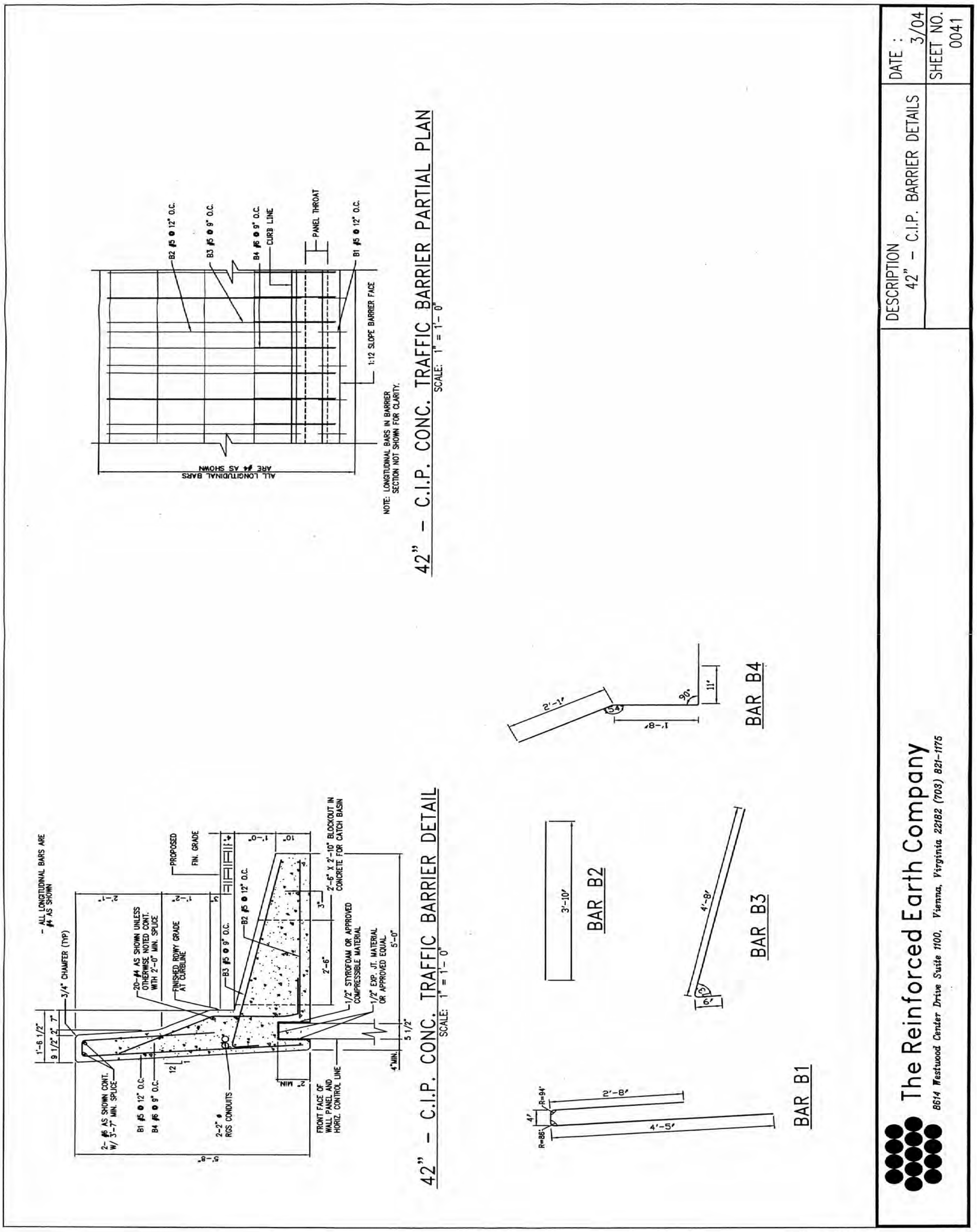
FRONT FACE OF WALL PANEL
TROWEL CONCRETE SMOOTH AGAINST FACE OF WALL PANEL AROUND PIPE
1" THICK EXPANSION JOINT MATERIAL BANDED AROUND PIPE AS SHOWN
PLACE 4"x 4" MAXIMUM OPENINGS, GALVANIZED 9 GAGE MINIMUM WIRE MESH AROUND PIPE AS SHOWN
5 1/2"
4" MIN.
4" MIN.
1
1
1
1
APPLY BOND BREAKER BETWEEN PRECAST PANELS AND CONCRETE
WRAP AND SECURE FILTER CLOTH AROUND PIPE OVER JOINT PRIOR TO BACKFILLING BEHIND WALL
FORM CONCRETE AROUND PIPE OVER EXP. JT. MAT'L AS SHOWN. ALLOW CONCRETE TO SET PRIOR TO BACKFILLING BEHIND WALL
PIPE PENETRATION AT WALL FACE DETAIL
(TREATMENT FOR CONCRETE PIPE SHOWN, CORRUGATED STEEL PIPE SIMILAR)
SCALE: 1" = 1'-0"
The Reinforced Earth Company
8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175
DESCRIPTION
PIPE PENETRATION AT WALL FACE
DATE :
3/04
SHEET NO.
0038

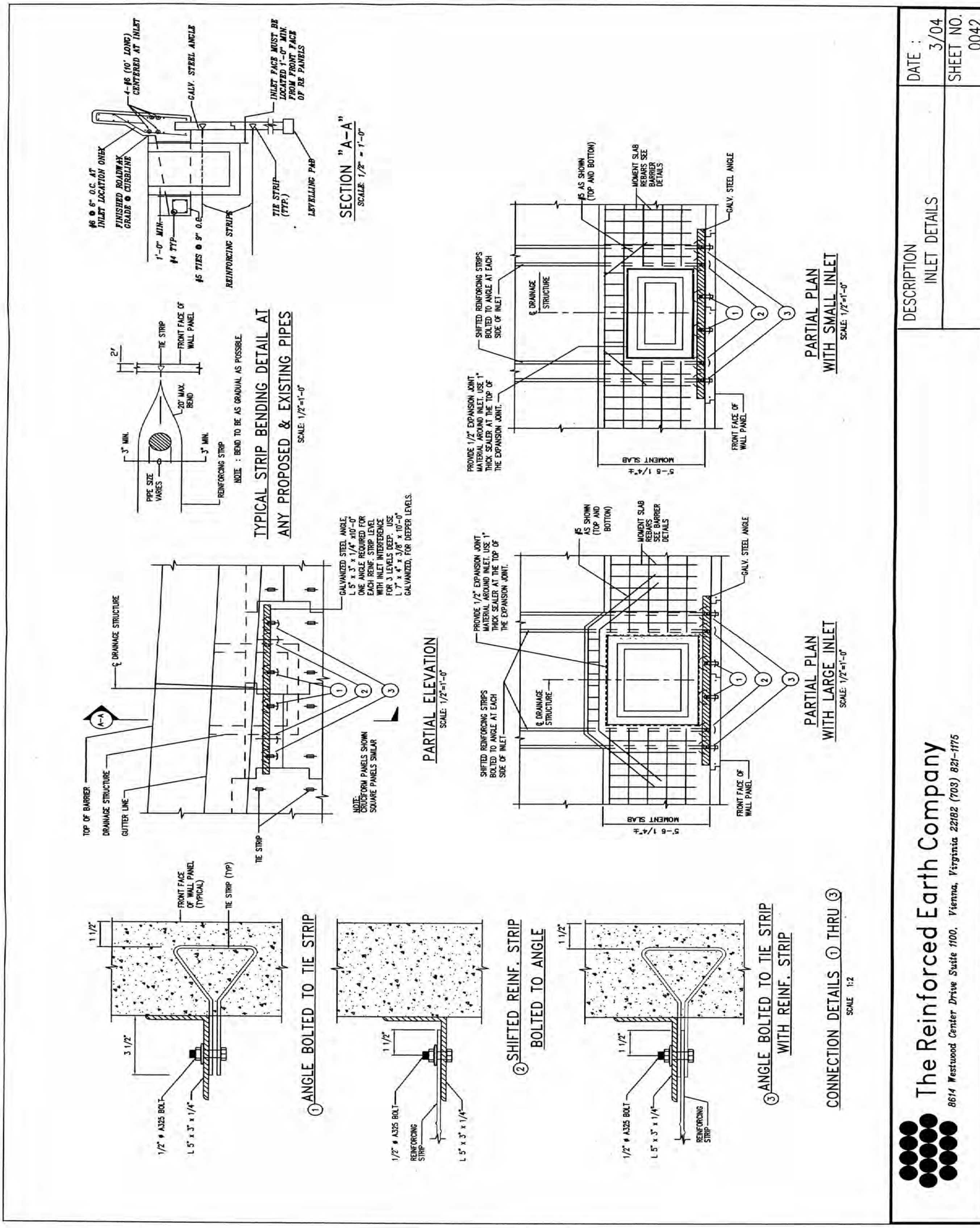


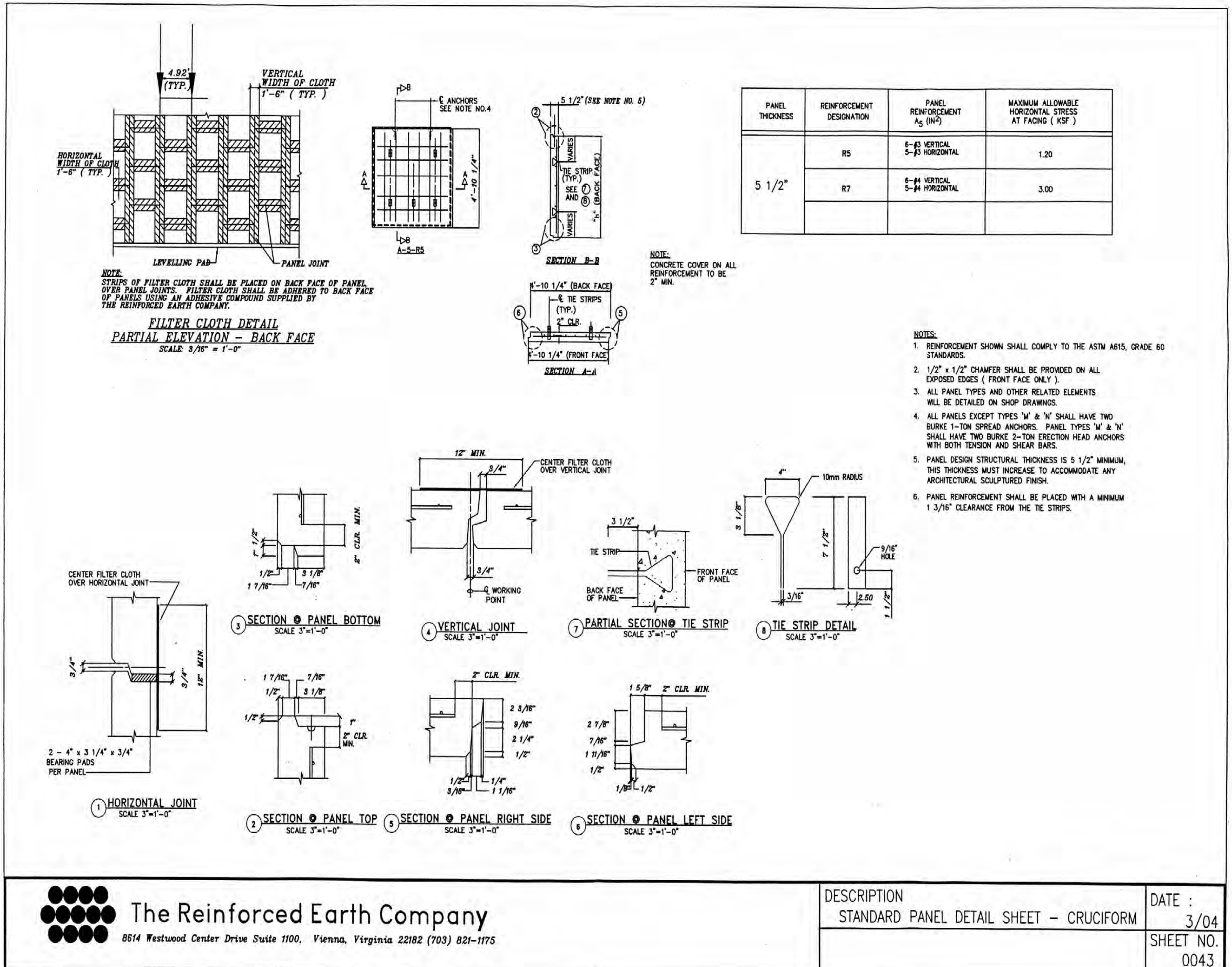
FRONT FACE OF WALL
FORM OR TROWEL CONCRETE TO MATCH FRONT FACE AND TOP OF WALL PANEL AS SHOWN
PLACE 4"x 4" MAXIMUM OPENINGS, GALVANIZED 9 GAGE MINIMUM WIRE MESH AROUND PIPE AS SHOWN
1" THICK EXPANSION JOINT MATERIAL BANDED AROUND PIPE AS SHOWN
TIE STRIP CAST IN WALL PANEL (TYPICAL)
5 1/2"
CONNECTION ANGLE
PLACE FILTER CLOTH OVER JOINT AS SHOWN
4" MIN.
FORM CONCRETE AROUND PIPE OVER EXP. JT. MAT'L AS SHOWN. ALLOW CONCRETE TO SET PRIOR TO BACKFILLING BEHIND WALL
CATCH BASIN
FOOTING
REINFORCING STRIP (TYPICAL)
PIPE PENETRATION AT WALL
FACE WITH CATCH BASIN DETAIL
SCALE: 1" = 1'-0"
The Reinforced Earth Company
DESCRIPTION
PIPE PENETRATION AT WALL FACE W/ CATCH BASIN
DATE : 3/04
SHEET NO. 0039



34" – C.I.P. CONC. TRAFFIC BARRIER PARTIAL PLAN
SCALE: 1" = 1'-0"
NOTE: LONGITUDINAL BARS IN BARRIER SECTION NOT SHOWN FOR CLARITY.
ALL LONGITUDINAL BARS ARE #4 AS SHOWN
B2 #5 @ 12" O.C.
B3 #5 @ 9" O.C.
B4 #6 @ 9" O.C.
CURB LINE
PANEL THROAT
B1 #5 @ 12" O.C.
1:12 SLOPE BARRIER FACE
34" – C.I.P. CONC. TRAFFIC BARRIER DETAIL
SCALE: 1" = 1'-0"
ALL LONGITUDINAL BARS ARE #4 AS SHOWN
3/4" CHAMFER (TYP)
PROPOSED FIN. GRADE
FINISHED RDWY GRADE AT CURBLINE
2'-6" X 2'-10" BLOCKOUT IN CONCRETE FOR CATCH BASIN
1/2" STYROFOAM OR APPROVED COMPRESSIBLE MATERIAL
1/2" EXP. JT. MATERIAL OR APPROVED EQUAL
2-2" ø RGS CONDUITS
FRONT FACE OF WALL PANEL AND HORIZ. CONTROL LINE
5'-0"
BAR B1
BAR B2
BAR B3
BAR B4
The Reinforced Earth Company
8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175
DESCRIPTION
34" – C.I.P. BARRIER DETAILS
DATE : 3/04
SHEET NO. 0040


42" – C.I.P. CONC. TRAFFIC BARRIER PARTIAL PLAN
SCALE: 1" = 1'- 0"
NOTE: LONGITUDINAL BARS IN BARRIER SECTION NOT SHOWN FOR CLARITY.
ALL LONGITUDINAL BARS ARE #4 AS SHOWN
B2 #5 @ 12" O.C.
B3 #5 @ 9" O.C.
B4 #6 @ 9" O.C.
CURB LINE
PANEL THROAT
B1 #5 @ 12" O.C.
1:12 SLOPE BARRIER FACE
42" – C.I.P. CONC. TRAFFIC BARRIER DETAIL
SCALE: 1" = 1'- 0"
BAR B1
BAR B2
BAR B3
BAR B4
DESCRIPTION
42" – C.I.P. BARRIER DETAILS
DATE :
3/04
SHEET NO.
0041
The Reinforced Earth Company
8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175


1/2" ø A325 BOLT
L 5" x 3" x 1/4"
3 1/2"
1 1/2"
FRONT FACE OF WALL PANEL (TYPICAL)
TIE STRIP (TYP)
① ANGLE BOLTED TO TIE STRIP
REINFORCING STRIP
② SHIFTED REINF. STRIP BOLTED TO ANGLE
③ ANGLE BOLTED TO TIE STRIP WITH REINF. STRIP
CONNECTION DETAILS ① THRU ③
SCALE 1:2
TOP OF BARRIER
DRAINAGE STRUCTURE
GUTTER LINE
℄ DRAINAGE STRUCTURE
A-A
TIE STRIP
NOTE: CRUCIFORM PANELS SHOWN SQUARE PANELS SIMILAR
GALVANIZED STEEL ANGLE, L 5" x 3" x 1/4" x10'-0" ONE ANGLE REQUIRED FOR EACH REINF. STRIP LEVEL WITH INLET INTERFERENCE FOR 3 LEVELS DEEP. USE L 7" x 4" x 3/8" x 10'-0" GALVANIZED, FOR DEEPER LEVELS.
PARTIAL ELEVATION
SCALE: 1/2"=1'-0"
PIPE SIZE VARIES
3" MIN.
20° MAX. BEND
2'
TIE STRIP
FRONT FACE OF WALL PANEL
REINFORCING STRIP
NOTE : BEND TO BE AS GRADUAL AS POSSIBLE.
TYPICAL STRIP BENDING DETAIL AT ANY PROPOSED & EXISTING PIPES
SCALE: 1/2"=1'-0"
#6 @ 6" O.C. AT INLET LOCATION ONLY
FINISHED ROADWAY GRADE @ CURBLINE
4-#6 (10' LONG) CENTERED AT INLET
1'-0" MIN.
#4 TYP
GALV. STEEL ANGLE
#5 TIES @ 9" O.C.
REINFORCING STRIPS
TIE STRIP (TYP.)
INLET FACE MUST BE LOCATED 1'-0" MIN. FROM FRONT FACE OF RE PANELS
LEVELLING PAD
SECTION "A-A"
SCALE 1/2" = 1'-0"
SHIFTED REINFORCING STRIPS BOLTED TO ANGLE AT EACH SIDE OF INLET
PROVIDE 1/2" EXPANSION JOINT MATERIAL AROUND INLET. USE 1" THICK SEALER AT THE TOP OF THE EXPANSION JOINT.
℄ DRAINAGE STRUCTURE
#5 AS SHOWN (TOP AND BOTTOM)
MOMENT SLAB REBARS SEE BARRIER DETAILS
5'-6 1/4"±
MOMENT SLAB
FRONT FACE OF WALL PANEL
GALV. STEEL ANGLE
PARTIAL PLAN WITH LARGE INLET
SCALE: 1/2"=1'-0"
PARTIAL PLAN WITH SMALL INLET
SCALE: 1/2"=1'-0"
The Reinforced Earth Company
8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175
DESCRIPTION
INLET DETAILS
DATE : 3/04
SHEET NO. 0042



PANEL THICKNESS	REINFORCEMENT DESIGNATION	PANEL REINFORCEMENT A_S (IN^2)	MAXIMUM ALLOWABLE HORIZONTAL STRESS AT FACING (KSF)
5 1/2"	R5	6-#3 VERTICAL 5-#3 HORIZONTAL	1.20
	R7	6-#4 VERTICAL 5-#4 HORIZONTAL	3.00

NOTES:
1. REINFORCEMENT SHOWN SHALL COMPLY TO THE ASTM A615, GRADE 60 STANDARDS.
2. 1/2" x 1/2" CHAMFER SHALL BE PROVIDED ON ALL EXPOSED EDGES (FRONT FACE ONLY).
3. ALL PANEL TYPES AND OTHER RELATED ELEMENTS WILL BE DETAILED ON SHOP DRAWINGS.
4. ALL PANELS EXCEPT TYPES 'M' & 'N' SHALL HAVE TWO BURKE 1-TON SPREAD ANCHORS. PANEL TYPES 'M' & 'N' SHALL HAVE TWO BURKE 2-TON ERECTION HEAD ANCHORS WITH BOTH TENSION AND SHEAR BARS.
5. PANEL DESIGN STRUCTURAL THICKNESS IS 5 1/2" MINIMUM, THIS THICKNESS MUST INCREASE TO ACCOMMODATE ANY ARCHITECTURAL SCULPTURED FINISH.
6. PANEL REINFORCEMENT SHALL BE PLACED WITH A MINIMUM 1 3/16" CLEARANCE FROM THE TIE STRIPS.

Appendix 15-K Preapproved Wall Appendix: Specific Requirements and Details for Tensar ARES Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply:

The detailed design methodology, design properties, and assumptions used by Tensar Earth Technologies for the ARES wall are summarized in the HITEC evaluation report for this wall system (HITEC, 1997, *Evaluation of the Tensar ARES Retaining Wall System*, ASCE, CERF Report No. 40301). The design methodology, which is based on the Standard Specifications for Highway Bridges (2002) is consistent with the general design requirements in Appendix 15-A, except as noted below. Interim approval is given for the continued use of the AASHTO Standard Specifications as the basis for design.

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications. For LRFD based design, while it is recognized that product and soil type specific pullout interaction coefficients obtained in accordance with the AASHTO LRFD Specifications for the Tensar products used with this wall system are provided in the HITEC report for the ARES Wall system, pullout resistance design using these product and soil specific interaction coefficients has not been calibrated using the available product specific data statistics and reliability theory. Therefore, the specified resistance factors in the GDM and AASHTO LRFD Specifications should not be considered applicable to the product specific pullout interaction coefficients provided in the HITEC report.

The reinforcement long-term tensile strengths (T_{al}) provided in the WSDOT *Qualified Products List* (QPL) for the Tensar Geogrid product series, which are based on the 2003 version of the product series, shall be used for wall design, until such time that they are updated, and the updated strengths approved for WSDOT use in accordance with WSDOT Standard Practice T 925. Until such time that the long-term reinforcement strengths are updated, it shall be verified that any material sent to the project site for this wall system is the 2003 version of the product. Furthermore, the short-term ultimate tensile strengths (ASTM D6637) listed in the QPL shall be used as the basis for quality assurance testing and acceptance of the product as shipped to the project site per the *Standard Specifications* for Construction.

The HITEC report provided details and design criteria for a panel slot connector to attach the geogrid reinforcement to the facing panel. Due to problems with cracking of the facing panel at the location of the slot, that connection system has been discontinued and replaced with a full thickness panel in which geogrid tabs have been embedded into the panel. For this new connection system, the geogrid reinforcement is connected to the geogrid tab through the use of a Bodkin joint. Construction and fabrication inspectors should verify that the panels to be used for WSDOT projects do not contain the discontinued slot connector.

The Bodkin connection test results provided by letter to WSDOT dated September 28, 2004, were performed on the 2003 version of the Tensar geogrid product line. In that letter, it was stated that UMESA6 (UX1700HS) will typically be used for the connector tabs, regardless of the product selected for the reinforcement. If a lighter weight product is used for the connector tabs, the connection strength will need to be reduced accordingly. Table 15-(Tensar ARES)-1 provides a summary of the connection strengths that are approved for use with the ARES wall system.

Table 15-K-1 Approved Connection Strength Design Values for Tensar Ares Walls

Tensar Soil Reinforcement Geogrid Product	Tensar Panel Connector Tab Geogrid Product	T_{ult} (MARV) for Geogrid Reinforcement per ASTM D6637 in WSDOT QPL (lbs/ft)	CR_u	RF	T_{ac} (lbs/ft)
UMESA3/UX1400HS	UMESA6/UX1700HS	4,820	1.0	3.6	1,340
UMESA4/UX1500HS	UMESA6/UX1700HS	7,880	1.0	3.5	2,250
UMESA5/UX1600HS	UMESA6/UX1700HS	9,870	1.0	3.4	2,900
UMESA6/UX1700HS	UMESA6/UX1700HS	12,200	0.91	3.3	3,360
UMESA3/UX1400HS	UMESA3/UX1400HS	4,820	0.85	3.6	1,140
UMESA4/UX1500HS	UMESA4/UX1500HS	7,880	0.79	3.5	1,780
UMESA5/UX1600HS	UMESA5/UX1600HS	9,870	0.87	3.4	2,530
UMESA6/UX1700HS	UMESA6/UX1700HS	12,200	0.91	3.3	3,360

T_{ac}, the long-term connection strength, shall be calculated as follows for the Tensar ARES wall:

$$T_{ac} = \frac{T_{MARV} \bullet CR_u}{RF} \qquad (15\text{-(Tensar ARES)-1})$$

Where:

$RF = RF_{ID} \times RF_{CR} \times RF_D$

and,

T_{MARV} = The minimum average roll value for the ultimate geosynthetic strength T_{ult}

CR_u = The ultimate connection strength $T_{ultconn}$ divided by the lot specific ultimate tensile strength, T_{lot} (i.e., the lot of material specific to the connection testing)

RF_{ID} = Reduction factor for installation damage

RF_{CR} = Creep reduction factor for the geosynthetic

RF_D = The durability reduction factor for the geosynthetic

Approved details for the Tensar ARES wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- For all plan sheets, the full height panel details are not preapproved. Full height panels may be used by special design, with the approval of the State Bridge Design Engineer and the State Geotechnical Engineer, provided a complete wall design with detailed plans are developed and included in the construction contract (i.e., full height panel walls shall not be submitted as shop drawings in design-bid-build projects).
- In plan sheet 3 of 19, there should be a minimum cover of 4 inches of soil between the geogrid and the traffic barrier reaction slab.
- In plan sheet 8 of 19, the strength of the geogrid and connection available shall be reduced by 10% to account for the skew of the geogrid reinforcement. The skew angle relative to the perpendicular from the wall face shall be no more than 10°.
- In plan sheets 10 and 14 of 19, regarding the filter fabric shown, WSDOT reserves the right to require the use *Standard Specification* materials as specified in *Standard Specification* Section 9-33 that are similar to those specified in this plan sheet.
- In plan sheet 15 of 19, the guard rail detail, the guard rail post shall either be installed through precut holes in the geogrid layers that must penetrated, or the geogrid layers shall be cut in a manner that prevents ripping or tearing of the geogrid.
- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 2 feet for culvert penetration through the face and up to 4 feet for obstruction avoidance. Larger diameter culverts or obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers.

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CONSTRUCTION DRAWINGS
Prepared For

STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION

STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS

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4.	ARES Full Height Panel Cross–Section
5.	ARES Articulated Panels
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15.	Typical Details
16.	32 Inch Type "F" Traffic Barrier Standard
17.	32 Inch Type "F" Traffic Barrier Standard
18.	42 Inch Type "F" Traffic Barrier Standard
19.	42 Inch Type "F" Traffic Barrier Standard

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Drawn by KJK
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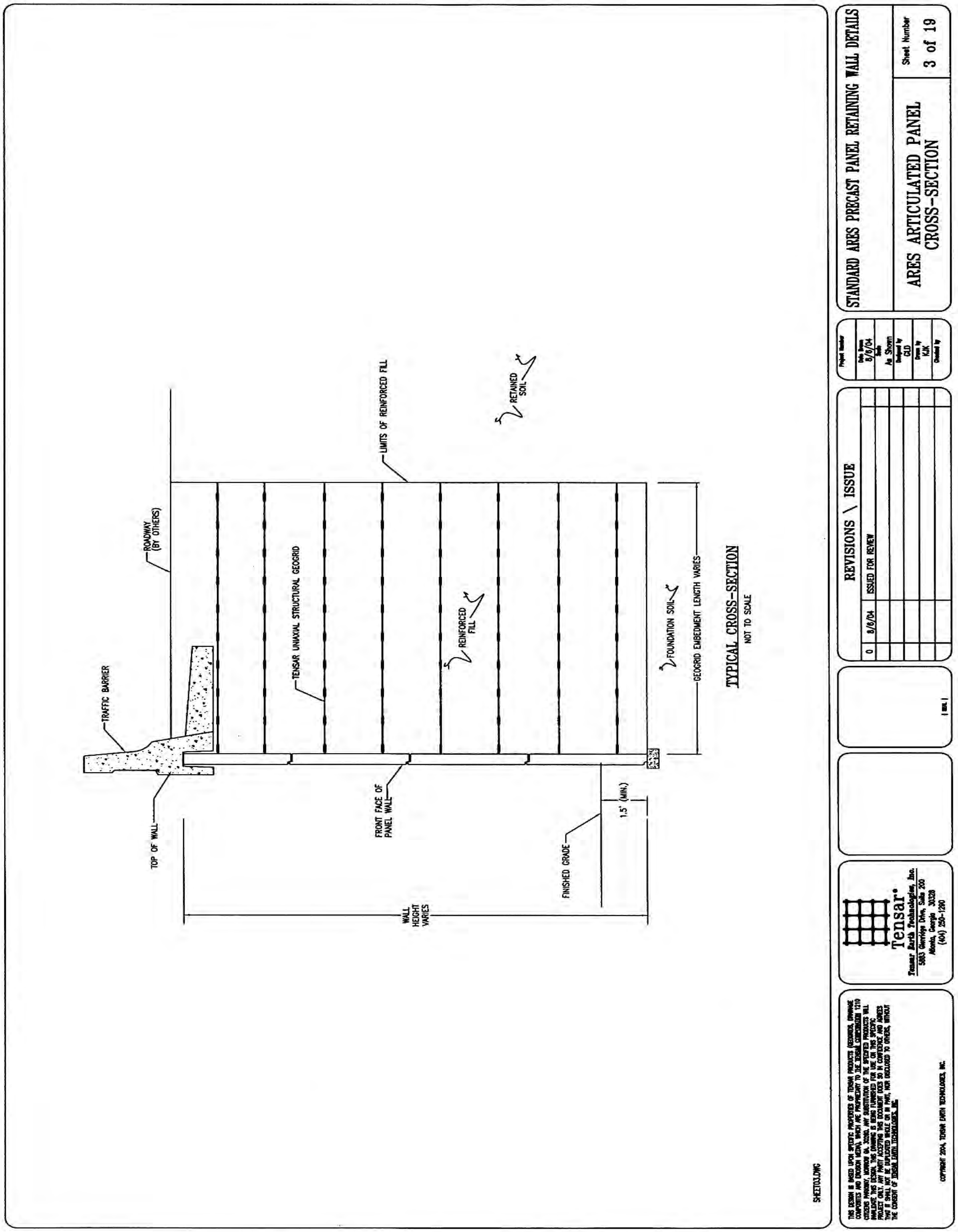
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS

TITLE SHEET

Sheet Number 1 of 19


TRAFFIC BARRIER
ROADWAY (BY OTHERS)
TENSAR UNIAXIAL STRUCTURAL GEOGRID
LIMITS OF REINFORCED FILL
RETAINED SOIL
REINFORCED FILL
FOUNDATION SOIL
GEOGRID EMBEDMENT LENGTH VARIES
TYPICAL CROSS-SECTION
NOT TO SCALE
TOP OF WALL
FRONT FACE OF PANEL WALL
1.5' (MIN.)
FINISHED GRADE
WALL HEIGHT VARIES
SHEET03.DWG
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS
ARES ARTICULATED PANEL CROSS-SECTION
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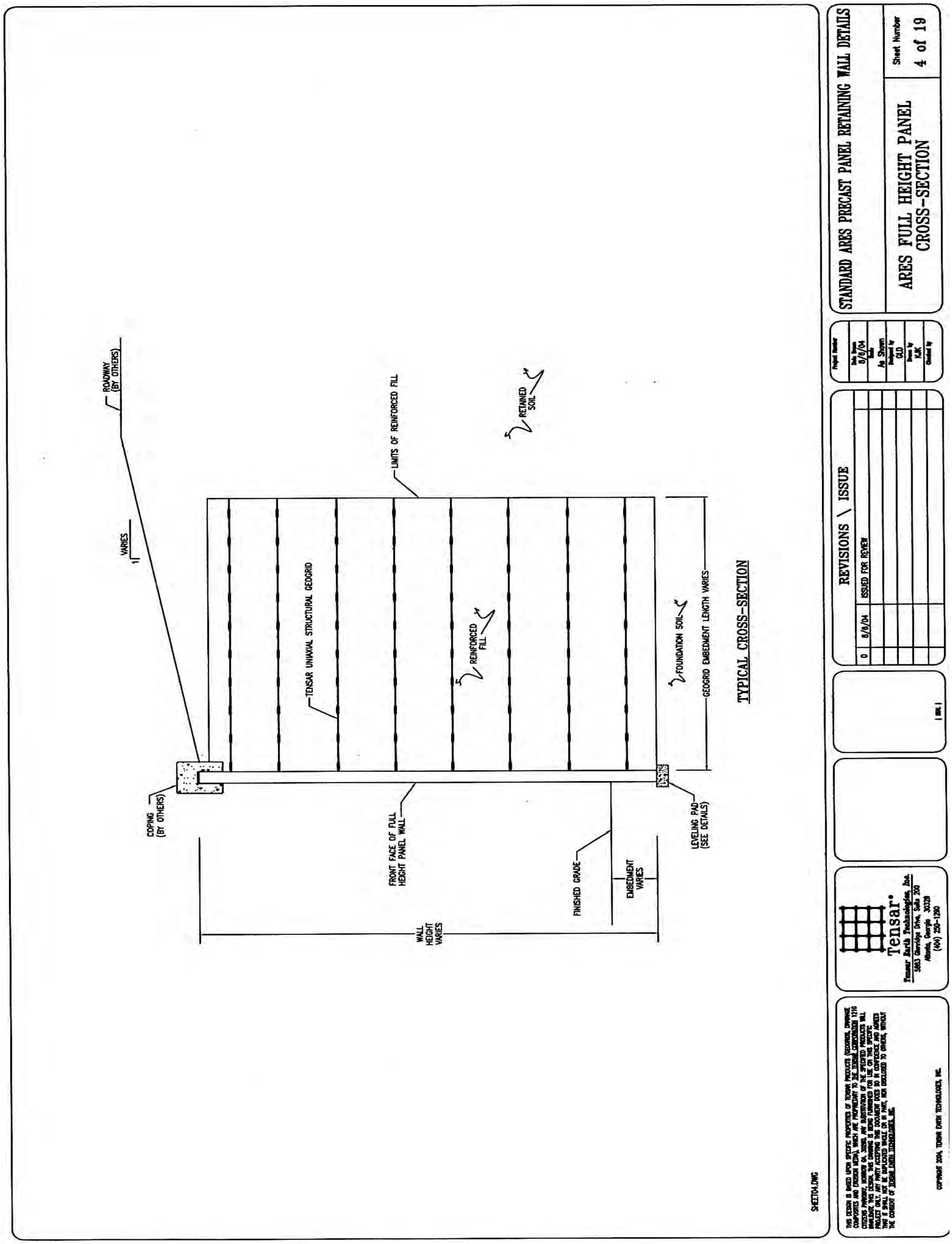
ROADWAY (BY OTHERS)
VARIES
1
LIMITS OF REINFORCED FILL
RETAINED SOIL
TENSAR UNIAXIAL STRUCTURAL GEOGRID
REINFORCED FILL
FOUNDATION SOIL
GEOGRID EMBEDMENT LENGTH VARIES
TYPICAL CROSS-SECTION
COPING (BY OTHERS)
FRONT FACE OF FULL HEIGHT PANEL WALL
FINISHED GRADE
EMBEDMENT VARIES
LEVELING PAD (SEE DETAILS)
WALL HEIGHT VARIES
SHEET04.DWG
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS
ARES FULL HEIGHT PANEL CROSS-SECTION
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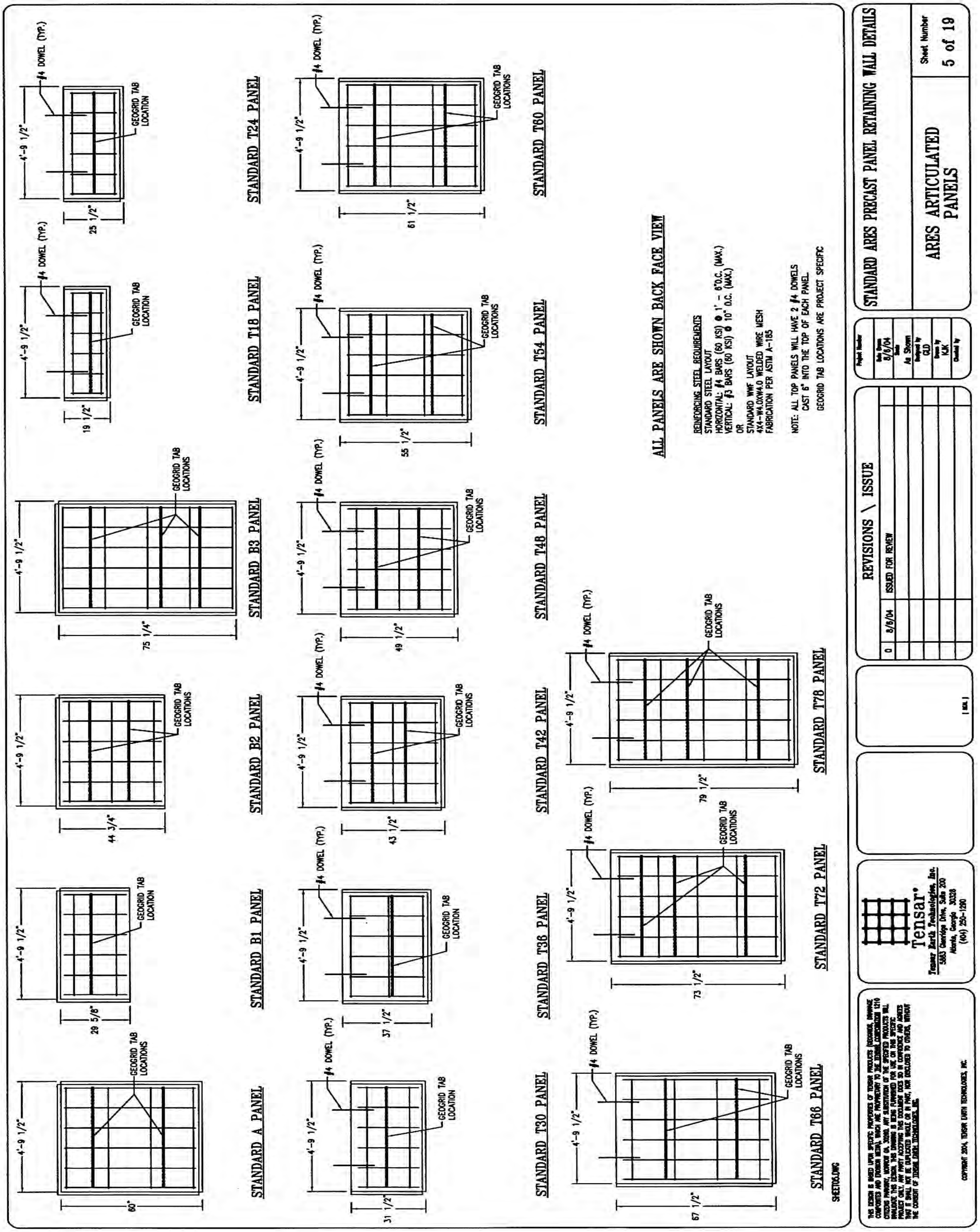
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS
ARES ARTICULATED PANELS
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STANDARD A PANEL
STANDARD B1 PANEL
STANDARD B2 PANEL
STANDARD B3 PANEL
STANDARD T18 PANEL
STANDARD T24 PANEL
STANDARD T30 PANEL
STANDARD T36 PANEL
STANDARD T42 PANEL
STANDARD T48 PANEL
STANDARD T54 PANEL
STANDARD T60 PANEL
STANDARD T66 PANEL
STANDARD T72 PANEL
STANDARD T78 PANEL
GEOGRID TAB LOCATIONS
#4 DOWEL (TYP.)
4'-9 1/2"
ALL PANELS ARE SHOWN BACK FACE VIEW
REINFORCING STEEL REQUIREMENTS
STANDARD STEEL LAYOUT
HORIZONTAL: #4 BARS (60 KSI) @ 1' - 6"O.C. (MAX.)
VERTICAL: #3 BARS (60 KSI) @ 10" O.C. (MAX.)
OR
STANDARD WWF LAYOUT
4X4-W4.0XW4.0 WELDED WIRE MESH
FABRICATION PER ASTM A-185
NOTE: ALL TOP PANELS WILL HAVE 2 #4 DOWELS CAST 6" INTO THE TOP OF EACH PANEL.
GEOGRID TAB LOCATIONS ARE PROJECT SPECIFIC
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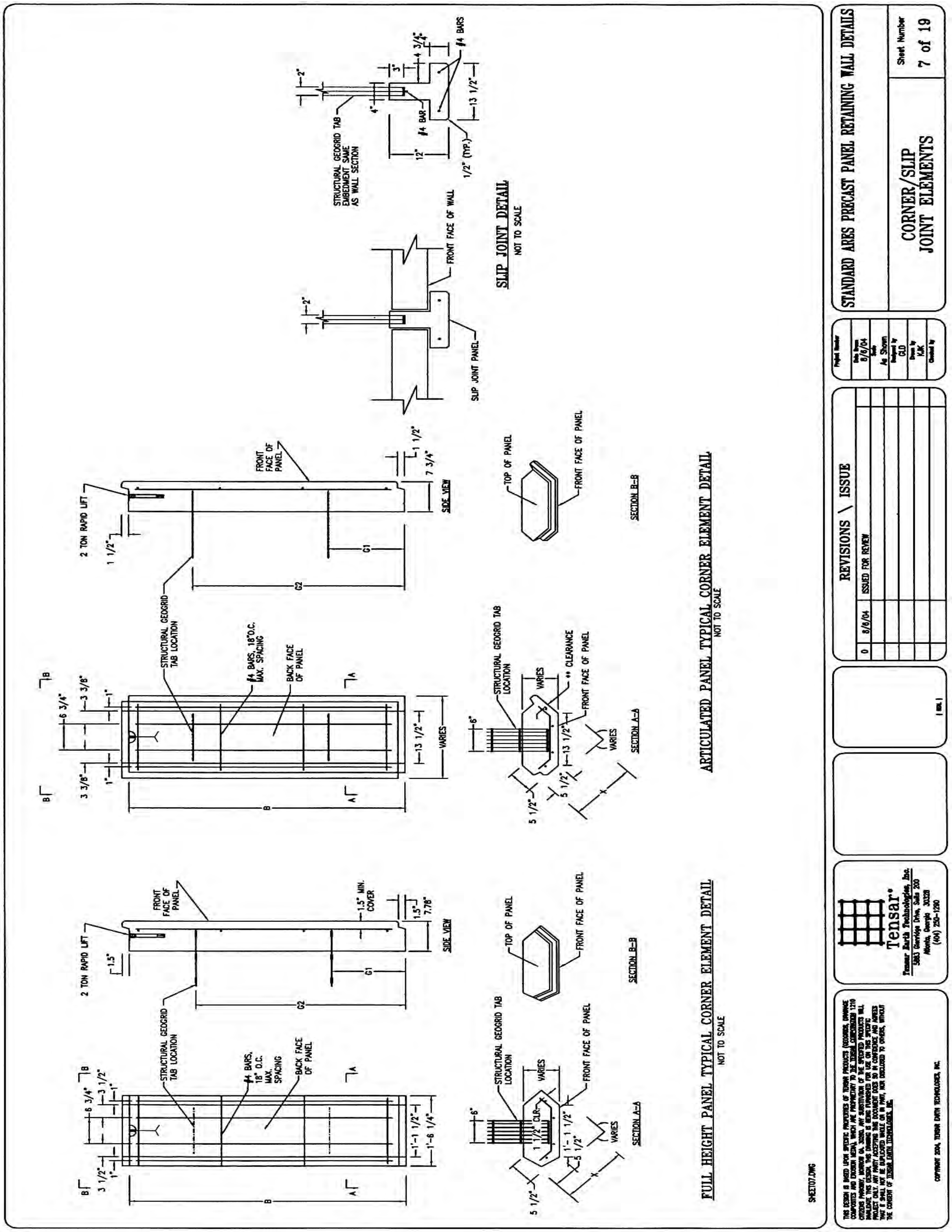
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS
CORNER/SLIP JOINT ELEMENTS
Sheet Number
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SLIP JOINT DETAIL
NOT TO SCALE
ARTICULATED PANEL TYPICAL CORNER ELEMENT DETAIL
NOT TO SCALE
FULL HEIGHT PANEL TYPICAL CORNER ELEMENT DETAIL
NOT TO SCALE
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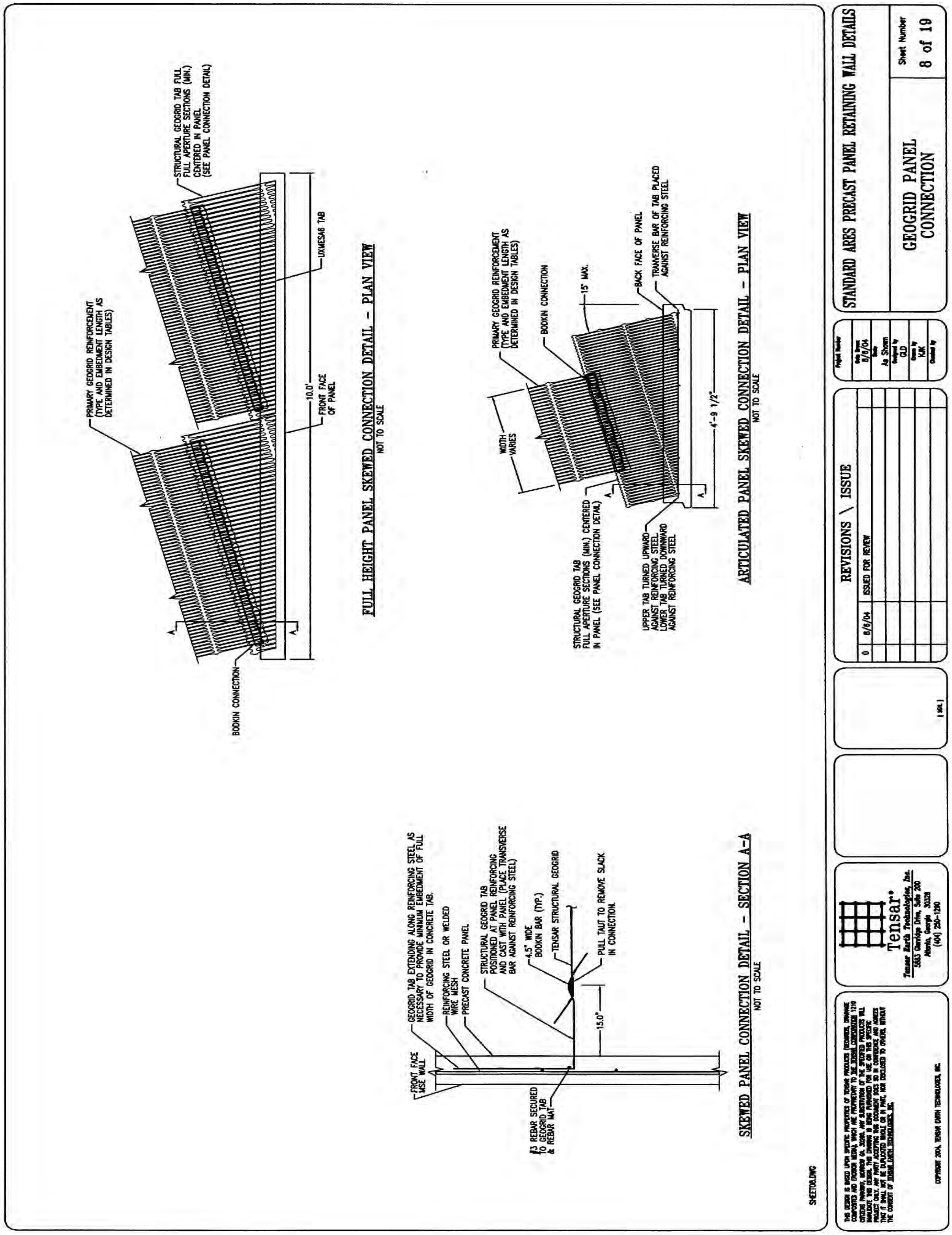
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS
GEOGRID PANEL CONNECTION
Sheet Number
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FULL HEIGHT PANEL SKEWED CONNECTION DETAIL - PLAN VIEW
NOT TO SCALE
ARTICULATED PANEL SKEWED CONNECTION DETAIL - PLAN VIEW
NOT TO SCALE
SKEWED PANEL CONNECTION DETAIL - SECTION A-A
NOT TO SCALE
STRUCTURAL GEOGRID TAB FULL FULL APERTURE SECTIONS (MIN.) CENTERED IN PANEL (SEE PANEL CONNECTION DETAIL)
PRIMARY GEOGRID REINFORCEMENT (TYPE AND EMBEDMENT LENGTH AS DETERMINED IN DESIGN TABLES)
UXMESA6 TAB
10.0'
FRONT FACE OF PANEL
BODKIN CONNECTION
WIDTH VARIES
15° MAX.
BACK FACE OF PANEL
TRANVERSE BAR OF TAB PLACED AGAINST REINFORCING STEEL
4'-9 1/2"
STRUCTURAL GEOGRID TAB FULL APERTURE SECTIONS (MIN.) CENTERED IN PANEL (SEE PANEL CONNECTION DETAIL)
UPPER TAB TURNED UPWARD AGAINST REINFORCING STEEL LOWER TAB TURNED DOWNWARD AGAINST REINFORCING STEEL
FRONT FACE MSE WALL
GEOGRID TAB EXTENDING ALONG REINFORCING STEEL AS NECESSARY TO PROVIDE MINIMUM EMBEDMENT OF FULL WIDTH OF GEOGRID IN CONCRETE TAB.
REINFORCING STEEL OR WELDED WIRE MESH
PRECAST CONCRETE PANEL
STRUCTURAL GEOGRID TAB POSITIONED AT PANEL REINFORCING AND CAST WITH PANEL (PLACE TRANSVERSE BAR AGAINST REINFORCING STEEL)
4.5" WIDE BODKIN BAR (TYP.)
TENSAR STRUCTURAL GEOGRID
PULL TAUT TO REMOVE SLACK IN CONNECTION.
15.0"
#3 REBAR SECURED TO GEOGRID TAB & REBAR MAT
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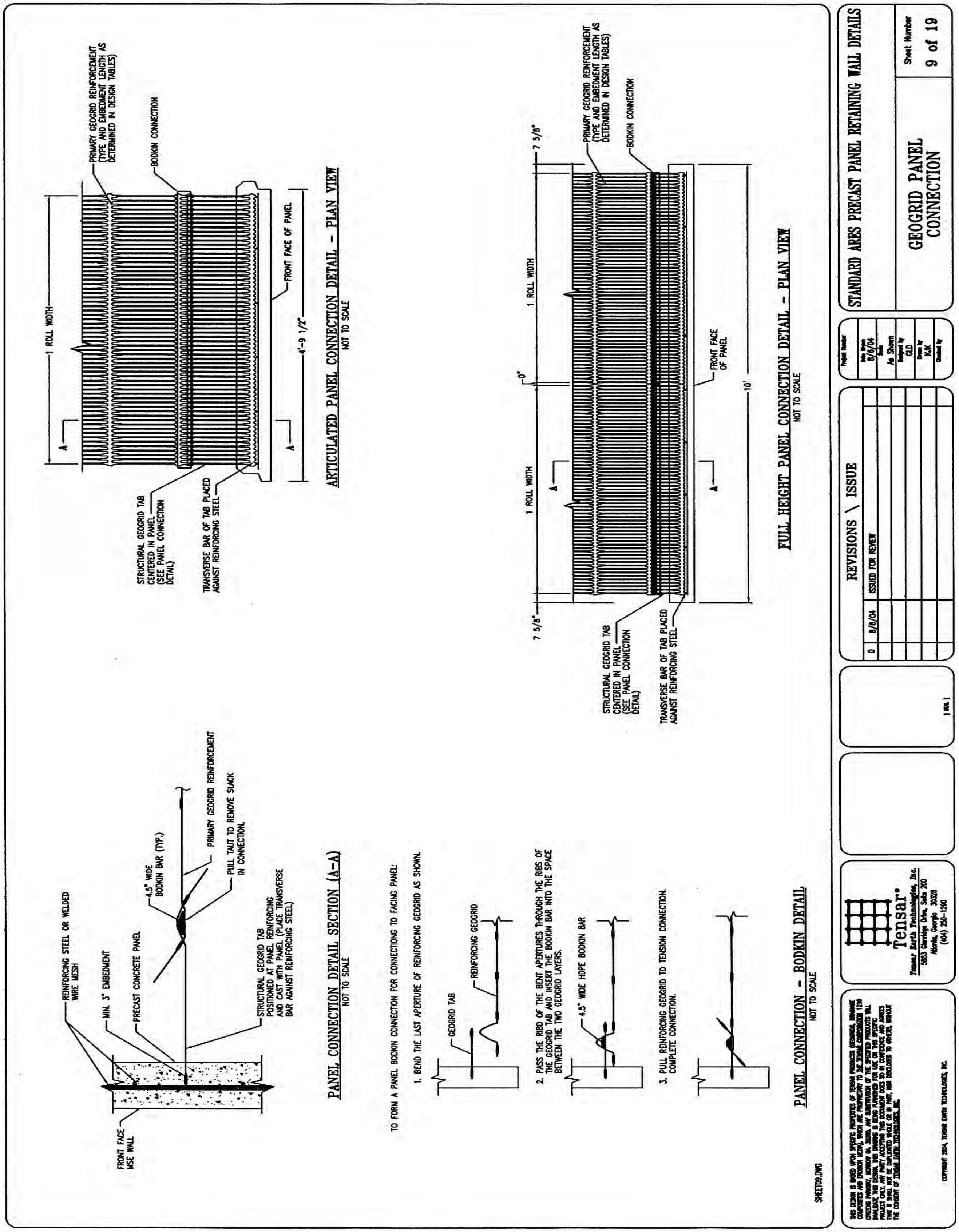
REINFORCING STEEL OR WELDED WIRE MESH
FRONT FACE MSE WALL
MIN. 3" EMBEDMENT
PRECAST CONCRETE PANEL
4.5" WIDE BODKIN BAR (TYP.)
PRIMARY GEOGRID REINFORCEMENT
PULL TAUT TO REMOVE SLACK IN CONNECTION.
STRUCTURAL GEOGRID TAB POSITIONED AT PANEL REINFORCING AND CAST WITH PANEL (PLACE TRANSVERSE BAR AGAINST REINFORCING STEEL)
PANEL CONNECTION DETAIL SECTION (A-A)
NOT TO SCALE
TO FORM A PANEL BODKIN CONNECTION FOR CONNECTIONG TO FACING PANEL:
1. BEND THE LAST APERTURE OF REINFORCING GEOGRID AS SHOWN.
GEOGRID TAB
REINFORCING GEOGRID
2. PASS THE RIBD OF THE BENT APERTURES THROUGH THE RIBS OF THE GEOGRID TAB AND INSERT THE BODKIN BAR INTO THE SPACE BETWEEN THE TWO GEOGRID LAYERS.
4.5" WIDE HDPE BODKIN BAR
3. PULL REINFORCING GEOGRID TO TENSION CONNECTION. COMPLETE CONNECTION.
PANEL CONNECTION - BODKIN DETAIL
NOT TO SCALE
SHEET09.DWG
1 ROLL WIDTH
A
PRIMARY GEOGRID REINFORCEMENT (TYPE AND EMBEDMENT LENGTH AS DETERMINED IN DESIGN TABLES)
STRUCTURAL GEOGRID TAB CENTERED IN PANEL (SEE PANEL CONNECTION DETAIL)
BODKIN CONNECTION
TRANSVERSE BAR OF TAB PLACED AGAINST REINFORCING STEEL
FRONT FACE OF PANEL
4'-9 1/2"
ARTICULATED PANEL CONNECTION DETAIL - PLAN VIEW
NOT TO SCALE
7 5/8"
0"
1 ROLL WIDTH
1 ROLL WIDTH
7 5/8"
FRONT FACE OF PANEL
10'
FULL HEIGHT PANEL CONNECTION DETAIL - PLAN VIEW
NOT TO SCALE
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Designed by
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Drawn by
KJK
Checked by
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS
GEOGRID PANEL CONNECTION
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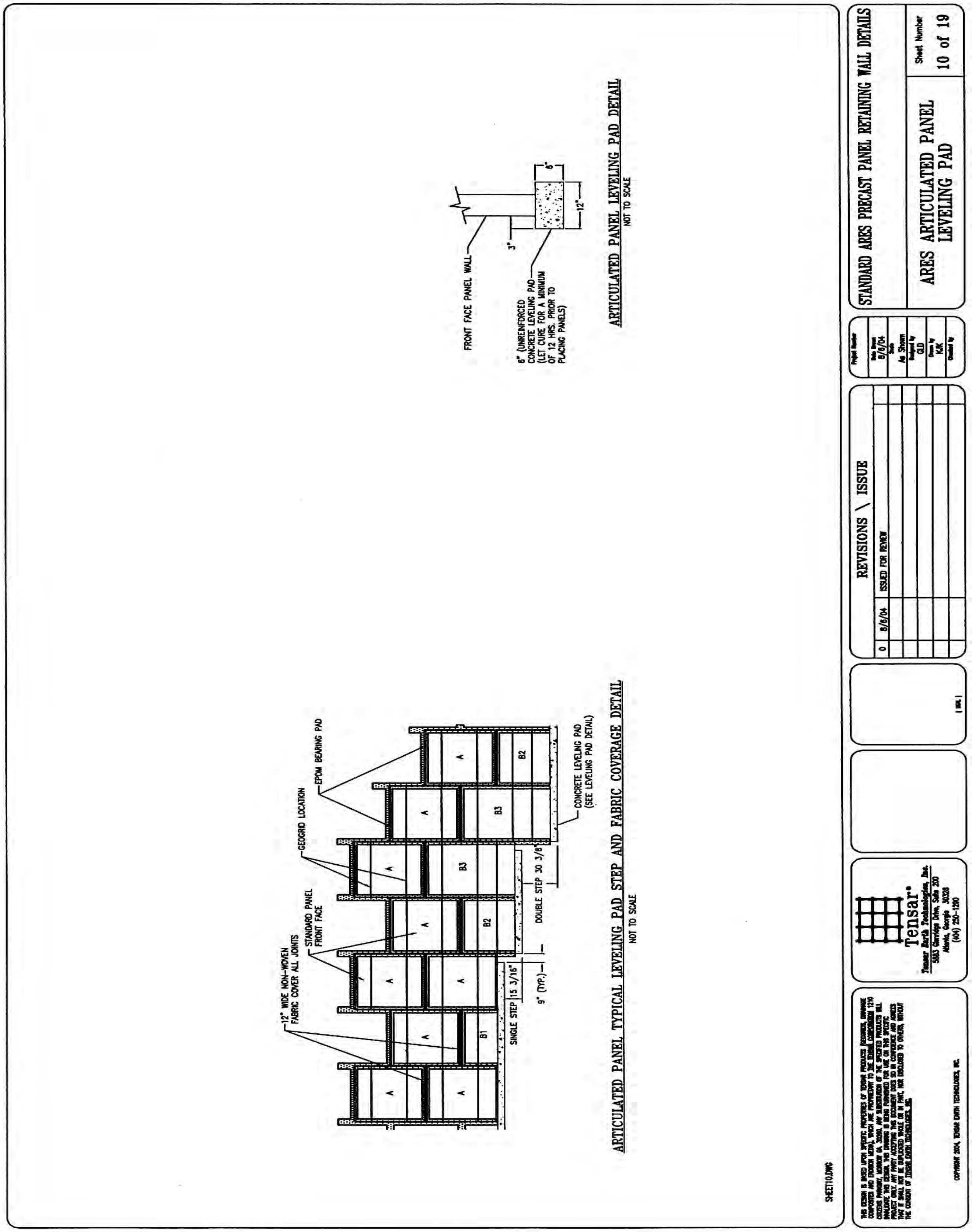

ARTICULATED PANEL LEVELING PAD DETAIL
NOT TO SCALE
FRONT FACE PANEL WALL
6" (UNREINFORCED CONCRETE LEVELING PAD (LET CURE FOR A MINIMUM OF 12 HRS. PRIOR TO PLACING PANELS)
3"
12"
6"
ARTICULATED PANEL TYPICAL LEVELING PAD STEP AND FABRIC COVERAGE DETAIL
NOT TO SCALE
12" WIDE NON-WOVEN FABRIC COVER ALL JOINTS
STANDARD PANEL FRONT FACE
GEOGRID LOCATION
EPDM BEARING PAD
CONCRETE LEVELING PAD (SEE LEVELING PAD DETAIL)
SINGLE STEP 15 3/16"
9" (TYP.)
DOUBLE STEP 30 3/8"
A
B1
B2
B3
SHEET10.DWG
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STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS
ARES ARTICULATED PANEL LEVELING PAD
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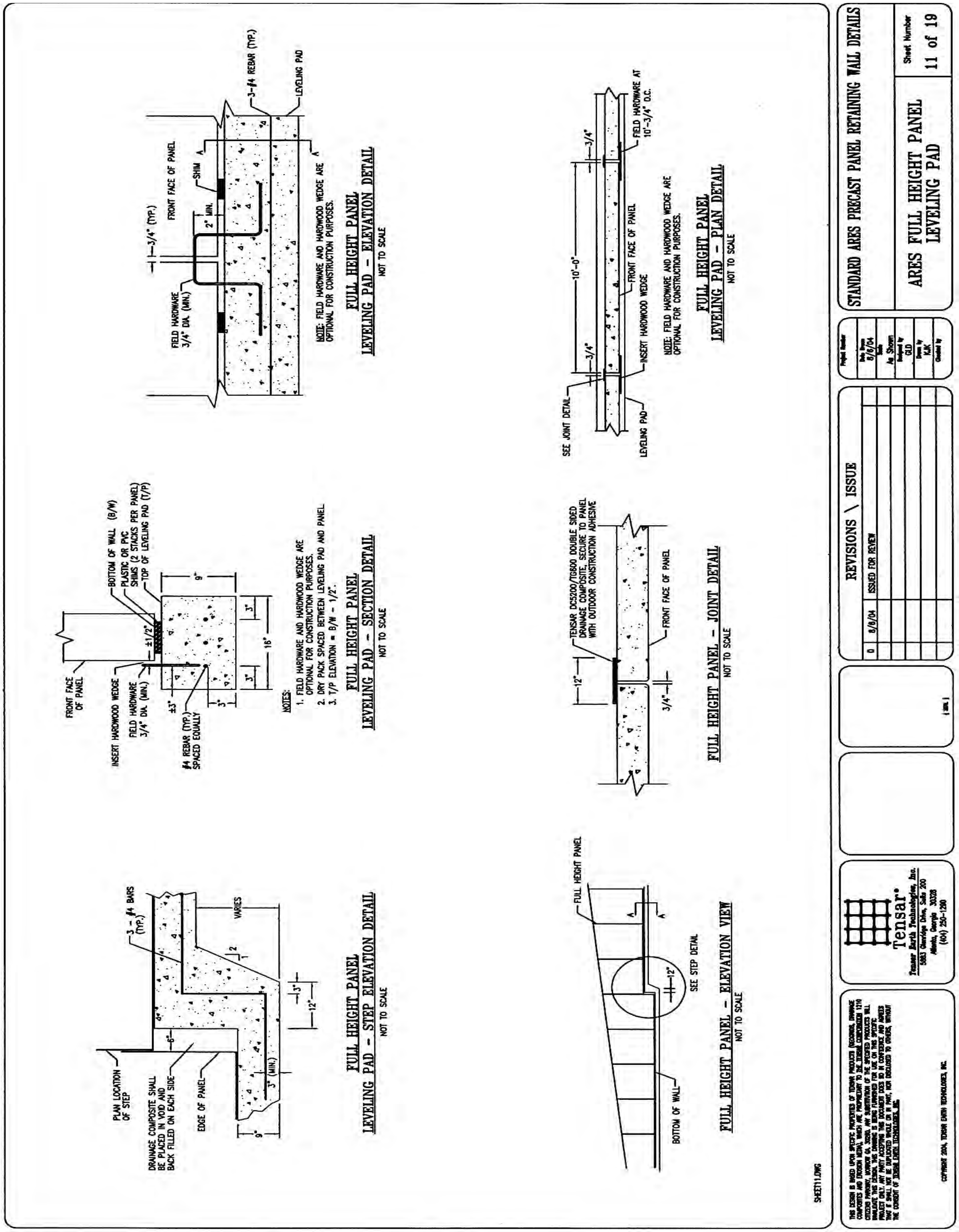
FULL HEIGHT PANEL
LEVELING PAD – ELEVATION DETAIL
NOT TO SCALE
NOTE: FIELD HARDWARE AND HARDWOOD WEDGE ARE OPTIONAL FOR CONSTRUCTION PURPOSES.
FIELD HARDWARE 3/4" DIA. (MIN.)
3/4" (TYP.)
FRONT FACE OF PANEL
SHIM
2" MIN.
3-#4 REBAR (TYP.)
LEVELING PAD
FULL HEIGHT PANEL
LEVELING PAD – SECTION DETAIL
NOT TO SCALE
NOTES:
1. FIELD HARDWARE AND HARDWOOD WEDGE ARE OPTIONAL FOR CONSTRUCTION PURPOSES.
2. DRY PACK SPACED BETWEEN LEVELING PAD AND PANEL
3. T/P ELEVATION = B/W – 1/2".
FRONT FACE OF PANEL
INSERT HARDWOOD WEDGE
FIELD HARDWARE 3/4" DIA. (MIN.)
#4 REBAR (TYP.) SPACED EQUALLY
BOTTOM OF WALL (B/W)
PLASTIC OR PVC SHIMS (2 STACKS PER PANEL)
TOP OF LEVELING PAD (T/P)
FULL HEIGHT PANEL
LEVELING PAD – STEP ELEVATION DETAIL
NOT TO SCALE
PLAN LOCATION OF STEP
DRAINAGE COMPOSITE SHALL BE PLACED IN VOID AND BACK FILLED ON EACH SIDE
EDGE OF PANEL
3 – #4 BARS (TYP.)
VARIES
3" (MIN.)
FULL HEIGHT PANEL
LEVELING PAD – PLAN DETAIL
NOT TO SCALE
SEE JOINT DETAIL
LEVELING PAD
INSERT HARDWOOD WEDGE
FRONT FACE OF PANEL
FIELD HARDWARE AT 10'-3/4" O.C.
10'-0"
FULL HEIGHT PANEL – JOINT DETAIL
NOT TO SCALE
TENSAR DC5200/TG500 DOUBLE SIDED DRAINAGE COMPOSITE, SECURE TO PANEL WITH OUTDOOR CONSTRUCTION ADHESIVE
FRONT FACE OF PANEL
FULL HEIGHT PANEL – ELEVATION VIEW
NOT TO SCALE
FULL HEIGHT PANEL
BOTTOM OF WALL
SEE STEP DETAIL
SHEET11.DWG
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STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS
ARES FULL HEIGHT PANEL LEVELING PAD
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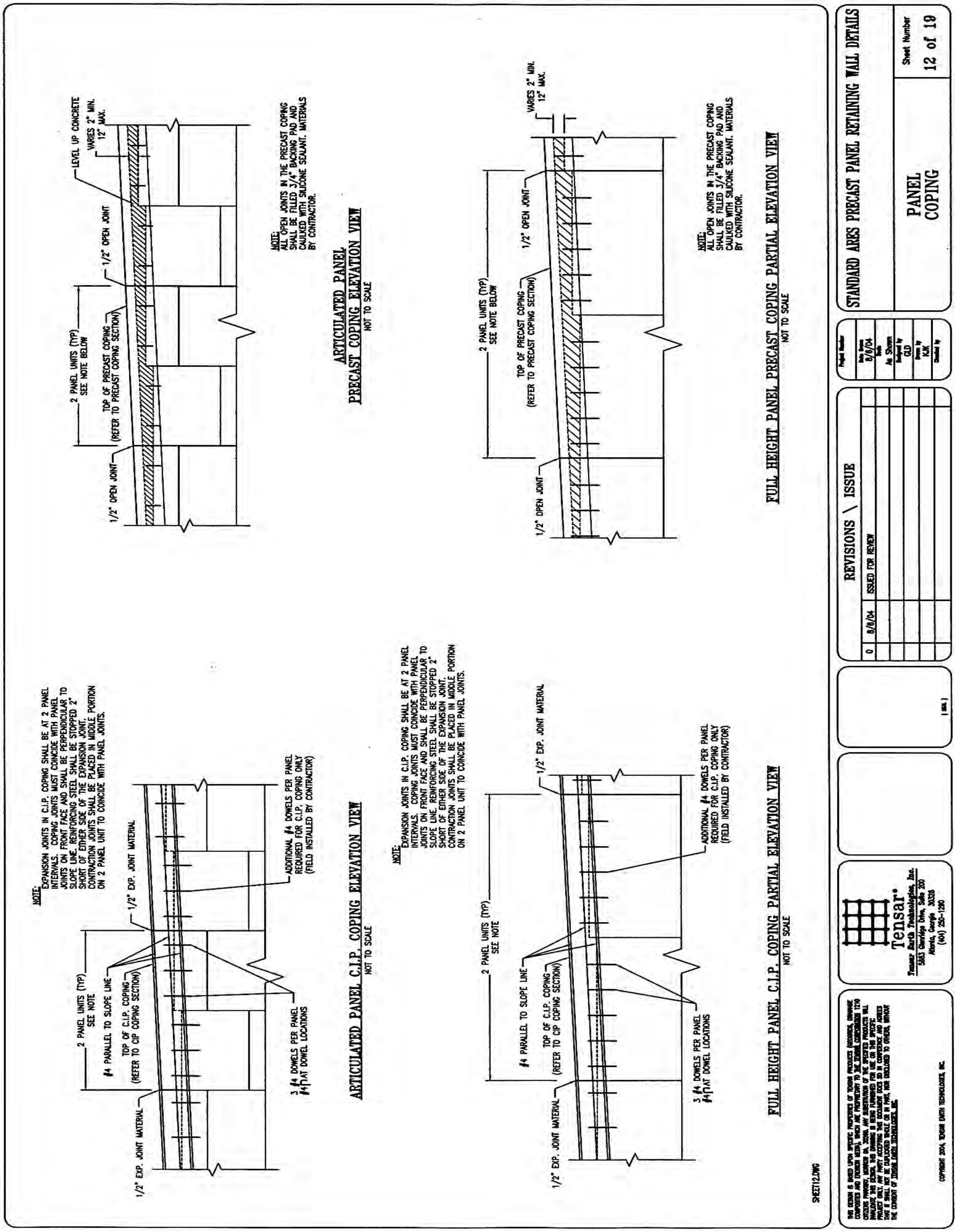
ARTICULATED PANEL PRECAST COPING ELEVATION VIEW
NOT TO SCALE
FULL HEIGHT PANEL PRECAST COPING PARTIAL ELEVATION VIEW
NOT TO SCALE
ARTICULATED PANEL C.I.P. COPING ELEVATION VIEW
NOT TO SCALE
FULL HEIGHT PANEL C.I.P. COPING PARTIAL ELEVATION VIEW
NOT TO SCALE
LEVEL UP CONCRETE
VARIES 2" MIN. 12" MAX.
1/2" OPEN JOINT
2 PANEL UNITS (TYP) SEE NOTE BELOW
TOP OF PRECAST COPING (REFER TO PRECAST COPING SECTION)
NOTE: ALL OPEN JOINTS IN THE PRECAST COPING SHALL BE FILLED 3/4" BACKING PAD AND CAULKED WITH SILICONE SEALANT. MATERIALS BY CONTRACTOR.
NOTE: EXPANSION JOINTS IN C.I.P. COPING SHALL BE AT 2 PANEL INTERVALS. COPING JOINTS MUST COINCIDE WITH PANEL JOINTS ON FRONT FACE AND SHALL BE PERPENDICULAR TO SLOPE LINE. REINFORCING STEEL SHALL BE STOPPED 2" SHORT OF EITHER SIDE OF THE EXPANSION JOINT. CONTRACTION JOINTS SHALL BE PLACED IN MIDDLE PORTION ON 2 PANEL UNIT TO COINCIDE WITH PANEL JOINTS.
1/2" EXP. JOINT MATERIAL
2 PANEL UNITS (TYP) SEE NOTE
#4 PARALLEL TO SLOPE LINE
TOP OF C.I.P. COPING (REFER TO CIP COPING SECTION)
3 #4 DOWELS PER PANEL #4 AT DOWEL LOCATIONS
ADDITIONAL #4 DOWELS PER PANEL REQUIRED FOR C.I.P. COPING ONLY (FIELD INSTALLED BY CONTRACTOR)
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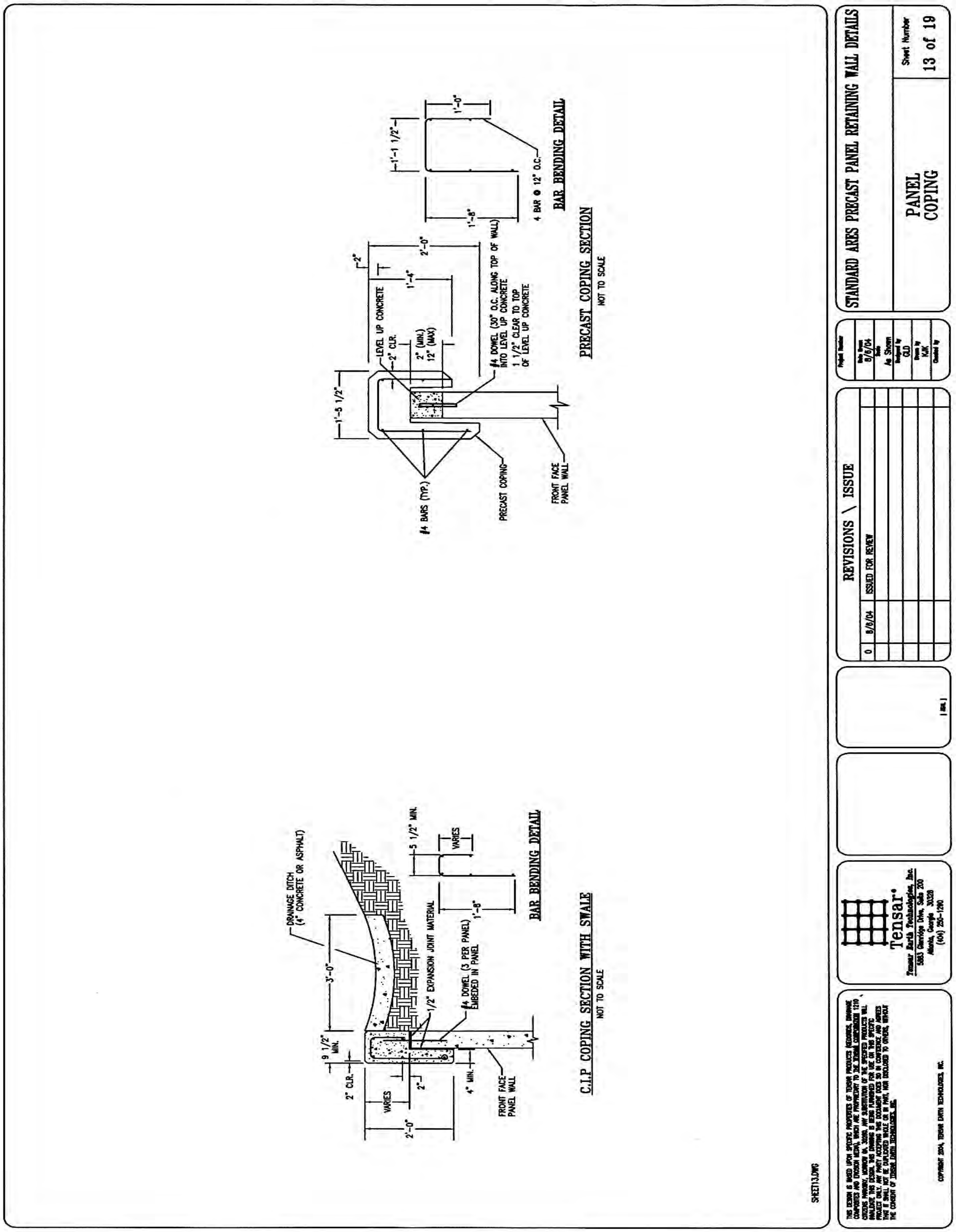
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PANEL
COPING
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Date Drawn
8/6/04
Scale
As Shown
Designed by
G.D
Drawn by
K.K
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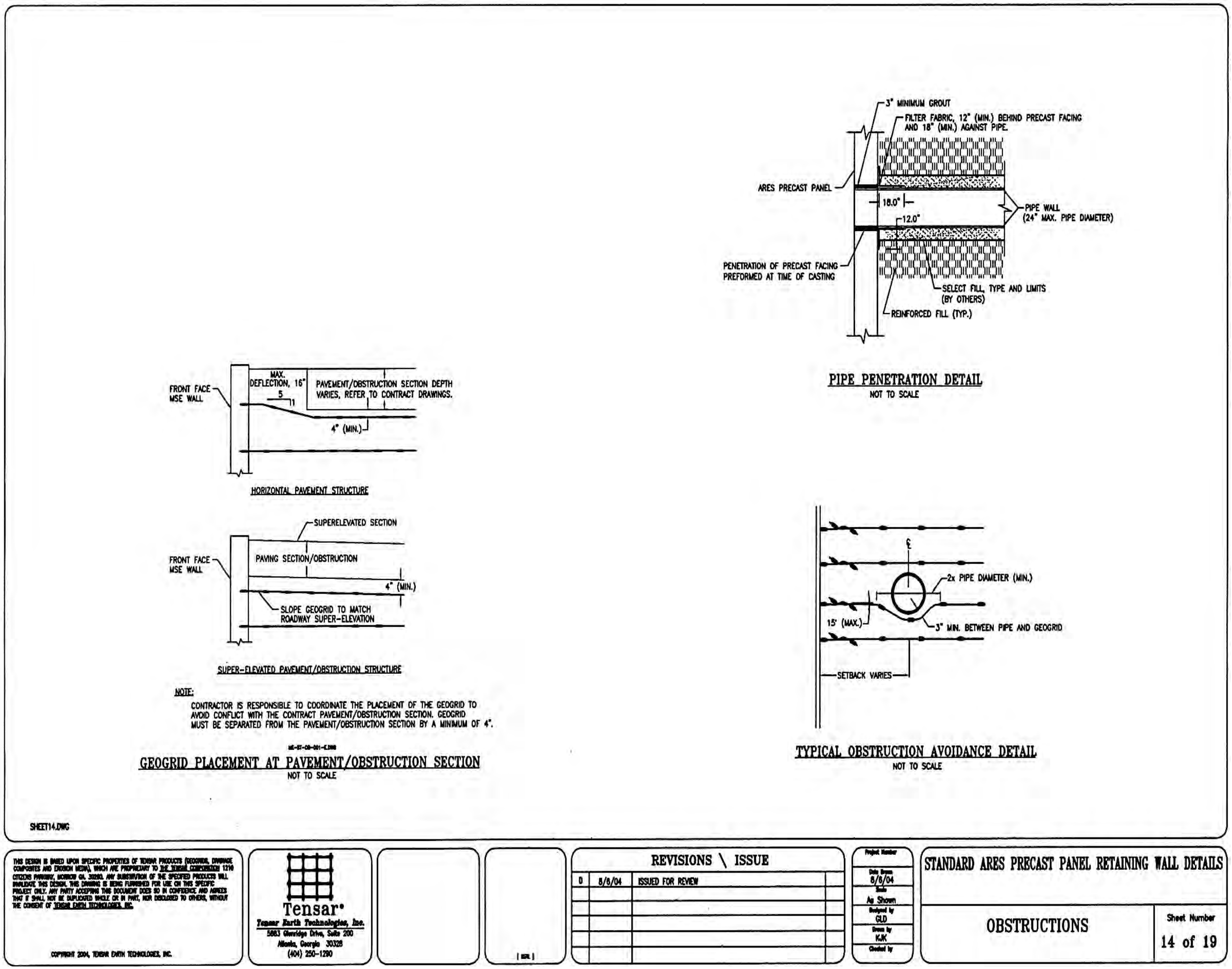
PRECAST COPING SECTION
NOT TO SCALE
#4 BARS (TYP.)
PRECAST COPING
FRONT FACE PANEL WALL
1'-5 1/2"
LEVEL UP CONCRETE
2" CLR.
2" (MIN.) 12" (MAX)
#4 DOWEL (30" O.C. ALONG TOP OF WALL) INTO LEVEL UP CONCRETE 1 1/2" CLEAR TO TOP OF LEVEL UP CONCRETE
2"
1'-4"
2'-0"
BAR BENDING DETAIL
1'-1 1/2"
1'-0"
1'-8"
4 BAR @ 12" O.C.
C.I.P COPING SECTION WITH SWALE
NOT TO SCALE
DRAINAGE DITCH (4" CONCRETE OR ASPHALT)
3'-0"
9 1/2" MIN.
2" CLR.
VARIES
2'-0"
2"
4" MIN.
FRONT FACE PANEL WALL
1/2" EXPANSION JOINT MATERIAL
#4 DOWEL (3 PER PANEL) EMBEDED IN PANEL
BAR BENDING DETAIL
5 1/2" MIN.
VARIES
1'-8"


3" MINIMUM GROUT
FILTER FABRIC, 12" (MIN.) BEHIND PRECAST FACING AND 18" (MIN.) AGAINST PIPE.
ARES PRECAST PANEL
18.0"
12.0"
PIPE WALL (24" MAX. PIPE DIAMETER)
PENETRATION OF PRECAST FACING PREFORMED AT TIME OF CASTING
SELECT FILL, TYPE AND LIMITS (BY OTHERS)
REINFORCED FILL (TYP.)
PIPE PENETRATION DETAIL
NOT TO SCALE
FRONT FACE MSE WALL
MAX. DEFLECTION, 16"
5
1
PAVEMENT/OBSTRUCTION SECTION DEPTH VARIES, REFER TO CONTRACT DRAWINGS.
4" (MIN.)
HORIZONTAL PAVEMENT STRUCTURE
SUPERELEVATED SECTION
FRONT FACE MSE WALL
PAVING SECTION/OBSTRUCTION
4" (MIN.)
SLOPE GEOGRID TO MATCH ROADWAY SUPER-ELEVATION
SUPER-ELEVATED PAVEMENT/OBSTRUCTION STRUCTURE
NOTE:
CONTRACTOR IS RESPONSIBLE TO COORDINATE THE PLACEMENT OF THE GEOGRID TO AVOID CONFLICT WITH THE CONTRACT PAVEMENT/OBSTRUCTION SECTION. GEOGRID MUST BE SEPARATED FROM THE PAVEMENT/OBSTRUCTION SECTION BY A MINIMUM OF 4".
GEOGRID PLACEMENT AT PAVEMENT/OBSTRUCTION SECTION
NOT TO SCALE
℄
2x PIPE DIAMETER (MIN.)
15' (MAX.)
3" MIN. BETWEEN PIPE AND GEOGRID
SETBACK VARIES
TYPICAL OBSTRUCTION AVOIDANCE DETAIL
NOT TO SCALE
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Designed by GLD
Drawn by KJK
Checked by
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OBSTRUCTIONS
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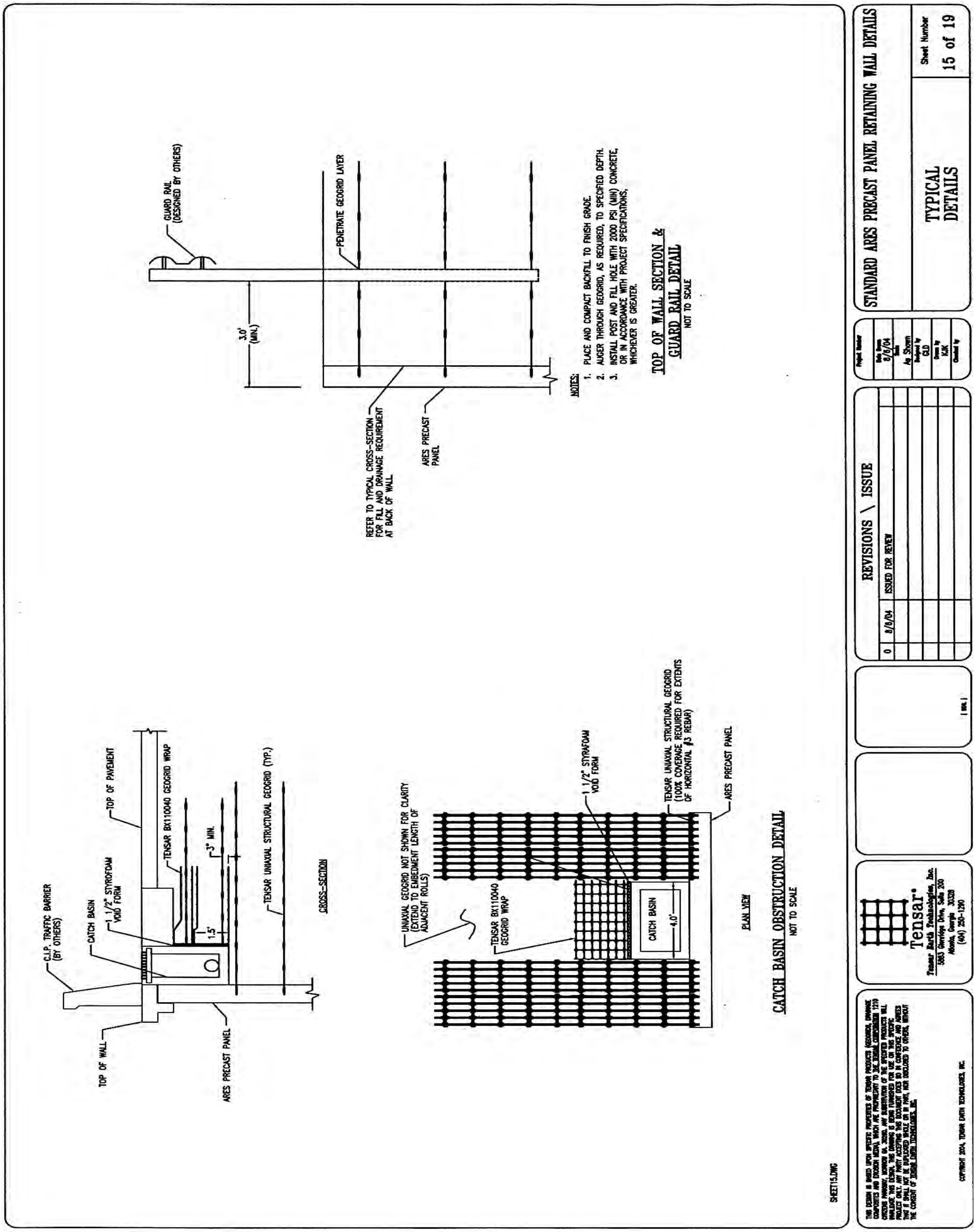

C.I.P. TRAFFIC BARRIER (BY OTHERS)
TOP OF WALL
CATCH BASIN
1 1/2" STYROFOAM VOID FORM
TOP OF PAVEMENT
TENSAR BX110040 GEOGRID WRAP
3" MIN.
1.5'
ARES PRECAST PANEL
TENSAR UNIAXIAL STRUCTURAL GEOGRID (TYP.)
CROSS-SECTION
UNIAXIAL GEOGRID NOT SHOWN FOR CLARITY (EXTEND TO EMBEDMENT LENGTH OF ADJACENT ROLLS)
TENSAR BX110040 GEOGRID WRAP
CATCH BASIN
4.0'
TENSAR UNIAXIAL STRUCTURAL GEOGRID (100% COVERAGE REQUIRED FOR EXTENTS OF HORIZONTAL #3 REBAR)
ARES PRECAST PANEL
PLAN VIEW
CATCH BASIN OBSTRUCTION DETAIL
NOT TO SCALE
GUARD RAIL (DESIGNED BY OTHERS)
3.0' (MIN.)
PENETRATE GEOGRID LAYER
REFER TO TYPICAL CROSS-SECTION FOR FILL AND DRAINAGE REQUIREMENT AT BACK OF WALL
ARES PRECAST PANEL
NOTES:
1. PLACE AND COMPACT BACKFILL TO FINISH GRADE.
2. AUGER THROUGH GEOGRID, AS REQUIRED, TO SPECIFIED DEPTH.
3. INSTALL POST AND FILL HOLE WITH 2000 PSI (MIN) CONCRETE, OR IN ACCORDANCE WITH PROJECT SPECIFICATIONS, WHICHEVER IS GREATER.
TOP OF WALL SECTION & GUARD RAIL DETAIL
NOT TO SCALE
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TYPICAL DETAILS
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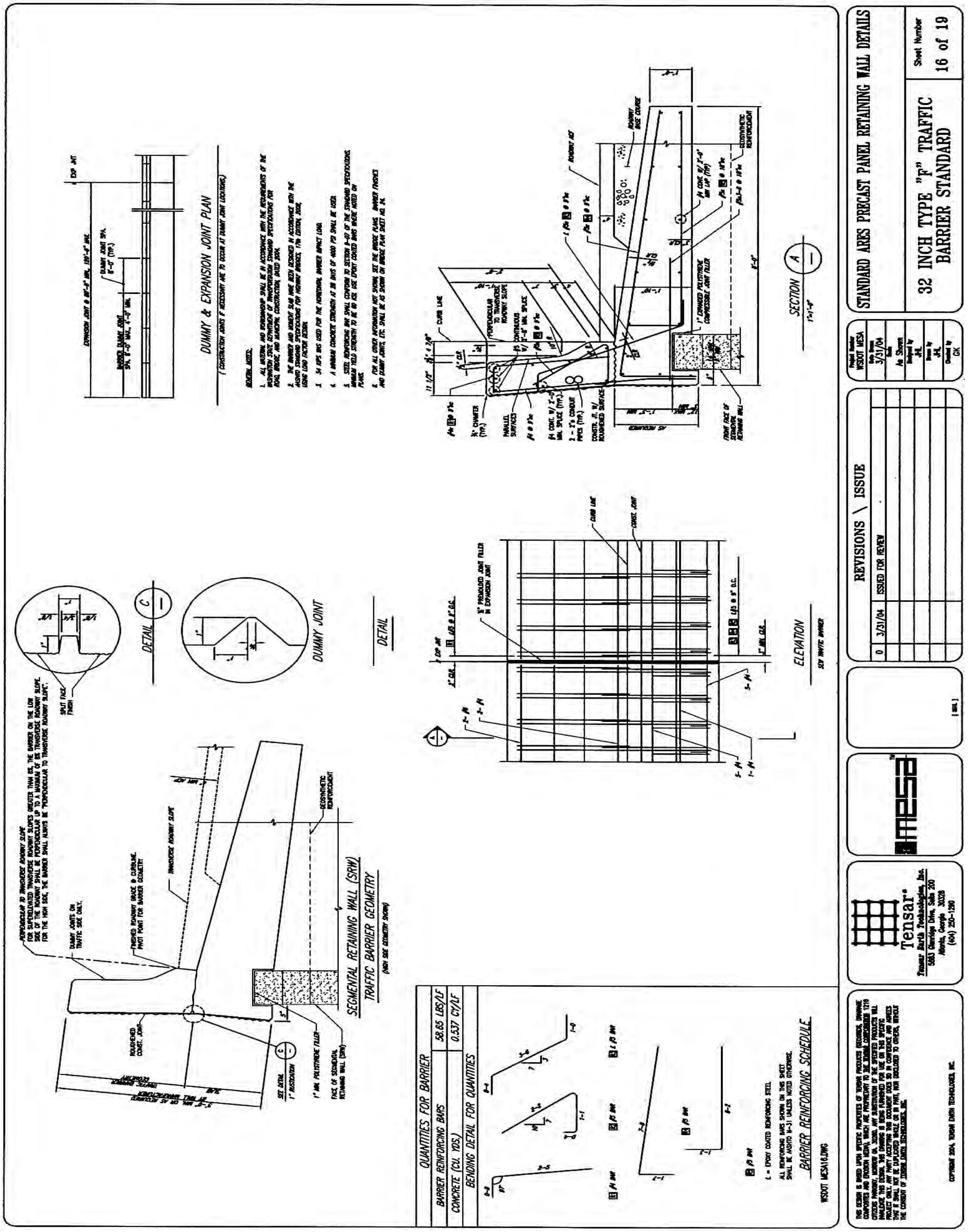
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS
32 INCH TYPE "F" TRAFFIC BARRIER STANDARD
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DUMMY & EXPANSION JOINT PLAN
SECTION A
DETAIL C
DUMMY JOINT
DETAIL
ELEVATION
SEGMENTAL RETAINING WALL (SRW)
TRAFFIC BARRIER GEOMETRY
QUANTITIES FOR BARRIER
BARRIER REINFORCING BARS 58.65 LBS/LF
CONCRETE (CU. YDS) 0.537 CY/LF
BENDING DETAIL FOR QUANTITIES
BARRIER REINFORCING SCHEDULE
WSDOT MESA16.DWG
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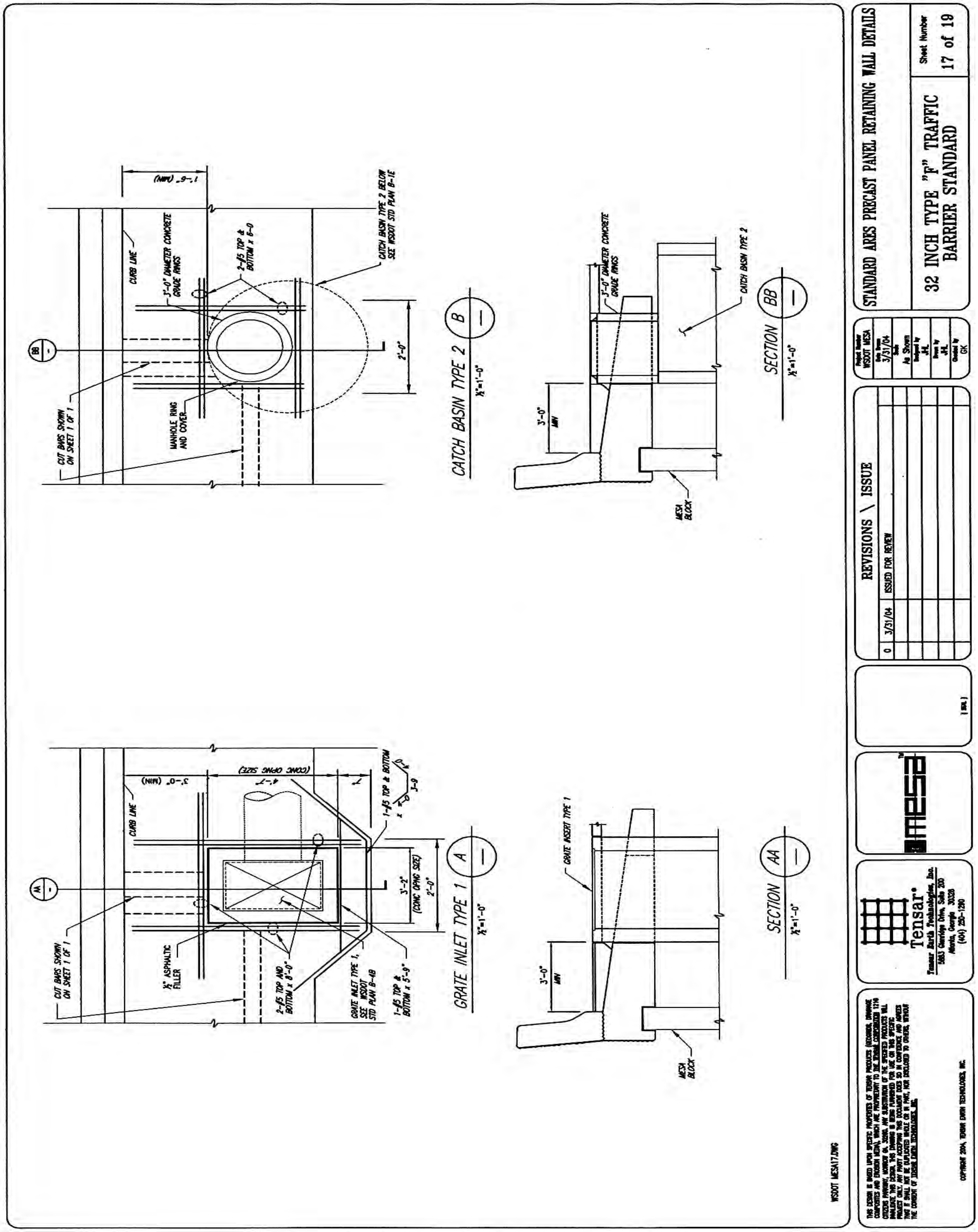

STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS
32 INCH TYPE "F" TRAFFIC BARRIER STANDARD
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CATCH BASIN TYPE 2
B
GRATE INLET TYPE 1
A
SECTION BB
SECTION AA
CURB LINE
CUT BARS SHOWN ON SHEET 1 OF 1
MANHOLE RING AND COVER
3'-0" DIAMETER CONCRETE GRADE RINGS
2-#5 TOP & BOTTOM x 6-0
CATCH BASIN TYPE 2 BELOW SEE WSDOT STD PLAN B-1E
1'-6" (MIN)
2'-0"
3'-0" MIN
MESA BLOCK
CATCH BASIN TYPE 2
GRATE INSERT TYPE 1
¾" ASPHALTIC FILLER
2-#5 TOP AND BOTTOM x 8-0"
GRATE INLET TYPE 1, SEE WSDOT STD PLAN B-4B
1-#5 TOP & BOTTOM x 5-9"
1-#5 TOP & BOTTOM
3'-0" (MIN)
(CONC OPNG SIZE)
3'-2"
2'-0"
WSDOT MESA17.DWG
Tensar
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mesa
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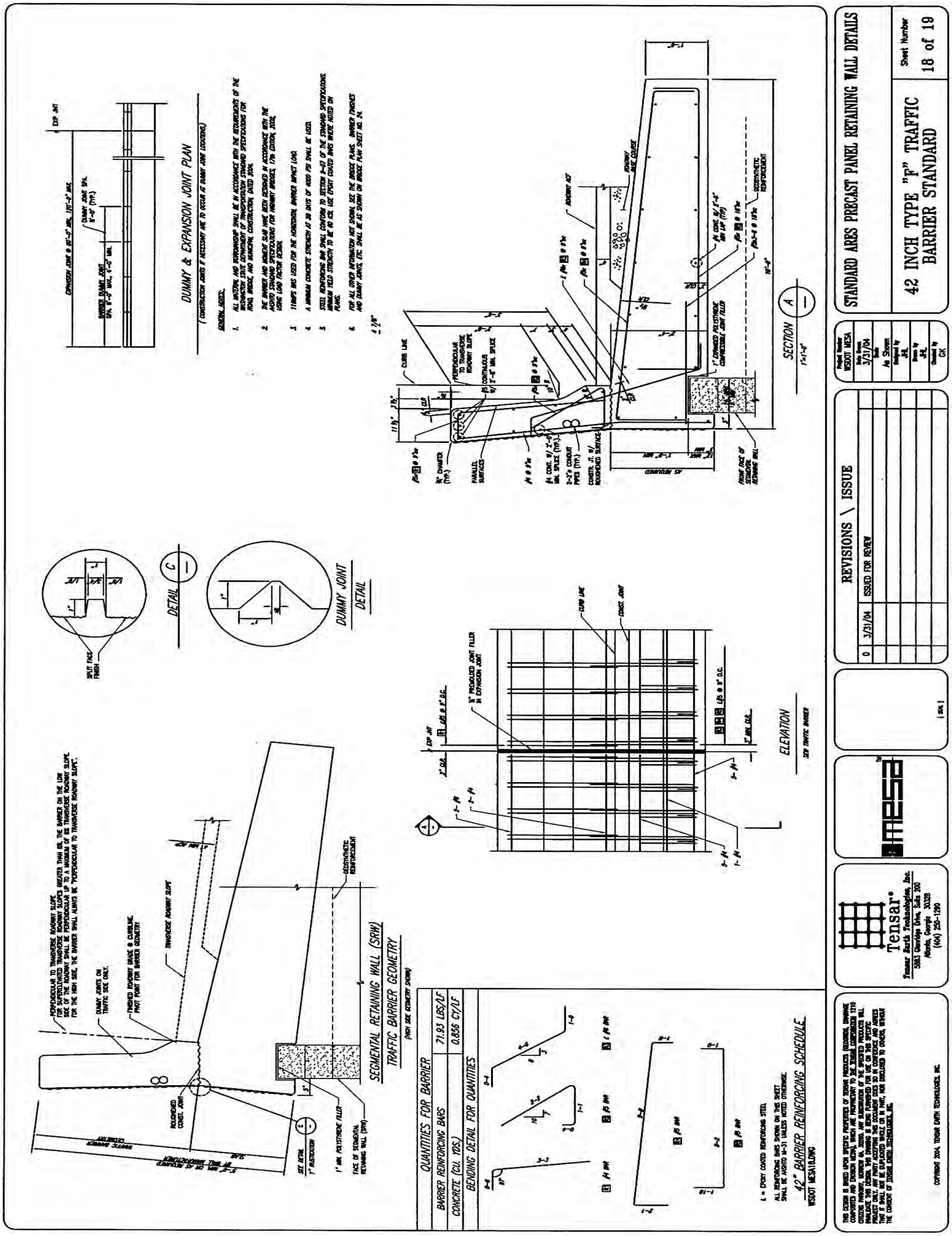
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42 INCH TYPE "F" TRAFFIC BARRIER STANDARD
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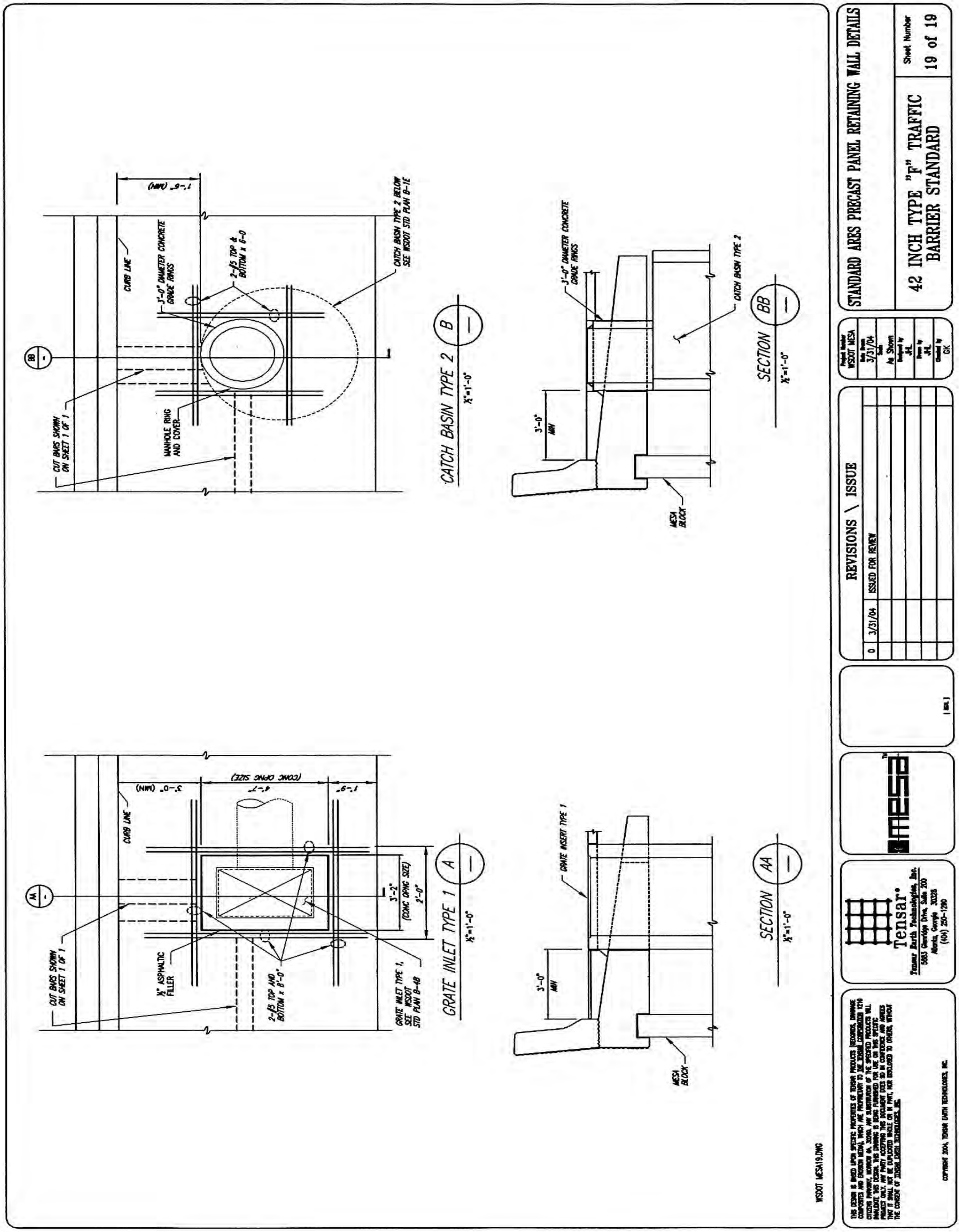
Tensar
SECTION A
1"=1'-0"
DUMMY & EXPANSION JOINT PLAN
ELEVATION
DETAIL C
DUMMY JOINT DETAIL
SEGMENTAL RETAINING WALL (SRW) TRAFFIC BARRIER GEOMETRY
QUANTITIES FOR BARRIER
BARRIER REINFORCING BARS 71.93 LBS/LF
CONCRETE (CU. YDS.) 0.856 CY/LF
BENDING DETAIL FOR QUANTITIES
42" BARRIER REINFORCING SCHEDULE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WSDOT MESA
3/31/04
As Shown
JHL
JHL
GK
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CATCH BASIN TYPE 2
SECTION BB
GRATE INLET TYPE 1
SECTION AA
CURB LINE
CUT BARS SHOWN ON SHEET 1 OF 1
MANHOLE RING AND COVER
3'-0" DIAMETER CONCRETE GRADE RINGS
2-#5 TOP & BOTTOM x 6-0
CATCH BASIN TYPE 2 BELOW SEE WSDOT STD PLAN B-1E
½" ASPHALTIC FILLER
2-#5 TOP AND BOTTOM x 8'-0"
GRATE INLET TYPE 1, SEE WSDOT STD PLAN B-4B
GRATE INSERT TYPE 1
MESA BLOCK
3'-0" MIN
WSDOT MESA19.DWG
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Appendix 15-L Preapproved Wall Appendix: Specific Requirements and Details for Tensar MESA Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply:

The detailed design methodology, design properties, and assumptions used by Tensar Earth Technologies for the MESA wall are summarized in the HITEC evaluation report for this wall system (HITEC, 2000, *Evaluation of the Tensar MESA Wall System*, ASCE, CERF Report No. 40358). The design methodology, which is based on the Standard Specifications for Highway Bridges (2002) is consistent with the general design requirements in Appendix 15-A, except as noted below. Interim approval is given for the continued use of the AASHTO Standard Specifications as the basis for design.

Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2 feet. Regarding horizontal spacing of reinforcement strips (i.e., rolls), reinforcement coverage ratios of greater than 0.7 are acceptable for this wall system. This is based on having a maximum of one facing block between reinforcement rolls, as allowed by the AASHTO Specifications.

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications. For LRFD based design, while it is recognized that product and soil type specific pullout interaction coefficients obtained in accordance with the AASHTO LRFD Specifications for the Tensar products used with this wall system are provided in the HITEC report for the MESA Wall system, pullout resistance design using these product and soil specific interaction coefficients has not been calibrated using the available product specific data statistics and reliability theory. Therefore, the specified resistance factors in the GDM and AASHTO LRFD Specifications should not be considered applicable to the product specific pullout interaction coefficients provided in the HITEC report.

The reinforcement long-term tensile strengths (T_{al}) provided in the *Qualified Products List* (QPL) for the Tensar Geogrid product series, which are based on the 2003 version of the product series, shall be used for wall design, until such time that they are updated, and the updated strengths approved for WSDOT use in accordance with WSDOT Standard Practice T 925. Until such time that the long-term reinforcement strengths are updated, it shall be verified that any material sent to the project site for this wall system is the 2003 version of the product. Furthermore, the short-term ultimate tensile strengths (ASTM D6637) listed in the QPL shall be used as the basis for quality assurance testing and acceptance of the product as shipped to the project site per the *Standard Specifications for Construction*.

The HITEC report provided connection data for the DOT^3 system and the HP System. Both systems provide partial connection coverage, with the DOT^3 system only providing 14 teeth per 21 openings, and the HP System providing 17 teeth per 21 openings. The DOT^3 system shall not be used.

The connection test results provided in the HITEC report for this wall system utilized an earlier version (i.e., before 2003) of the Tensar product series that had lower ultimate short-term geogrid tensile strengths than are currently approved in the QPL. Since connection test data have not been provided for the combination of the stronger Tensar geogrid product series (i.e., the 2003 series), the connection strengths in the HITEC report for the older product series shall be used, which is likely conservative. Based on the connection data provided in the HITEC report for this wall system, the short-term, ultimate connection strength reduction factor, CR_u, for the Tensar geogrid, MESA block combination using the HP Connector system is as provided in Table 15-(Tensar MESA)-1 for each product approved for use with the MESA system. Table 15-(Tensar MESA)-1 also provides the approved value of T_{ac}, as defined in the AASHTO LRFD Specifications, assuming a durability reduction factor of 1.1.

Table 15-L-1 Approved Connection Strength Design Values for Tensar MESA Walls

Tensar Geogrid Product	T_{ult} (MARV) for Geogrid per ASTM D6637 in HITEC Report (lbs/ft)	T_{ult} (MARV) for Geogrid per ASTM D6637 for 2003 Product (lbs/ft)	CR_u from HITEC Report	*CR_u if 2003 Tult (MARV) Values Used	RF_{CR}	CR_{cr} if 2003 T_{ult} (MARV) Values Used	T_{ac} (lbs/ft)
UMESA3	4400	4820	0.79	0.72	2.6	0.28	1200
UMESA4	6850	7880	0.73	0.63	2.6	0.24	1720
UMESA5	9030	9870	0.80	0.73	2.6	0.28	2510
UMESA6	10,700	12200	0.75	0.66	2.6	0.25	2770

*i.e., to get same Tultconn value as in HITEC report.

T_{ac}, the long-term connection strength, shall be calculated as follows:

$$T_{ac} = \frac{T_{MARV} \bullet CR_u}{RF_{CR} \bullet RF_D} \qquad (15\text{-L-1})$$

where,

T_{MARV} the minimum average roll value for the ultimate geosynthetic strength T_{ult},
CR_u = the ultimate connection strength $T_{ultconn}$ divided by the lot specific ultimate tensile strength, T_{lot} (i.e., the lot of material specific to the connection testing),
RF_{CR} = creep reduction factor for the geosynthetic, and
RF_D = the durability reduction factor for the geosynthetic.

Since the HITEC report was developed, Tensar Earth Technologies has developed a new connector that provides, for the most part, a full coverage connector, providing 19 teeth per 21 openings. Short-term connection tests on the strongest geogrid product in the series shows that connection strengths higher than those obtained with the HP System will be obtained with the new connector, which is called the DOT system (note that the 3 has been dropped – this is not the same as the DOT^3 system). This new DOT System may be used, provided that the values for T_{ac} shown in Table 15-(Tensar MESA)-1 are used for design, which should be conservative, until a more complete set of test results are available. Photographs illustrating the new DOT connector system are provided in Figures 15-(Tensar MESA)-1 through 15-(Tensar MESA)-3.

The longitudinal (i.e., in the direction of loading) and transverse (i.e., parallel to the wall or slope face) ribs that make up the geogrid shall be perpendicular to one another. The maximum deviation of the cross-rib from being perpendicular to the longitudinal rib (skew) shall be manufactured to be no more than 1 inch in 5 feet of geogrid width. The maximum deviation of the cross-rib at any point from a line perpendicular to the longitudinal ribs located at the cross-rib (bow) shall be 0.5 inches.

The gap between the connector tabs and the bearing surface of the geogrid reinforcement cross-rib shall not exceed 0.5 inches. A maximum of 10% of connector tabs may have a gap between 0.3 inches and 0.5 inches. Gaps in the remaining connector tabs shall not exceed 0.3 inches.

Concrete for dry cast concrete blocks used in the Tensar MESA wall system shall meet the following requirements:

1. Have a minimum 28 day compressive strength of 4,000 psi.
2. Conform to ASTM C1372.
3. The lot of blocks produced for use in this project shall conform to the following freeze-thaw test requirements when tested in accordance with ASTM C 1262:
   - Minimum acceptable performance shall be defined as weight loss at the conclusion of 150 freeze-thaw cycles not exceeding one percent of the block's initial weight for a minimum of four of the five block specimens tested.
4. The concrete blocks shall have a maximum water absorption of one percent above the water absorption content of the lot of blocks produced and successfully tested for the freeze-thaw test specified in the preceding paragraph.

It is noted in ASTM C1372 that a dimensional tolerance for the height of the block of ⅛ inch is allowed, but that Elias, et al. (2001), which is referenced in Chapter 15 and by the AASHTO Standard Specifications for Highway Bridges (2002) recommends a tighter dimensional tolerance of 1⁄16 inch. Based on WSDOT experience, for walls greater than 25 feet in height, some cracking of facing blocks due to differential vertical stresses tends to occur in the bottom portion of the wall. Therefore, blocks placed at depths below the wall top of 25 feet or more should be cast to a vertical dimensional tolerance of 1⁄16 inch to reduce the risk of significant cracking of facing blocks.

Figure 15-L-1 MESA DOT System Connector and Block

Figure 15-L-2 MESA DOT System Connector and Block as Assembled

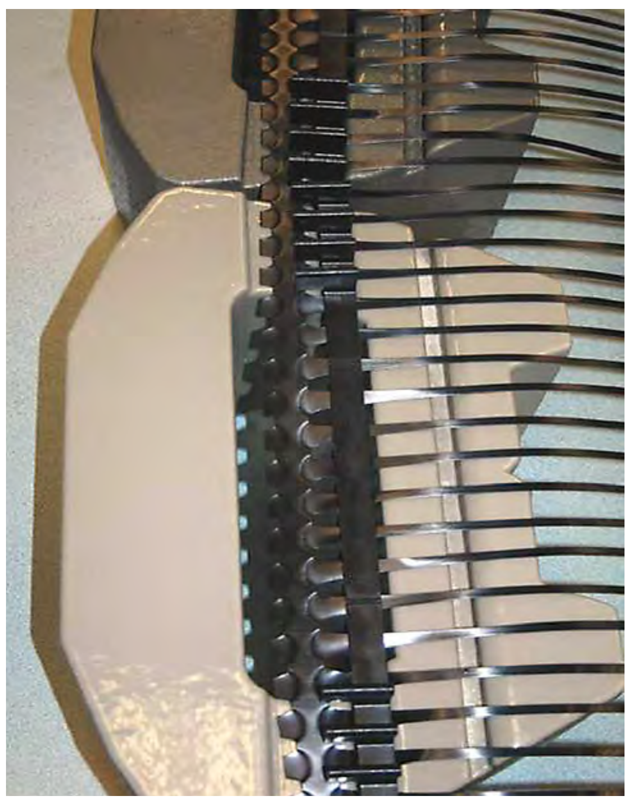
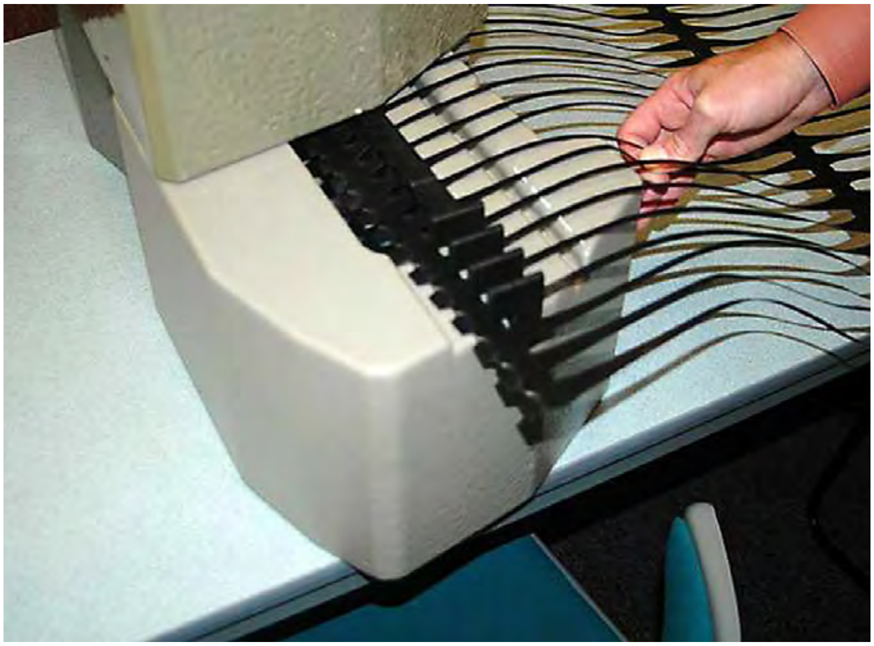


Figure 15-L-3 MESA DOT System Connector and Block as Assembled, With Block Placed on Top



Block connectors for block courses with geogrid reinforcement shall be glass fiber reinforced high-density polypropylene conforming to the following minimum material specifications:

Property	Specification	Value
Polypropylene	ASTM D 4101 Group 1 Class 1 Grade 2	73 ± 2 percent
Fiberglass Content	ASTM D 2584	25 ± 3 percent
Carbon Black	ASTM D 4218	2 percent minimum
Specific Gravity	ASTM D 792	1.08 ± 0.04
Tensile Strength at yield	ASTM D 638	8,700 ± 1,450 psi
Melt Flow Rate	ASTM D 1238	0.37 ± 0.16 ounces/10 min.

Block connectors for block courses without geogrid reinforcement shall be glass fiber reinforced high-density polyethylene (HDPE) conforming to the following minimum material specifications:

Property	Specification	Value
HDPE	ASTM D 1248 Group 3 Class 1 Grade 5	68 ± 3 percent
Fiberglass Content	ASTM D 2584	30 ± 3 percent
Carbon Black	ASTM D 4218	2 percent minimum
Specific Gravity	ASTM D 792	1.16 ± 0.06
Tensile Strength at yield	ASTM D 638	8,700 ± 725 psi
Melt Flow Rate	ASTM D 1238	0.11 ± 0.07 ounces/10 min.

Approved details for the Tensar MESA wall system with the DOT System connector are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- In plan sheet 5 of 13, the guard rail detail, the guard rail post shall either be installed through precut holes in the geogrid layers that must penetrated, or the geogrid layers shall be cut in a manner that prevents ripping or tearing of the geogrid.
- In plan sheets 4, 6, and 8 of 13, regarding the geotextiles and drainage composites shown, WSDOT reserves the right to require the use Standard Specifications materials as specified in Standard Specification Section 9-33 that are similar to those specified in this plan sheet.
- In plan sheet 7 of 13, regarding the geogrid at wall corner detail, cords in the wall facing alignment to form an angle point or a radius shall be no shorter than the width of the roll to insure good contact between the connectors and the geogrid cross-bar throughout the width of the geogrid. Alternatively, the geogrid roll could be cut longitudinally in half to allow a tighter radius, if necessary.
- In plan sheet 7 of 13, regarding the typical geogrid percent coverage, the maximum distance X between geogrid strips shall be one block width. Therefore, the minimum percent coverage shall be 73 percent.
- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 2 feet for culvert penetration through the face and up to 4 feet for obstruction avoidance. Larger diameter culverts or obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers.

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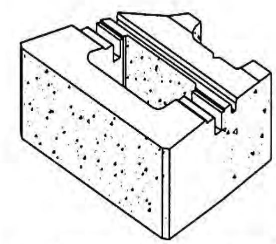
CONSTRUCTION DRAWINGS
Prepared For

STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION



MESA™

STANDARD MESA DETAILS & CONSTRUCTION NOTES

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2.	Construction Notes
3.	Typical Details
4.	Typical Details
5.	Typical Details
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7.	Typical Details
8.	Fabric Separator Detail
9.	Typical Details
10.	32 Inch Type "F" Barrier Standard
11.	32 Inch Type "F" Barrier Standard
12.	42 Inch Type "F" Barrier Standard
13.	42 Inch Type "F" Barrier Standard

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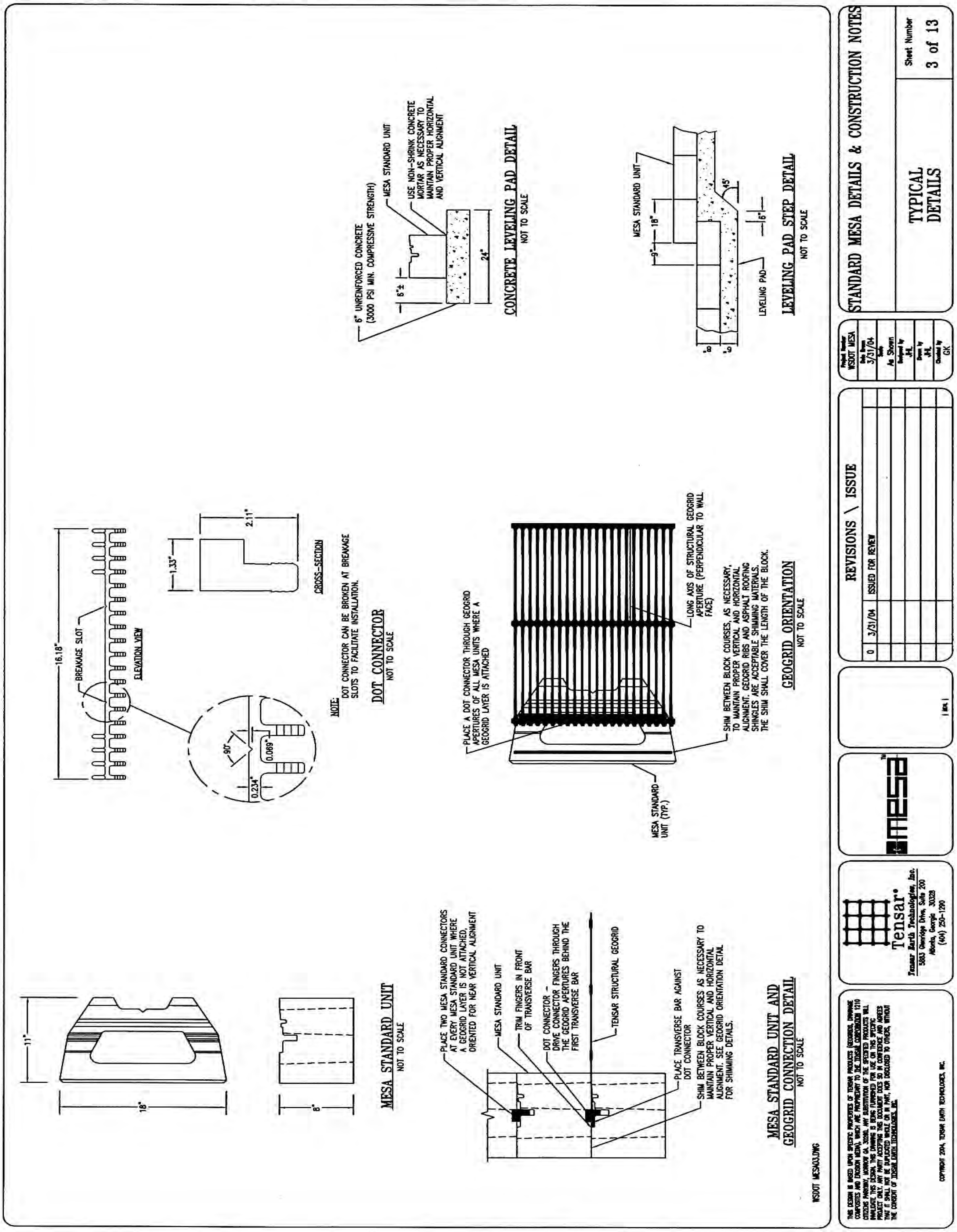
Drawn by: JHL
Checked by: GK

STANDARD MESA DETAILS & CONSTRUCTION NOTES

TITLE SHEET

Sheet Number: 1 of 13


MESA STANDARD UNIT
NOT TO SCALE
18"
11"
8"
MESA STANDARD UNIT AND GEOGRID CONNECTION DETAIL
NOT TO SCALE
PLACE TWO MESA STANDARD CONNECTORS AT EVERY MESA STANDARD UNIT WHERE A GEOGRID LAYER IS NOT ATTACHED. ORIENTED FOR NEAR VERTICAL ALIGNMENT
MESA STANDARD UNIT
TRIM FINGERS IN FRONT OF TRANSVERSE BAR
DOT CONNECTOR - DRIVE CONNECTOR FINGERS THROUGH THE GEOGRID APERTURES BEHIND THE FIRST TRANSVERSE BAR
TENSAR STRUCTURAL GEOGRID
PLACE TRANSVERSE BAR AGAINST DOT CONNECTOR
SHIM BETWEEN BLOCK COURSES AS NECESSARY TO MAINTAIN PROPER VERTICAL AND HORIZONTAL ALIGNMENT. SEE GEOGRID ORIENTATION DETAIL FOR SHIMMING DETAILS.
WSDOT MESA03.DWG
16.18"
BREAKAGE SLOT
ELEVATION VIEW
90°
0.234"
0.069"
1.33"
2.11"
CROSS-SECTION
NOTE: DOT CONNECTOR CAN BE BROKEN AT BREAKAGE SLOTS TO FACILITATE INSTALLATION.
DOT CONNECTOR
NOT TO SCALE
PLACE A DOT CONNECTOR THROUGH GEOGRID APERTURES OF ALL MESA UNITS WHERE A GEOGRID LAYER IS ATTACHED
MESA STANDARD UNIT (TYP.)
SHIM BETWEEN BLOCK COURSES, AS NECESSARY, TO MAINTAIN PROPER VERTICAL AND HORIZONTAL ALIGNMENT. GEOGRID RIBS AND ASPHALT ROOFING SHINGLES ARE ACCEPTABLE SHIMMING MATERIALS. THE SHIM SHALL COVER THE LENGTH OF THE BLOCK.
LONG AXIS OF STRUCTURAL GEOGRID APERTURE (PERPENDICULAR TO WALL FACE)
GEOGRID ORIENTATION
NOT TO SCALE
6" UNREINFORCED CONCRETE (3000 PSI MIN. COMPRESSIVE STRENGTH)
MESA STANDARD UNIT
USE NON-SHRINK CONCRETE MORTAR AS NECESSARY TO MAINTAIN PROPER HORIZONTAL AND VERTICAL ALIGNMENT
6"±
24"
CONCRETE LEVELING PAD DETAIL
NOT TO SCALE
MESA STANDARD UNIT
9"
18"
45°
6"
8"
LEVELING PAD
LEVELING PAD STEP DETAIL
NOT TO SCALE
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STANDARD MESA DETAILS & CONSTRUCTION NOTES
TYPICAL DETAILS
Sheet Number
3 of 13
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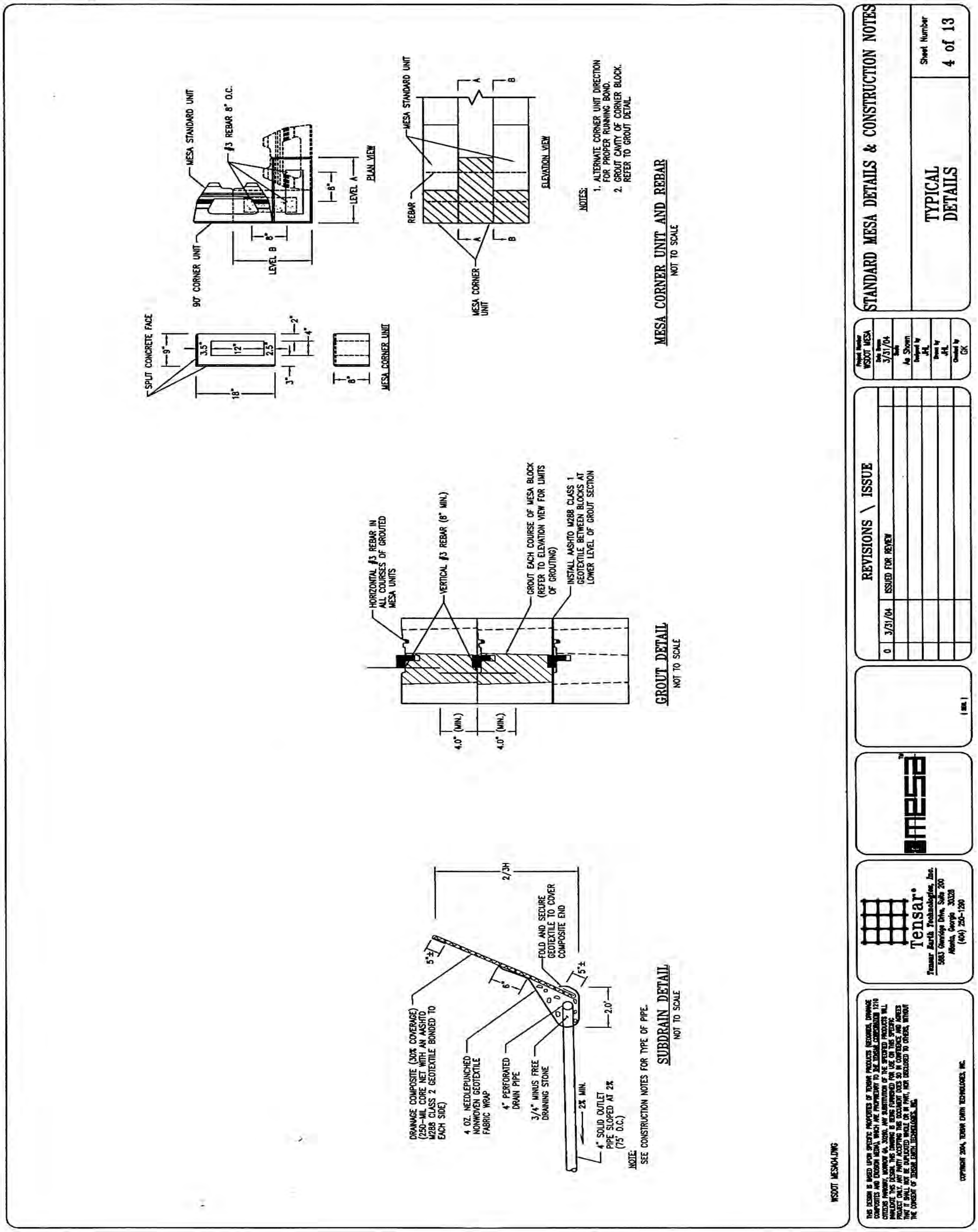
STANDARD MESA DETAILS & CONSTRUCTION NOTES
TYPICAL DETAILS
Sheet Number
4 of 13
MESA CORNER UNIT AND REBAR
NOT TO SCALE
SPLIT CONCRETE FACE
MESA CORNER UNIT
90° CORNER UNIT
MESA STANDARD UNIT
#3 REBAR 8" O.C.
LEVEL B
LEVEL A
PLAN VIEW
REBAR
MESA CORNER UNIT
ELEVATION VIEW
NOTES:
1. ALTERNATE CORNER UNIT DIRECTION FOR PROPER RUNNING BOND.
2. GROUT CAVITY OF CORNER BLOCK. REFER TO GROUT DETAIL.
GROUT DETAIL
NOT TO SCALE
HORIZONTAL #3 REBAR IN ALL COURSES OF GROUTED MESA UNITS
VERTICAL #3 REBAR (8" MIN.)
GROUT EACH COURSE OF MESA BLOCK (REFER TO ELEVATION VIEW FOR LIMITS OF GROUTING)
INSTALL AASHTO M288 CLASS 1 GEOTEXTILE BETWEEN BLOCKS AT LOWER LEVEL OF GROUT SECTION
4.0" (MIN.)
SUBDRAIN DETAIL
NOT TO SCALE
DRAINAGE COMPOSITE (30% COVERAGE) (250-MIL CORE NET WITH AN AASHTO M288 CLASS 2 GEOTEXTILE BONDED TO EACH SIDE)
4 OZ. NEEDLEPUNCHED NONWOVEN GEOTEXTILE FABRIC WRAP
4" PERFORATED DRAIN PIPE
3/4" MINUS FREE DRAINING STONE
4" SOLID OUTLET PIPE SLOPED AT 2% (75' O.C.)
FOLD AND SECURE GEOTEXTILE TO COVER COMPOSITE END
2/3H
2% MIN.
NOTE:
SEE CONSTRUCTION NOTES FOR TYPE OF PIPE.
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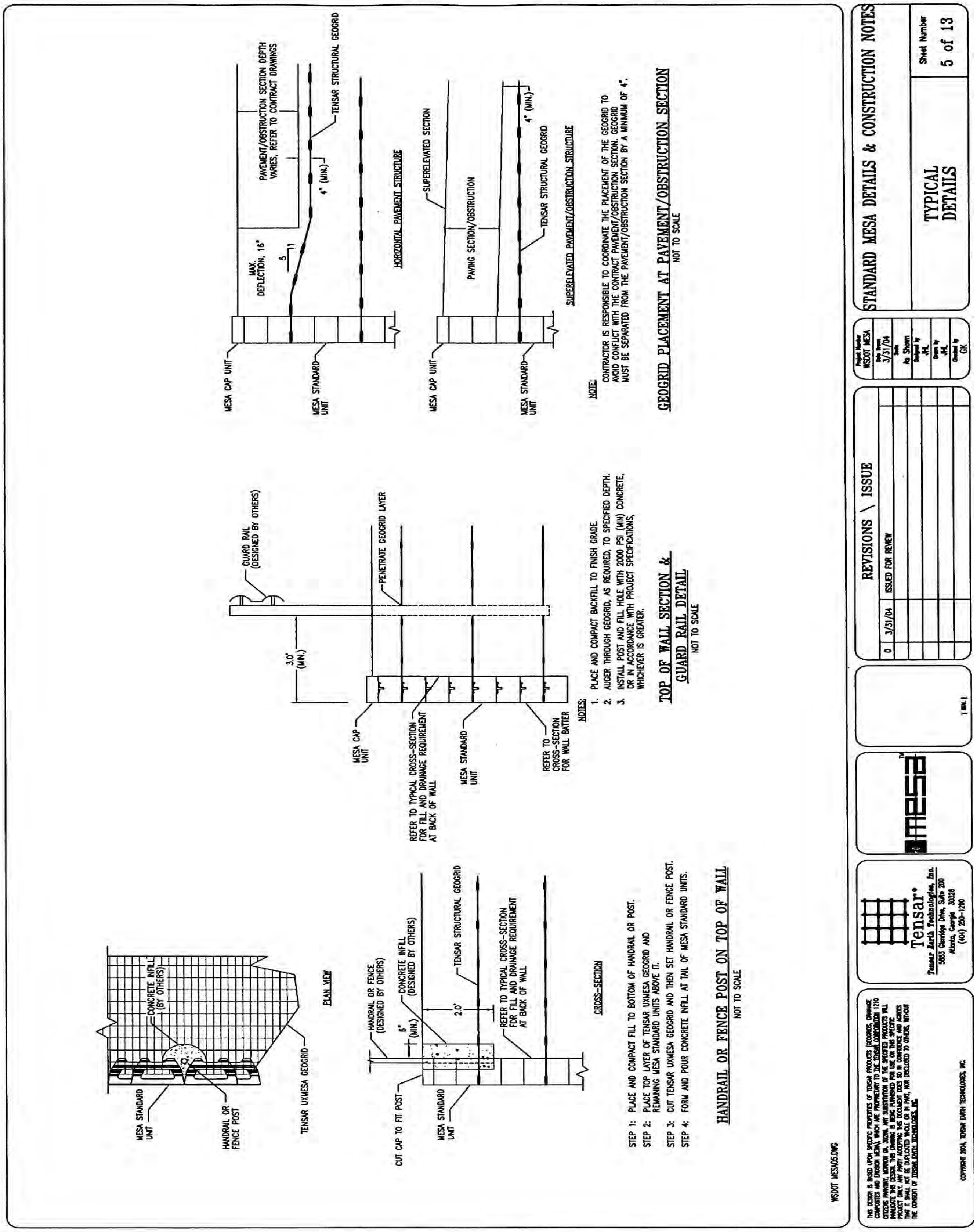
MESA STANDARD UNIT
HANDRAIL OR FENCE POST
TENSAR UXMESA GEOGRID
CONCRETE INFILL (BY OTHERS)
PLAN VIEW
CUT CAP TO FIT POST
MESA STANDARD UNIT
HANDRAIL OR FENCE (DESIGNED BY OTHERS)
6" (MIN.)
CONCRETE INFILL (DESIGNED BY OTHERS)
2.0'
TENSAR STRUCTURAL GEOGRID
REFER TO TYPICAL CROSS-SECTION FOR FILL AND DRAINAGE REQUIREMENT AT BACK OF WALL
CROSS-SECTION
STEP 1: PLACE AND COMPACT FILL TO BOTTOM OF HANDRAIL OR POST.
STEP 2: PLACE TOP LAYER OF TENSAR UXMESA GEOGRID AND REMAINING MESA STANDARD UNITS ABOVE IT.
STEP 3: CUT TENSAR UXMESA GEOGRID AND THEN SET HANDRAIL OR FENCE POST.
STEP 4: FORM AND POUR CONCRETE INFILL AT TAIL OF MESA STANDARD UNITS.
HANDRAIL OR FENCE POST ON TOP OF WALL
NOT TO SCALE
MESA CAP UNIT
REFER TO TYPICAL CROSS-SECTION FOR FILL AND DRAINAGE REQUIREMENT AT BACK OF WALL
MESA STANDARD UNIT
REFER TO CROSS-SECTION FOR WALL BATTER
3.0' (MIN.)
GUARD RAIL (DESIGNED BY OTHERS)
PENETRATE GEOGRID LAYER
NOTES:
1. PLACE AND COMPACT BACKFILL TO FINISH GRADE.
2. AUGER THROUGH GEOGRID, AS REQUIRED, TO SPECIFIED DEPTH.
3. INSTALL POST AND FILL HOLE WITH 2000 PSI (MIN) CONCRETE, OR IN ACCORDANCE WITH PROJECT SPECIFICATIONS, WHICHEVER IS GREATER.
TOP OF WALL SECTION & GUARD RAIL DETAIL
NOT TO SCALE
MESA CAP UNIT
MESA STANDARD UNIT
MAX. DEFLECTION, 16"
5
1
PAVEMENT/OBSTRUCTION SECTION DEPTH VARIES, REFER TO CONTRACT DRAWINGS
4" (MIN.)
TENSAR STRUCTURAL GEOGRID
HORIZONTAL PAVEMENT STRUCTURE
MESA CAP UNIT
MESA STANDARD UNIT
PAVING SECTION/OBSTRUCTION
SUPERELEVATED SECTION
TENSAR STRUCTURAL GEOGRID
4" (MIN.)
SUPERELEVATED PAVEMENT/OBSTRUCTION STRUCTURE
NOTE:
CONTRACTOR IS RESPONSIBLE TO COORDINATE THE PLACEMENT OF THE GEOGRID TO AVOID CONFLICT WITH THE CONTRACT PAVEMENT/OBSTRUCTION SECTION. GEOGRID MUST BE SEPARATED FROM THE PAVEMENT/OBSTRUCTION SECTION BY A MINIMUM OF 4".
GEOGRID PLACEMENT AT PAVEMENT/OBSTRUCTION SECTION
NOT TO SCALE
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Drawn by JHL
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STANDARD MESA DETAILS & CONSTRUCTION NOTES
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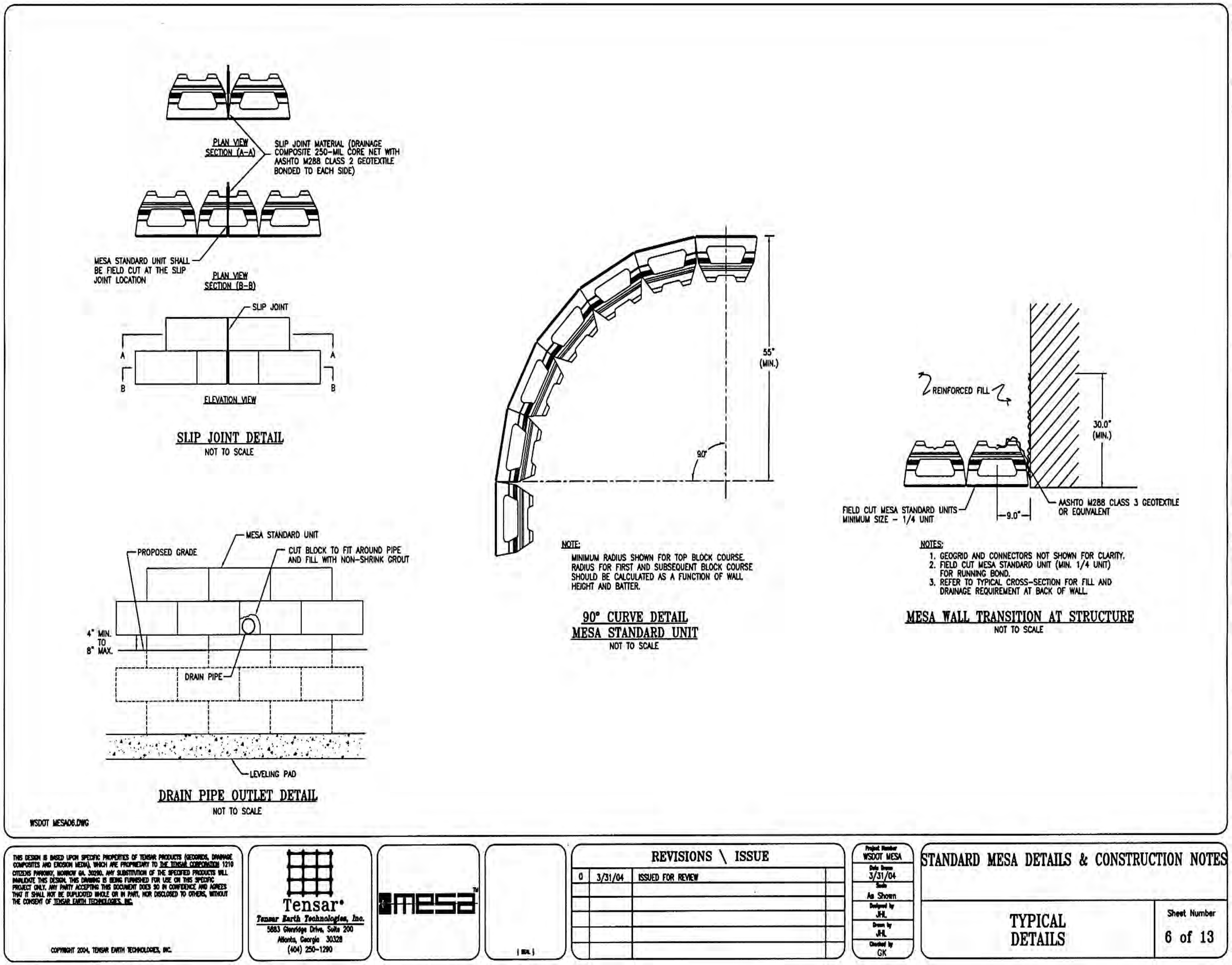
PLAN VIEW
SECTION (A-A)
SLIP JOINT MATERIAL (DRAINAGE COMPOSITE 250-MIL CORE NET WITH AASHTO M288 CLASS 2 GEOTEXTILE BONDED TO EACH SIDE)
MESA STANDARD UNIT SHALL BE FIELD CUT AT THE SLIP JOINT LOCATION
PLAN VIEW
SECTION (B-B)
SLIP JOINT
ELEVATION VIEW
SLIP JOINT DETAIL
NOT TO SCALE
55" (MIN.)
90°
NOTE:
MINIMUM RADIUS SHOWN FOR TOP BLOCK COURSE. RADIUS FOR FIRST AND SUBSEQUENT BLOCK COURSE SHOULD BE CALCULATED AS A FUNCTION OF WALL HEIGHT AND BATTER.
90° CURVE DETAIL
MESA STANDARD UNIT
NOT TO SCALE
REINFORCED FILL
30.0" (MIN.)
FIELD CUT MESA STANDARD UNITS MINIMUM SIZE - 1/4 UNIT
9.0"
AASHTO M288 CLASS 3 GEOTEXTILE OR EQUIVALENT
NOTES:
1. GEOGRID AND CONNECTORS NOT SHOWN FOR CLARITY.
2. FIELD CUT MESA STANDARD UNIT (MIN. 1/4 UNIT) FOR RUNNING BOND.
3. REFER TO TYPICAL CROSS-SECTION FOR FILL AND DRAINAGE REQUIREMENT AT BACK OF WALL.
MESA WALL TRANSITION AT STRUCTURE
NOT TO SCALE
MESA STANDARD UNIT
PROPOSED GRADE
CUT BLOCK TO FIT AROUND PIPE AND FILL WITH NON-SHRINK GROUT
4" MIN. TO 8" MAX.
DRAIN PIPE
LEVELING PAD
DRAIN PIPE OUTLET DETAIL
NOT TO SCALE
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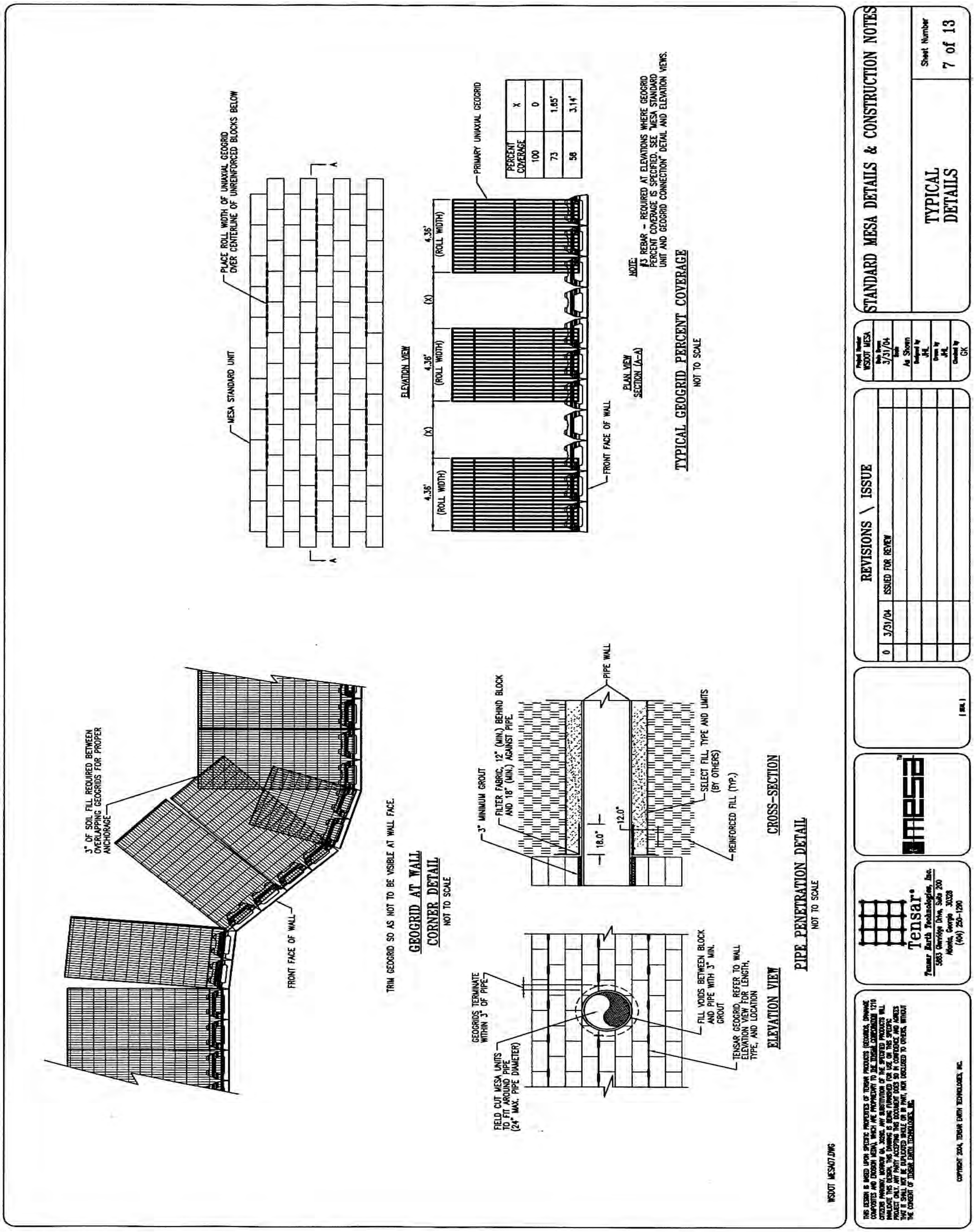

3" OF SOIL FILL REQUIRED BETWEEN OVERLAPPING GEOGRIDS FOR PROPER ANCHORAGE
FRONT FACE OF WALL
TRIM GEOGRID SO AS NOT TO BE VISIBLE AT WALL FACE.
GEOGRID AT WALL CORNER DETAIL
NOT TO SCALE
PLACE ROLL WIDTH OF UNIAXIAL GEOGRID OVER CENTERLINE OF UNREINFORCED BLOCKS BELOW
MESA STANDARD UNIT
ELEVATION VIEW
4.36' (ROLL WIDTH)
(X)
PRIMARY UNIAXIAL GEOGRID
FRONT FACE OF WALL
PLAN VIEW SECTION (A-A)
PERCENT COVERAGE
X
100
0
73
1.85'
58
3.14'
NOTE:
#3 REBAR - REQUIRED AT ELEVATIONS WHERE GEOGRID PERCENT COVERAGE IS SPECIFIED. SEE "MESA STANDARD UNIT AND GEOGRID CONNECTION" DETAIL AND ELEVATION VIEWS.
TYPICAL GEOGRID PERCENT COVERAGE
NOT TO SCALE
FIELD CUT MESA UNITS TO FIT AROUND PIPE (24" MAX. PIPE DIAMETER)
GEOGRIDS TERMINATE WITHIN 3" OF PIPE
FILL VOIDS BETWEEN BLOCK AND PIPE WITH 3" MIN. GROUT
TENSAR GEOGRID, REFER TO WALL ELEVATION VIEW FOR LENGTH, TYPE, AND LOCATION
ELEVATION VIEW
3" MINIMUM GROUT
FILTER FABRIC, 12" (MIN.) BEHIND BLOCK AND 18" (MIN.) AGAINST PIPE.
PIPE WALL
18.0"
12.0"
SELECT FILL, TYPE AND LIMITS (BY OTHERS)
REINFORCED FILL (TYP.)
CROSS-SECTION
PIPE PENETRATION DETAIL
NOT TO SCALE
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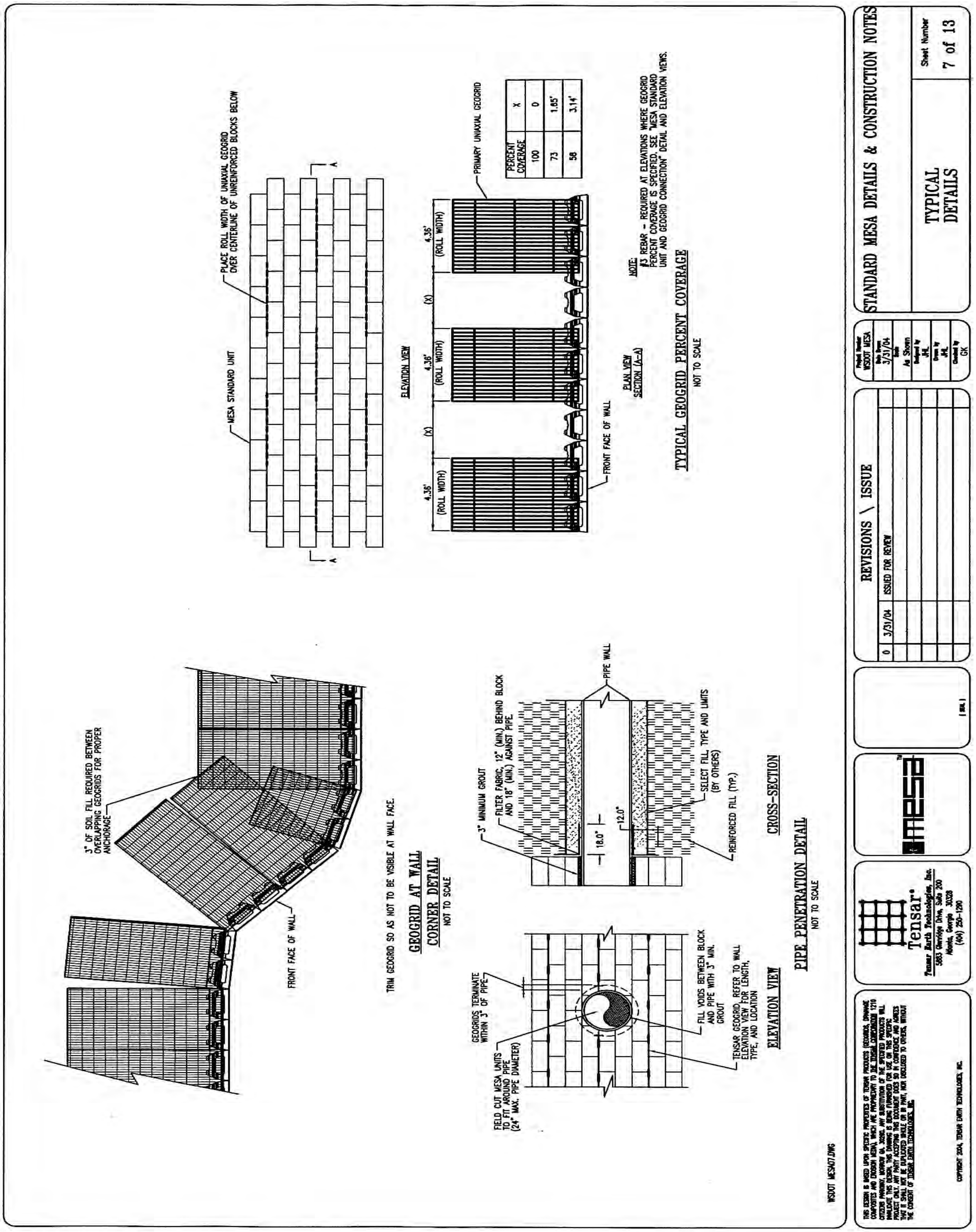

STANDARD MESA DETAILS & CONSTRUCTION NOTES
TYPICAL DETAILS
Sheet Number 7 of 13
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TYPICAL GEOGRID PERCENT COVERAGE
NOT TO SCALE
ELEVATION VIEW
PLAN VIEW SECTION (A-A)
MESA STANDARD UNIT
PLACE ROLL WIDTH OF UNIAXIAL GEOGRID OVER CENTERLINE OF UNREINFORCED BLOCKS BELOW
PRIMARY UNIAXIAL GEOGRID
FRONT FACE OF WALL
4.36' (ROLL WIDTH)
(X)
PERCENT COVERAGE
X
100
0
73
1.85'
58
3.14'
NOTE:
#3 REBAR – REQUIRED AT ELEVATIONS WHERE GEOGRID PERCENT COVERAGE IS SPECIFIED. SEE "MESA STANDARD UNIT AND GEOGRID CONNECTION" DETAIL AND ELEVATION VIEWS.
GEOGRID AT WALL CORNER DETAIL
NOT TO SCALE
3" OF SOIL FILL REQUIRED BETWEEN OVERLAPPING GEOGRIDS FOR PROPER ANCHORAGE
FRONT FACE OF WALL
TRIM GEOGRID SO AS NOT TO BE VISIBLE AT WALL FACE.
PIPE PENETRATION DETAIL
NOT TO SCALE
ELEVATION VIEW
CROSS-SECTION
FIELD CUT MESA UNITS TO FIT AROUND PIPE (24" MAX. PIPE DIAMETER)
GEOGRIDS TERMINATE WITHIN 3" OF PIPE
FILL VOIDS BETWEEN BLOCK AND PIPE WITH 3" MIN. GROUT
TENSAR GEOGRID. REFER TO WALL ELEVATION VIEW FOR LENGTH, TYPE, AND LOCATION
3" MINIMUM GROUT
FILTER FABRIC, 12" (MIN.) BEHIND BLOCK AND 18" (MIN.) AGAINST PIPE.
PIPE WALL
18.0"
12.0"
SELECT FILL. TYPE AND LIMITS (BY OTHERS)
REINFORCED FILL (TYP.)
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FABRIC DRAINAGE SPECIFICATION

Geotextile may be used in lieu of aggregate for the drainage medium behind the Mesa facing units if the select fill used in the reinforced zone, as a minimum, complies with the AASHTO Guidelines for Mechanically Stabilized Earth Retaining Walls. Section 5.8, Mechanically Stabilized Earth (MSE) Wall Design of the 2002 AASHTO guidelines defines the soil's physical properties.

An eight (8) oz per square yard nonwoven needle punched geotextile, AASHTO M288 CLASS 1, shall be used as the filter medium.

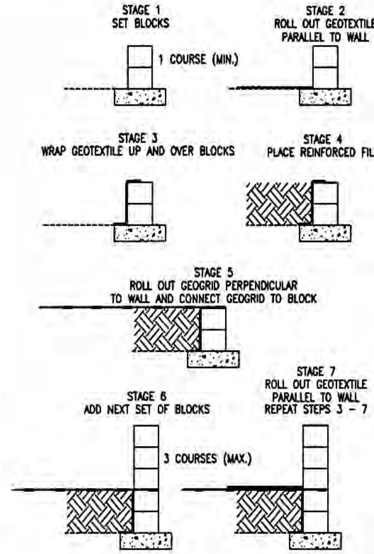
Unless otherwise approved by Tensar, the geotextile shall be delivered to the project site in rolls that have been factory cut to the specified widths for the installation.

INSTALLATION PROCEDURE

1. Install each course of the Mesa facing units between the top of the last geogrid placed and the bottom of the next layer of geogrid above as shown on the approved construction drawings. The facing units shall be aligned and leveled in accordance with the wall installation guide.
2. Prior to placing the select backfill, the geotextile shall be placed behind the units such that a minimum of six (6) inches of material is turned into the fill at the top and the bottom. The geotextile shall then be adjusted to present a relatively smooth surface.
3. The select backfill shall then be placed and compacted in accordance with the approved construction drawings and project specifications.
4. After the select backfill has been compacted and properly graded for the installation of the next layer of geogrid reinforcement, the geotextile on the top units can remain in position on the facing unit (see Stage 4 detail) or be pulled back onto the backfill (contractor's option).
5. Install the geogrid reinforcement and repeat the process commencing with Item 1.
6. After the last level of primary geogrid reinforcement has been placed, install the remaining courses of Mesa facing units except for the last standard course and the cap units, in accordance with the details on the approved construction drawings.
7. Place a line of an approved construction adhesive along the top of the Mesa Standard units approximately one (1) inch behind the face as shown in Detail S1.
8. Place the eight (8) oz. geotextile such that the leading edge of the material is approximately 1/2 inch behind the face and press into the adhesive. The bottom of the geotextile shall extend a minimum of six (6) inches into the select fill. Allow adhesive to obtain an initial set for approximately 20 minutes (Detail S1).
9. Install the Mesa connectors in the slots as shown in the wall installation guide. Connectors will push the fabric into the slot as shown in the Typical Section, this sheet.
10. Install the last course of Mesa Standard units and align and level as required in the installation guide.
11. Place a line of adhesive in the depressed area between the connector slot and the face of the unit per Detail S2.
12. Install the eight (8) oz. geotextile such that the leading edge of the material just contacts the step at the edge of the depressed area as shown in the Typical Section, this sheet. The bottom of the geotextile shall extend a minimum of six (6) inches beyond the geotextile in the Mesa Standard unit.
13. Place a line of adhesive along the top of the Standard units just behind the face per Detail S3.
14. Install the cap units as shown in Detail S4.

Geotextile widths required for the detail:

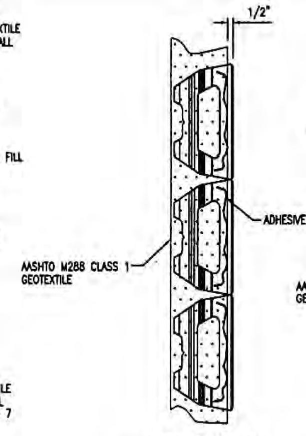
AASHTO M288 Class 1: 36 inch



STAGES OF GEOTEXTILE LAYOUT

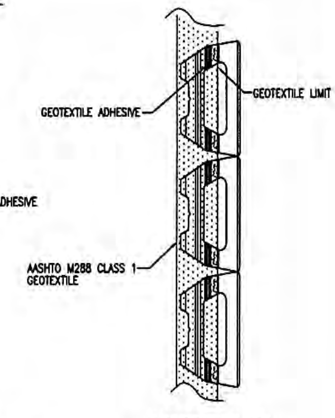
MESA WALL GEOTEXTILE SEPARATOR

NOT TO SCALE



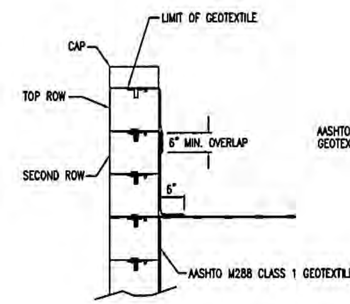
GEOTEXTILE ON SECOND ROW MESA UNIT

DETAIL S1



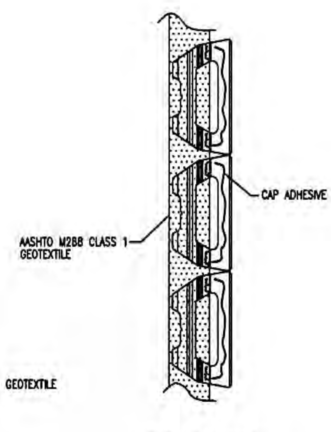
GEOTEXTILE ON TOP MESA UNIT

DETAIL S2



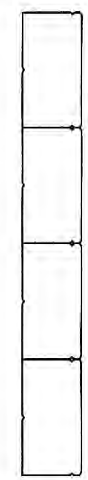
TYPICAL SECTION

NOT TO SCALE



CAP ADHESIVE

DETAIL S3



MESA CAP

DETAIL S4

WSDOT MESA08.DWG

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Drawn by: JHL
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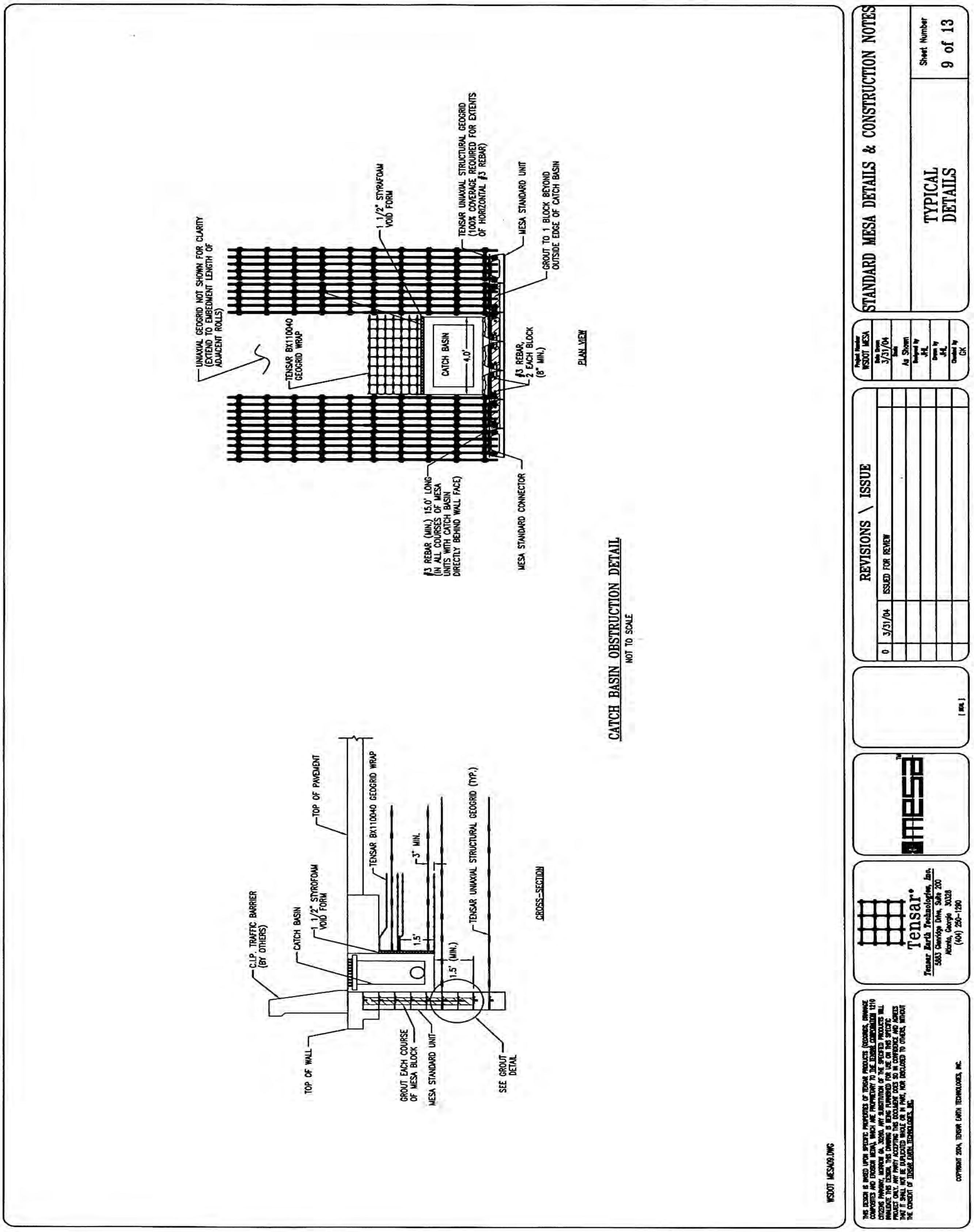
STANDARD MESA DETAILS & CONSTRUCTION NOTES

FABRIC SEPARATOR DETAIL

Sheet Number: 8 of 13


UNIAXIAL GEOGRID NOT SHOWN FOR CLARITY (EXTEND TO EMBEDMENT LENGTH OF ADJACENT ROLLS)
TENSAR BX110040 GEOGRID WRAP
1 1/2" STYRAFOAM VOID FORM
TENSAR UNIAXIAL STRUCTURAL GEOGRID (100% COVERAGE REQUIRED FOR EXTENTS OF HORIZONTAL #3 REBAR)
MESA STANDARD UNIT
GROUT TO 1 BLOCK BEYOND OUTSIDE EDGE OF CATCH BASIN
CATCH BASIN
4.0'
#3 REBAR, 2 EACH BLOCK (8" MIN.)
PLAN VIEW
#3 REBAR (MIN.) 15.0' LONG (IN ALL COURSES OF MESA UNITS WITH CATCH BASIN DIRECTLY BEHIND WALL FACE)
MESA STANDARD CONNECTOR
CATCH BASIN OBSTRUCTION DETAIL
NOT TO SCALE
C.I.P. TRAFFIC BARRIER (BY OTHERS)
CATCH BASIN
1 1/2" STYROFOAM VOID FORM
TOP OF PAVEMENT
TENSAR BX110040 GEOGRID WRAP
3" MIN.
1.5'
1.5' (MIN.)
TENSAR UNIAXIAL STRUCTURAL GEOGRID (TYP.)
TOP OF WALL
GROUT EACH COURSE OF MESA BLOCK
MESA STANDARD UNIT
SEE GROUT DETAIL
CROSS-SECTION
WSDOT MESA09.DWG
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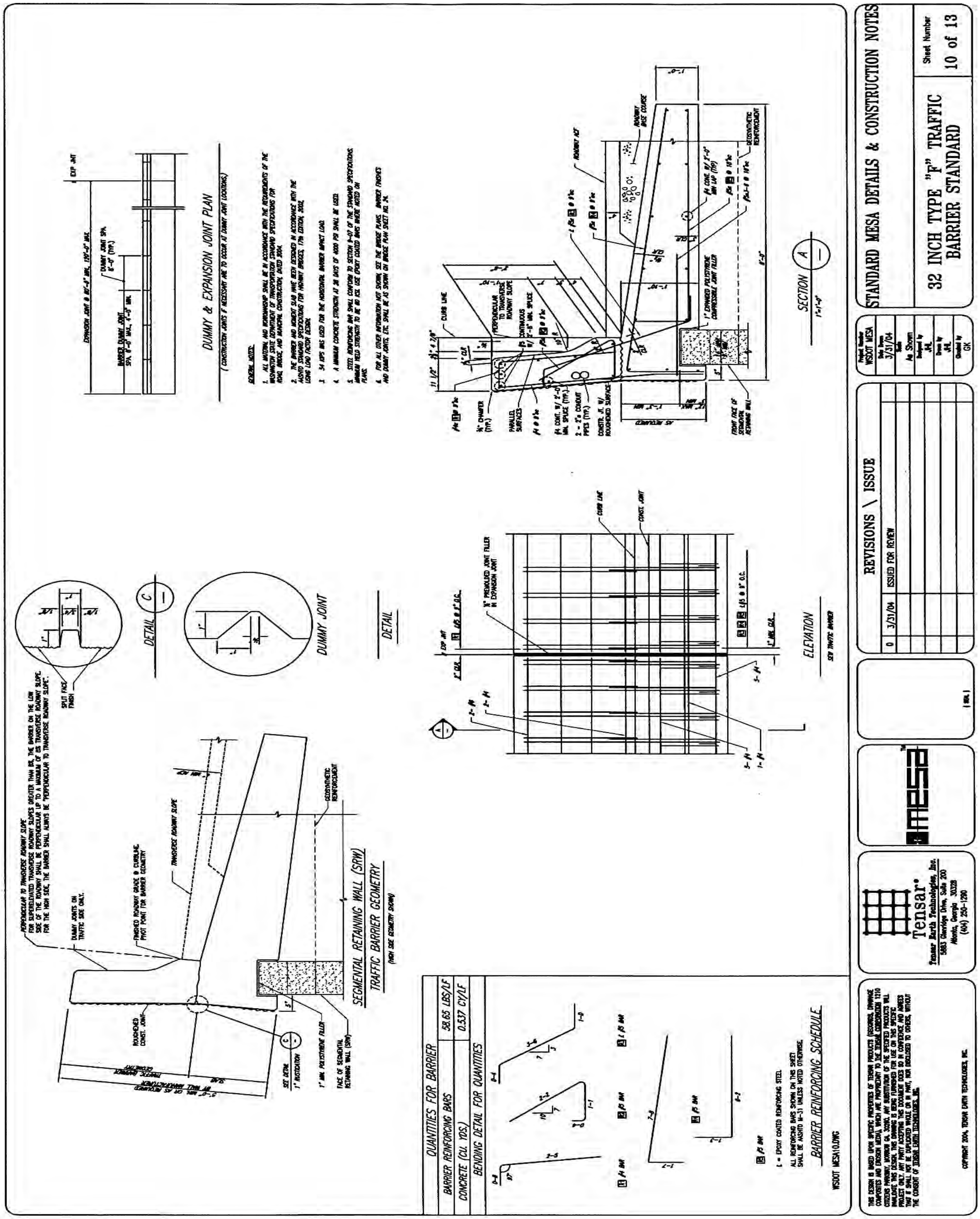
STANDARD MESA DETAILS & CONSTRUCTION NOTES
32 INCH TYPE "F" TRAFFIC BARRIER STANDARD
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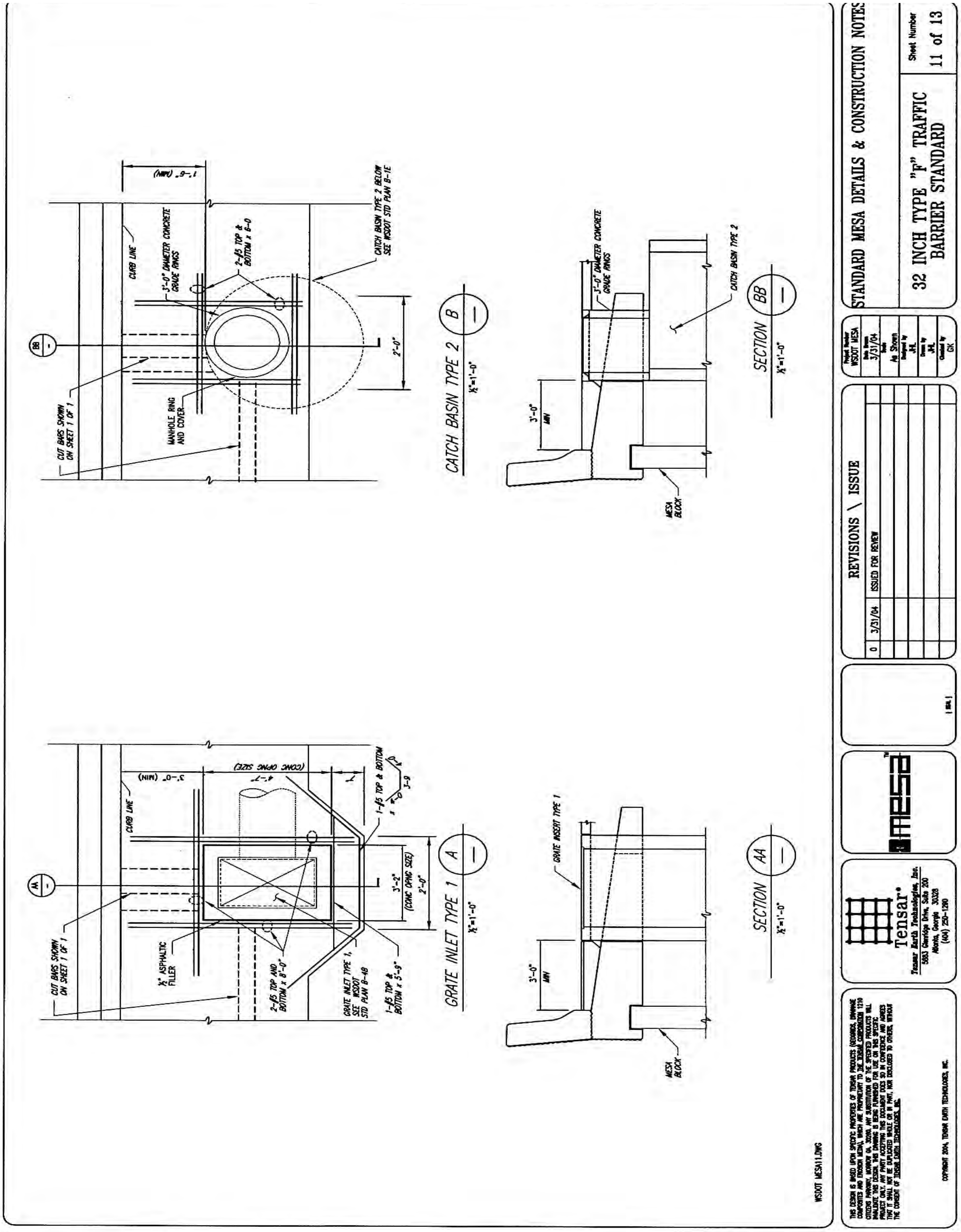
DUMMY & EXPANSION JOINT PLAN
SECTION A
ELEVATION
DETAIL C
DUMMY JOINT
DETAIL
SEGMENTAL RETAINING WALL (SRW)
TRAFFIC BARRIER GEOMETRY
QUANTITIES FOR BARRIER
BARRIER REINFORCING BARS 58.65 LBS/LF
CONCRETE (CU. YDS.) 0.537 CY/LF
BENDING DETAIL FOR QUANTITIES
BARRIER REINFORCING SCHEDULE
Tensar


GRATE INLET TYPE 1
SECTION AA
CATCH BASIN TYPE 2
SECTION BB
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STANDARD MESA DETAILS & CONSTRUCTION NOTES
32 INCH TYPE "F" TRAFFIC BARRIER STANDARD
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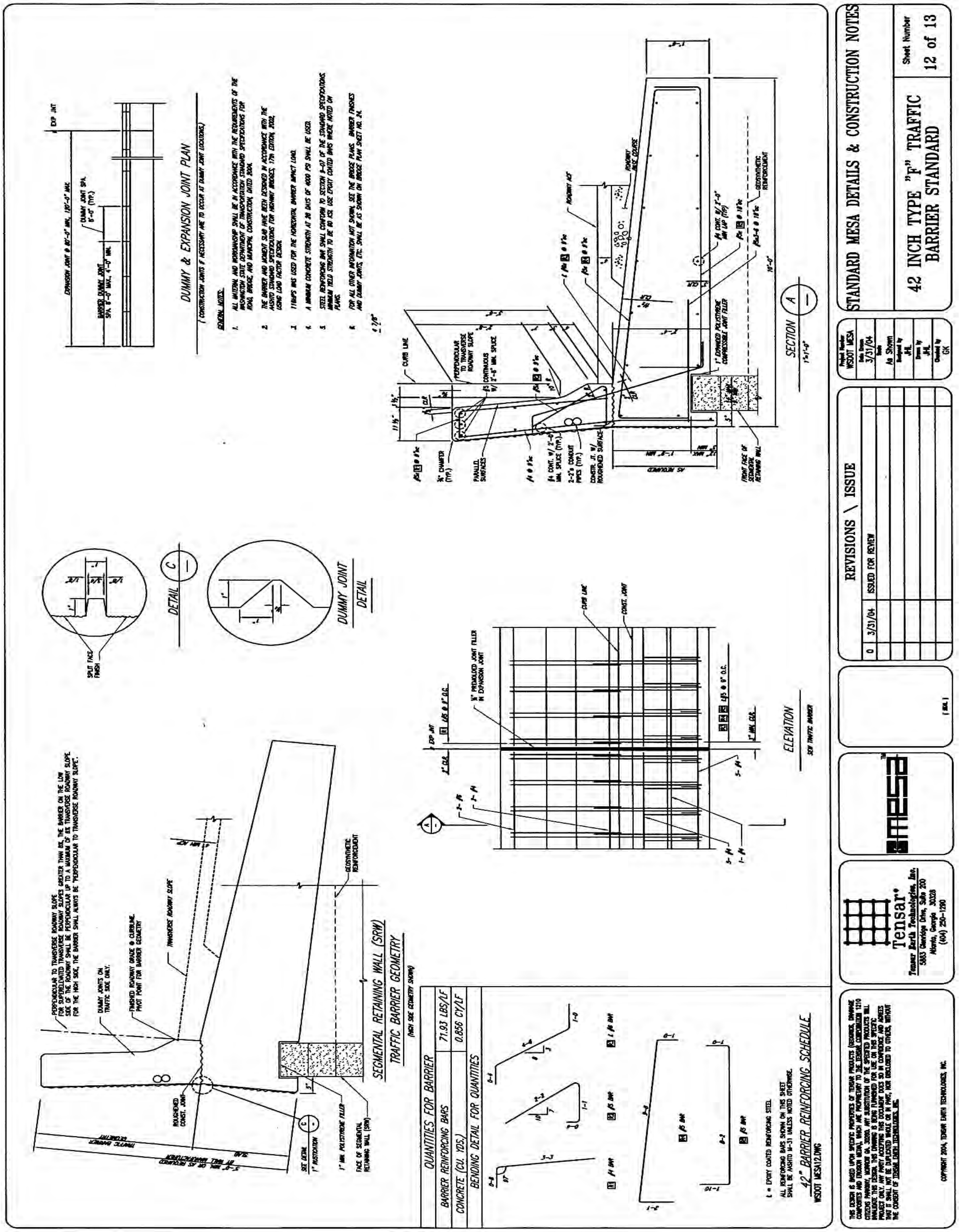
STANDARD MESA DETAILS & CONSTRUCTION NOTES
42 INCH TYPE "F" TRAFFIC BARRIER STANDARD
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DUMMY & EXPANSION JOINT PLAN
SECTION A
ELEVATION
DETAIL C
DUMMY JOINT DETAIL
SEGMENTAL RETAINING WALL (SRW) TRAFFIC BARRIER GEOMETRY
QUANTITIES FOR BARRIER
BARRIER REINFORCING BARS 71.93 LBS/LF
CONCRETE (CU. YDS.) 0.856 CY/LF
BENDING DETAIL FOR QUANTITIES
42" BARRIER REINFORCING SCHEDULE
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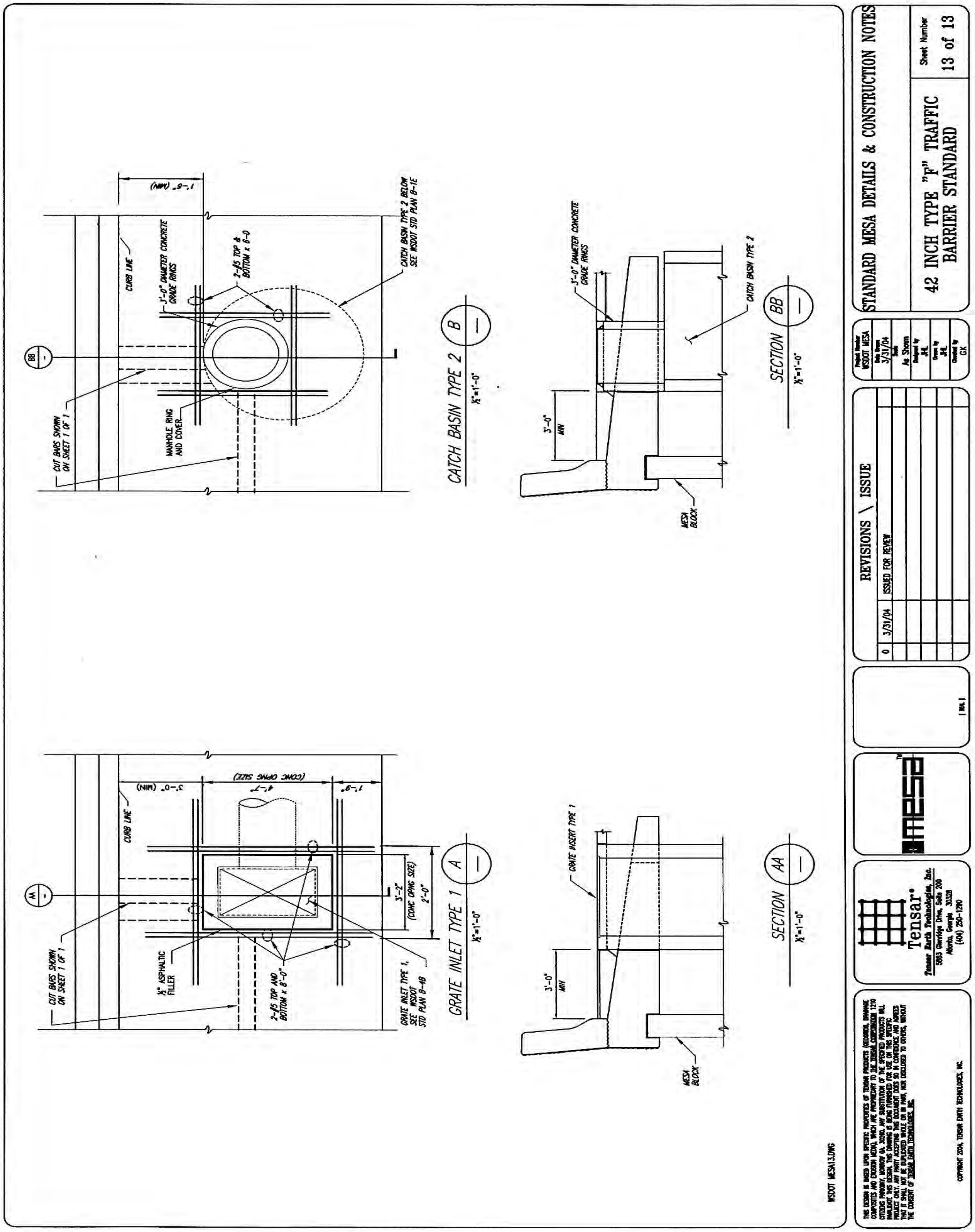
STANDARD MESA DETAILS & CONSTRUCTION NOTES
42 INCH TYPE "F" TRAFFIC BARRIER STANDARD
Sheet Number
13 of 13
REVISIONS \ ISSUE
0 3/31/04 ISSUED FOR REVIEW
Project Number WSDOT MESA
Date Drawn 3/31/04
Scale As Shown
Designed by JHL
Drawn by JHL
Checked by GK
Tensar
Tensar Earth Technologies, Inc.
5883 Glenridge Drive, Suite 200
Atlanta, Georgia 30328
(404) 250-1290
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WSDOT MESA13.DWG
GRATE INLET TYPE 1
A
½"=1'-0"
CATCH BASIN TYPE 2
B
½"=1'-0"
SECTION
AA
½"=1'-0"
SECTION
BB
½"=1'-0"
CUT BARS SHOWN ON SHEET 1 OF 1
CURB LINE
½" ASPHALTIC FILLER
2-#5 TOP AND BOTTOM x 8'-0"
GRATE INLET TYPE 1, SEE WSDOT STD PLAN B-4B
3'-2" (CONC OPNG SIZE)
2'-0"
4'-7" (CONC OPNG SIZE)
1'-9"
3'-0" (MIN)
GRATE INSERT TYPE 1
3'-0" MIN
MESA BLOCK
MANHOLE RING AND COVER
3'-0" DIAMETER CONCRETE GRADE RINGS
2-#5 TOP & BOTTOM x 6-0
CATCH BASIN TYPE 2 BELOW SEE WSDOT STD PLAN B-1E
1'-6" (MIN)
CATCH BASIN TYPE 2

Appendix 15-M Preapproved Wall Appendix: Specific Requirements and Details for Tensar Welded Wire Form Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific design requirements shall be met:

No HITEC evaluation report is currently available for this wall system. Design procedures for specific elements of the wall system have been provided to WSDOT in a submittal dated May 20, 2005, and final Wall Details submitted May 26, 2005. The design procedures used by Tensar Earth Technologies (TET) are in full conformance with the AASHTO LRFD Bridge Design Specifications (2004).

This wall system consists of Tensar geogrid reinforcement that is connected to a welded wire facing panel. Regarding the welded wire facing panel, the minimum wire size acceptable for permanent walls is W4.5, and the welded wire shall be galvanized in accordance with the AASHTO LRFD specifications. The actual wire size submitted is W4.0. The exception regarding the wire size is allowed. Due to the smaller wire size, there is some risk that the welded wire form will not provide the full 75 year life required for the wall. Therefore, to insure internal stability of the wall, the geogrid reinforcement shall be wrapped fully behind the face to add the redundancy needed to insure the wall face system is stable for the required design life. The galvanization requirement for the welded wire form still applies, however, as failure of the welded wire form at some point during the wall design life could allow some local sagging of the wall face to occur. The minimum clear opening dimension of the facing panel, or backing mat if present, shall not exceed the minimum particle size of the wall facing backfill. The maximum particle size for the wall facing backfill shall be 4 inches. The maximum vertical spacing of soil reinforcement shall be 18 inches for vertical and battered wall facings.

The geogrid tensile strengths used for design for this wall system shall be aslisted in the WSDOT Qualified Products List (QPL).

The Bodkin connection shown in the typical cross-section (page 15-(Tensar WW)-1) may be used subject to the following conditions:

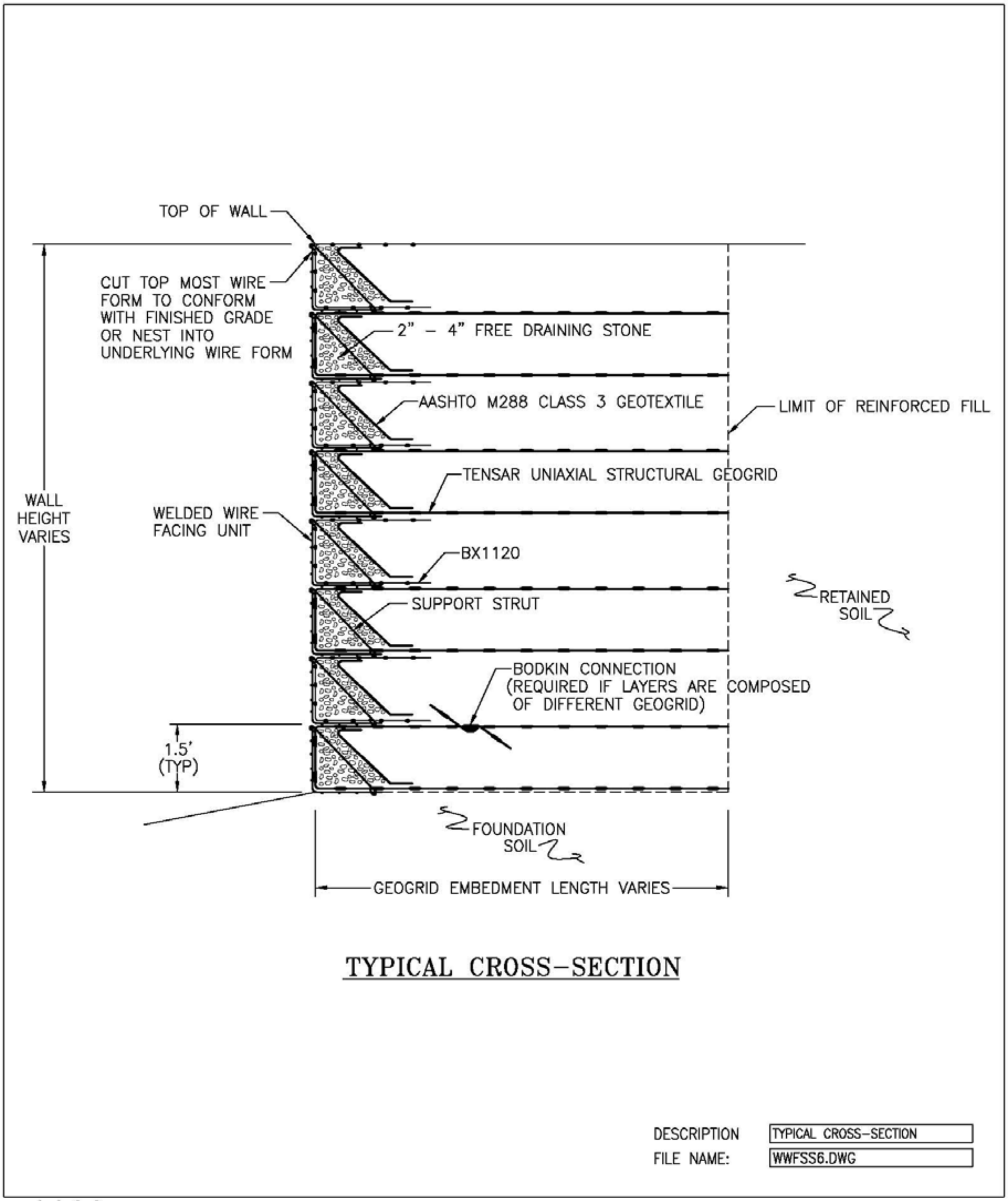
- No more than one Bodkin connection may be used within any given layer, andon no more than 50% of the layers in a given section ofwall.
- If the Bodkin connection is located outside of the active zone for the wall as defined in the AASHTO LRFD Bridge Design Specifications plus 3 feet and is located at least 4 feet from the face, no reduction in design tensile strength due to the presence of the Bodkin connection is required.
- If the Bodkin connection is located closer to the wall face than as described immediately above, the design tensile strength of the reinforcement shall be reduced to account for the Bodkin connection. Table 15-(Tensar WW)-1 provides a summary of the reduction factors to be applied to account for the presence of the Bodkin connection.

Table 15-M-1 Approved Bodkin Connection Strength Reduction Factors for Tensar Welded Wire Form Walls

Tensar Primary SoilReinforcement Geogrid Product	Tensar Product to Which Soil Reinforcement is Connected	Connection Strength Reduction Factor, CR_u
UMESA3/UX1400HS	UMESA6/UX1700HS	1.0
UMESA4/UX1500HS	UMESA6/UX1700HS	1.0
UMESA5/UX1600HS	UMESA6/UX1700HS	1.0
UMESA6/UX1700HS	UMESA6/UX1700HS	0.91
UMESA3/UX1400HS	UMESA3/UX1400HS	0.85
UMESA4/UX1500HS	UMESA4/UX1500HS	0.79
UMESA5/UX1600HS	UMESA5/UX1600HS	0.87
UMESA6/UX1700HS	UMESA6/UX1700HS	0.91

Approved details for the Tensar Welded Wire Form Wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- Though not shown in the approved plan sheets, if guard rail is to be placed at the top of the wall, the guard rail post shall either be installed through precut holes in the geogrid layers that must penetrated, or the geogrid layers shall be cut in a manner that prevents ripping or tearing of the geogrid.
- In plan sheets on pages 3, 4, 5, and 13, regarding the geotextiles shown,WSDOT reserves the right to require the use *Standard Specification* materials as specified in *Standard Specification* Section 9-33 that are similar to those specified in this plansheet.
- Regarding the plantable face alternate plan details on page 6, this alternativeshall only be considered approved if specifically called out in the contract specifications.
- Regarding the welded wire form and support strut details on page 7,galvanization is required per the contract specifications for all permanent walls.
- Regarding the geogrid penetration plan sheet detail on page 15, alternative 1 from Article 11.10.10.4 of AASHTO LRFD Bridge Design Specifications shall be followed to account for the portion of the geogrid layer cut through by the penetration. For penetration diameters larger than 30 inches or closer than 3 feet
- from the wall face, Alternative 2 in AASHTO LRFD Article 11.10.10.4 shall apply to accommodate the load transfer and to provide a stable wall face.
- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet for culvert penetration through the face and up to 2.5 feet for obstruction avoidance. Larger diameter culverts or obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers.
- This wall system is preapproved for both a welded wire/gravel fill face for vertical to near vertical facing batter, and welded wire vegetated face, provided aminimum horizontal step of 6 inches between each facing lift is used, effectively battering the wall face at 3V:1H or flatter. The horizontal step is necessary to reduce vertical stress on the relatively compressible topsoil placed immediately behind the facing so that settlement of the facing does notoccur.



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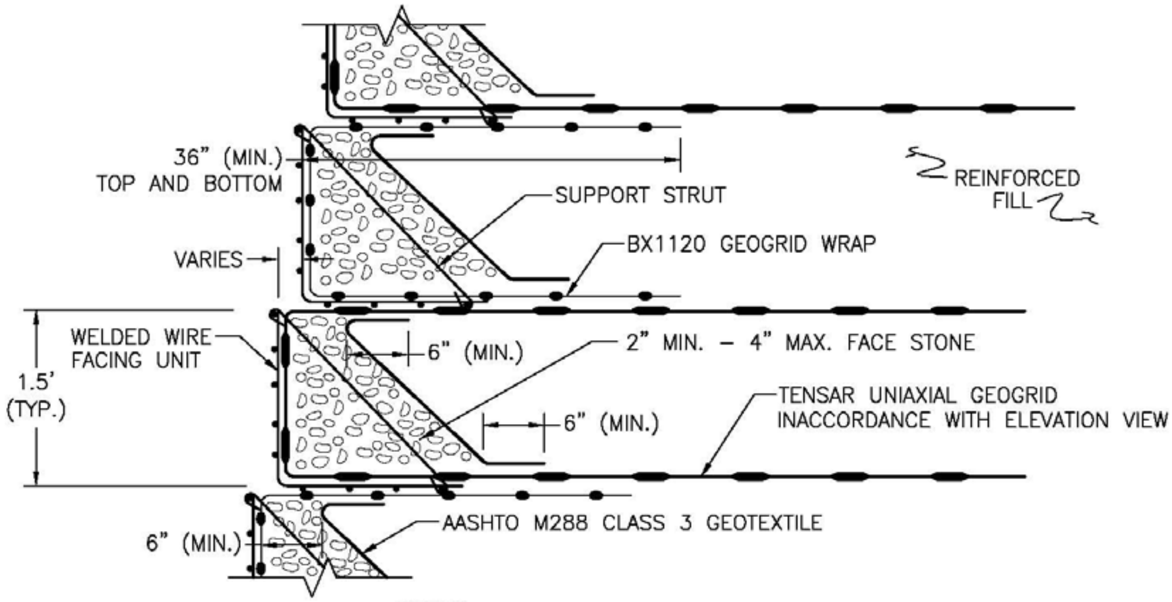
TYPICAL CROSS–SECTION



NOTES:

1. SEE WELDED WIRE FACING UNIT DETAIL FOR MATERIAL AND DIMENSIONS.
2. ALL FACING UNITS SHALL BE GALVANIZED AS PER ASTM A123 AFTER FABRICATION.
3. OPTIONAL THIN LAYER OF FINER STONE MAY BE PLACED AT THE TOP OF EACH UNIT TO PROVIDE A LEVEL SURFACE FOR THE UNIT ABOVE.

ALTERNATE WELDED WIRE FACING DETAIL

NOT TO SCALE

DESCRIPTION	ALTERNATE WELDED WIRE FACING DETAIL
FILE NAME:	WFD1e20305.DWG

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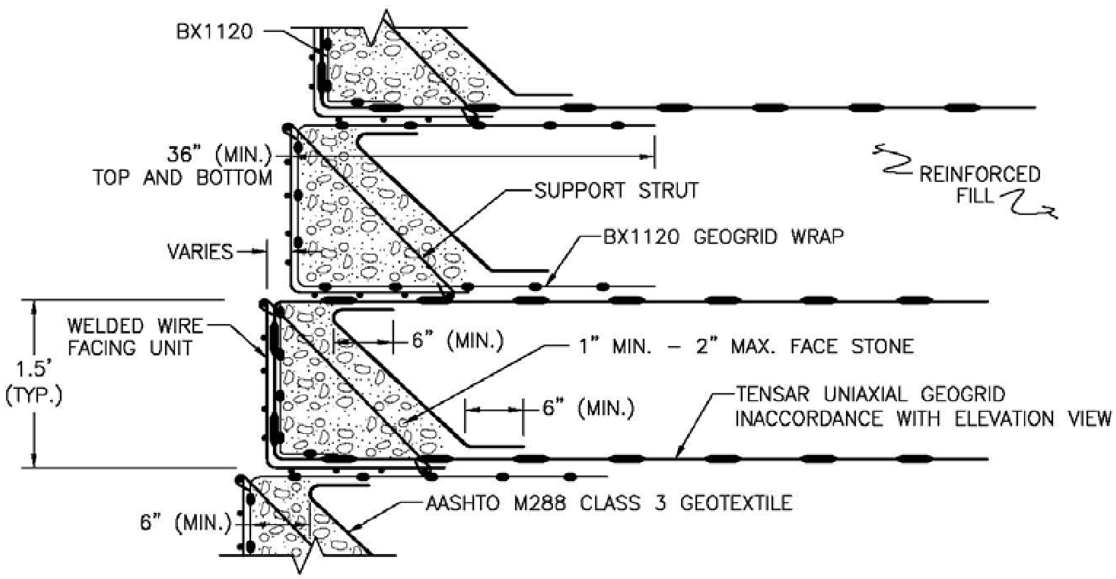
TYPICAL DETAIL



NOTES:
1. SEE WELDED WIRE FACING UNIT DETAIL FOR MATERIAL AND DIMENSIONS.
2. ALL FACING UNITS SHALL BE GALVANIZED AS PER ASTM A123 AFTER FABRICATION.

ALTERNATE WELDED WIRE FACING DETAIL (1" – 2" FACE FILL)

NOT TO SCALE

DESCRIPTION	ALTERNATE SIERRASCAPE FACING DETAIL (1"–2" FACE FILL)
FILE NAME:	WWFSS2e020305.DWG

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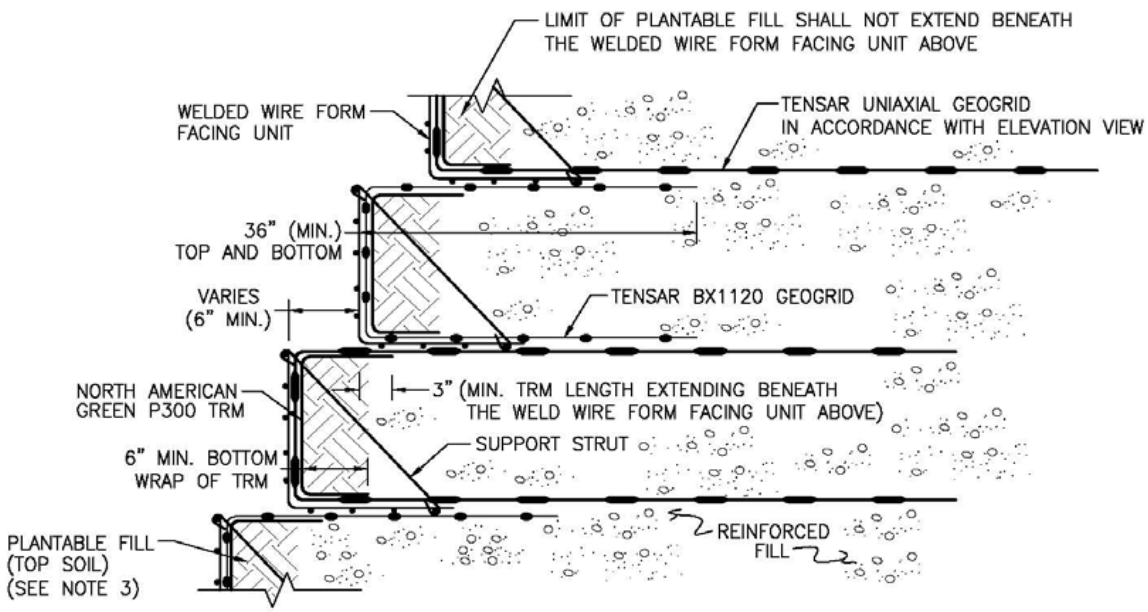
TYPICAL DETAIL



NOTES:

1. SEE WELDED WIRE FORM FACING UNIT DETAIL FOR FACING MATERIAL AND DIMENSIONS.
2. FACING UNITS SHALL BE CONSTRUCTED FROM BLACK STEEL.
3. PLANTABLE FILL OR TOP SOIL MAY BE PLACED AT THE FACE TO SUPPORT VEGETATION GROWTH.

ALTERNATE WELDED WIRE FORM FACING DETAIL (PLANTABLE FACE FILL)

NOT TO SCALE

DESCRIPTION	SIERRASCAPE FACING DETAIL
FILE NAME:	WWFSS3e20305.DWG

Tensar Earth Technologies Inc.

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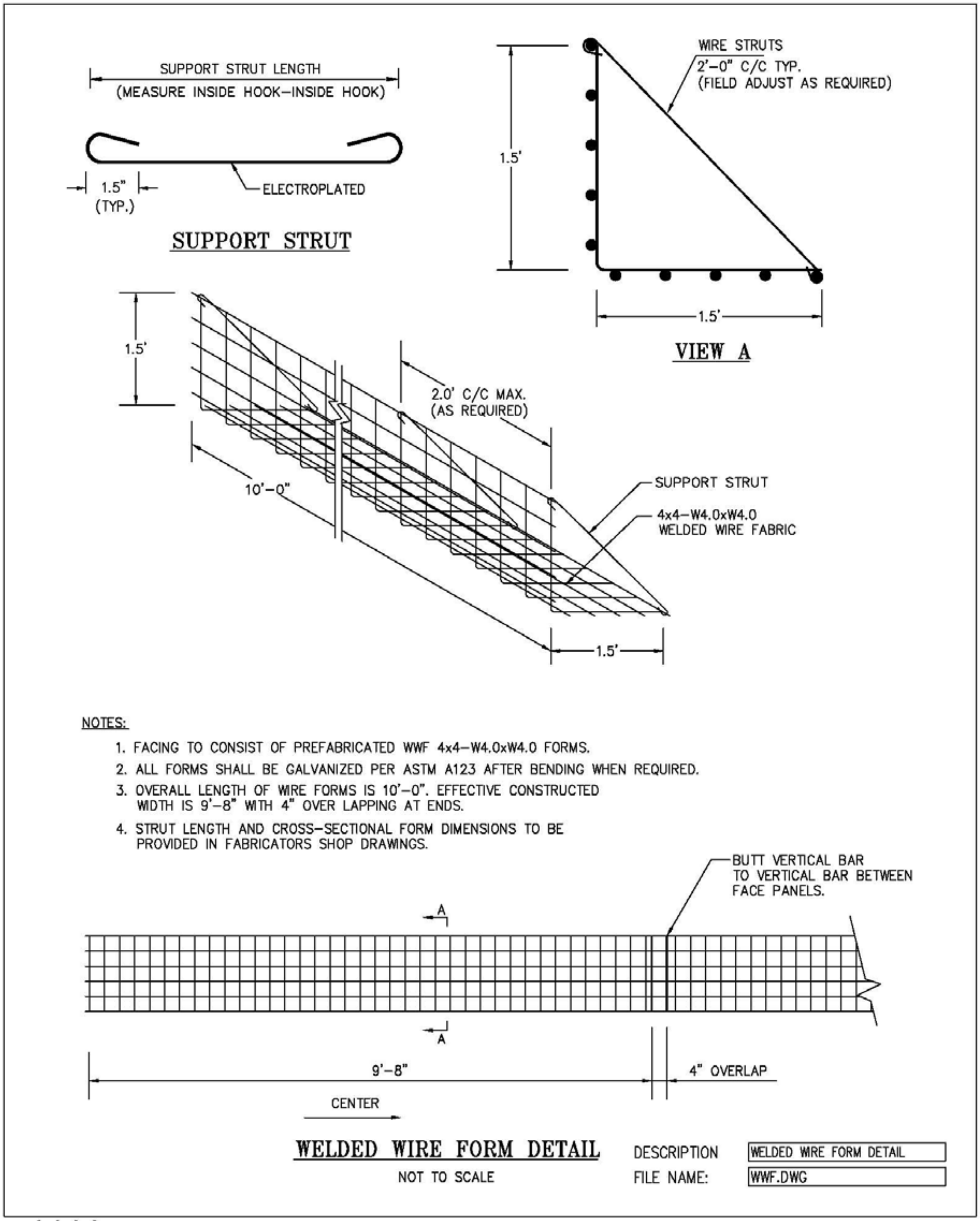
TYPICAL DETAIL



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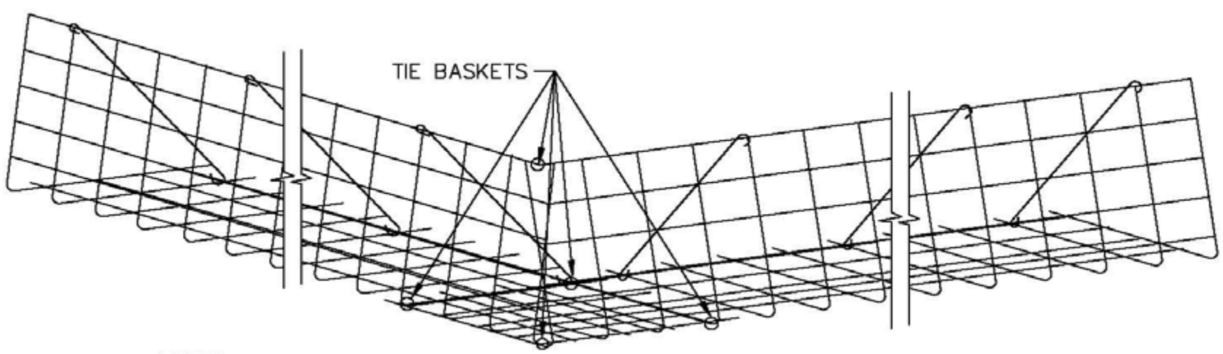
TYPICAL DETAIL



NOTES:

BEND OR CUT BASKETS TO FIT FIELD CONDITIONS

ENSURE THAT GEOTEXTILE AND BIAXIAL GEOGRID OVERLAP 1' MINIMUM

90° OUTSIDE CORNER DETAIL

NOT TO SCALE

DESCRIPTION	OUTSIDE CORNER DETAIL
FILE NAME:	WWFCD.DWG

Tensar Earth Technologies Inc.

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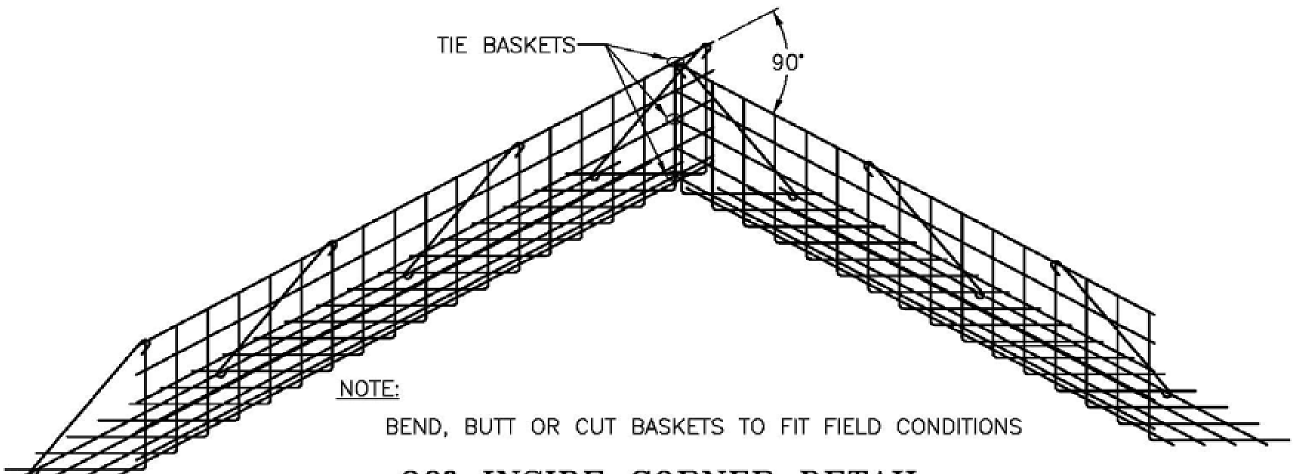
TYPICAL DETAIL



NOTE:

BEND, BUTT OR CUT BASKETS TO FIT FIELD CONDITIONS

90° INSIDE CORNER DETAIL

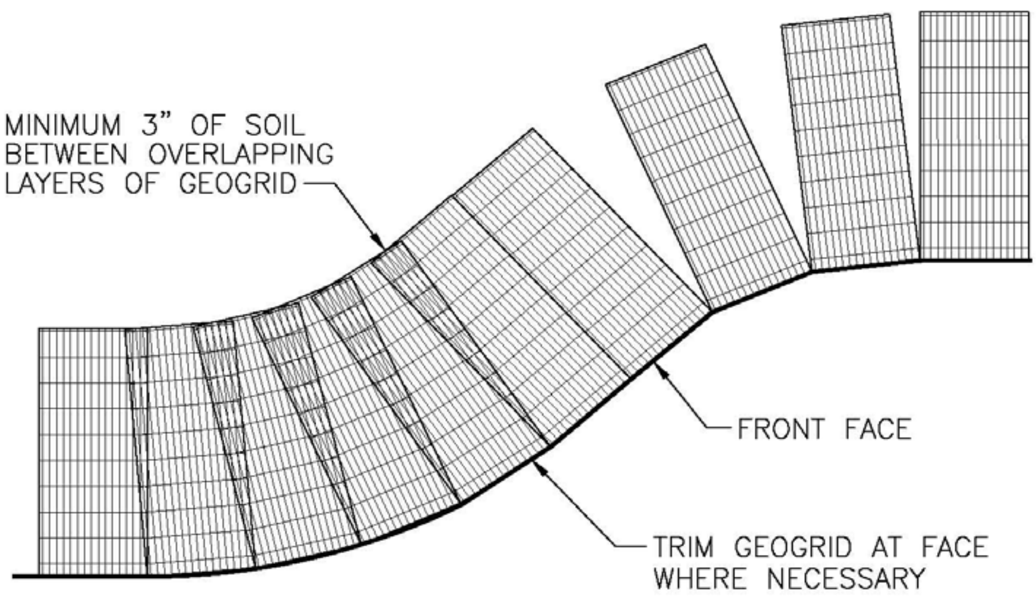
NOT TO SCALE

DESCRIPTION	INSIDE CORNER DETAIL
FILE NAME:	WWFCD2.DWG

Tensar Earth Technologies Inc.

Tensar

TYPICAL DETAIL



GEOGRID PLACEMENT ON CURVES

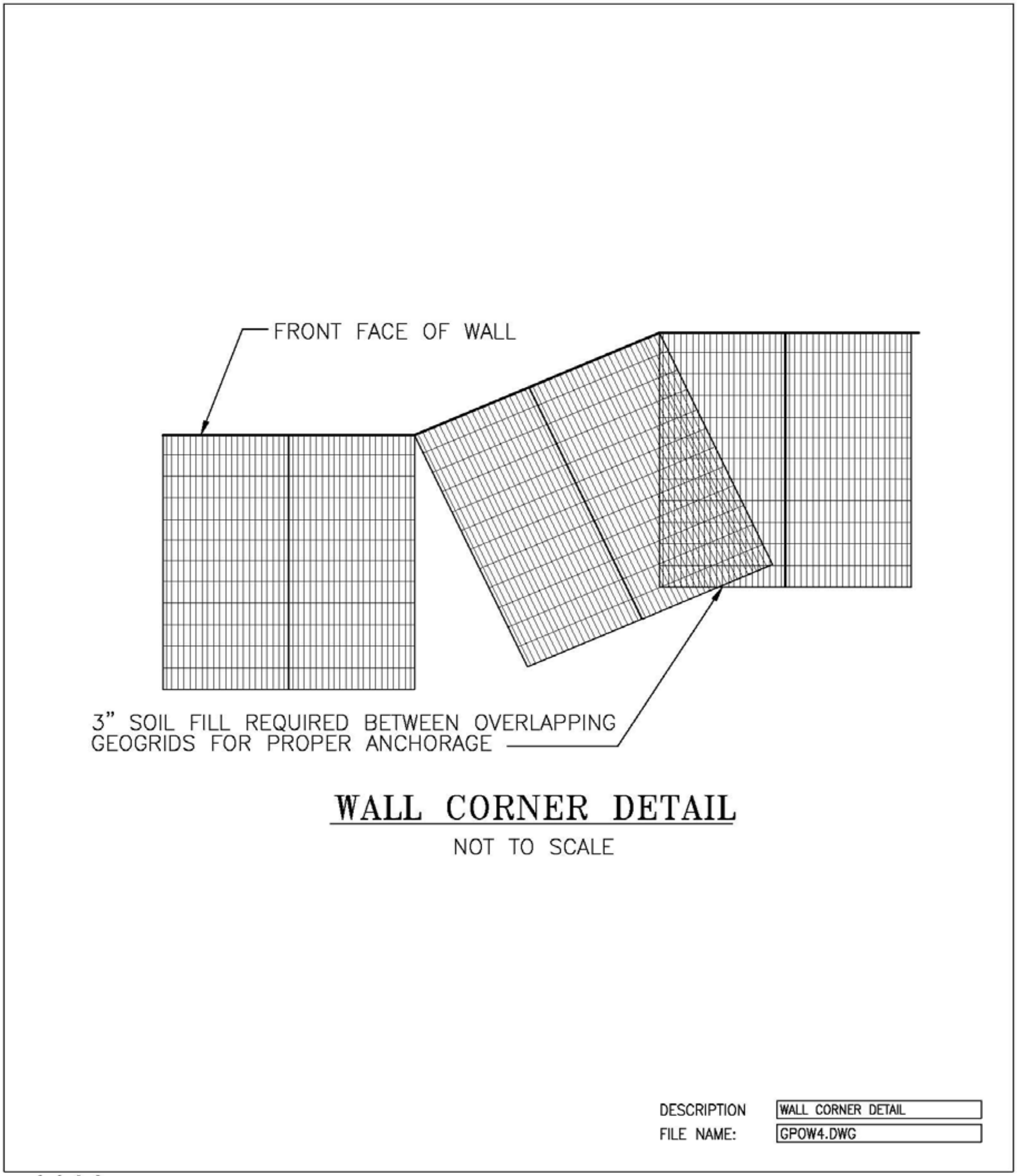
NOT TO SCALE

DESCRIPTION	GEOGRID PLACEMENT ON CURVES
FILE NAME:	GPOC2.DWG

Tensar Earth Technologies Inc.

Tensar

TYPICAL DETAIL



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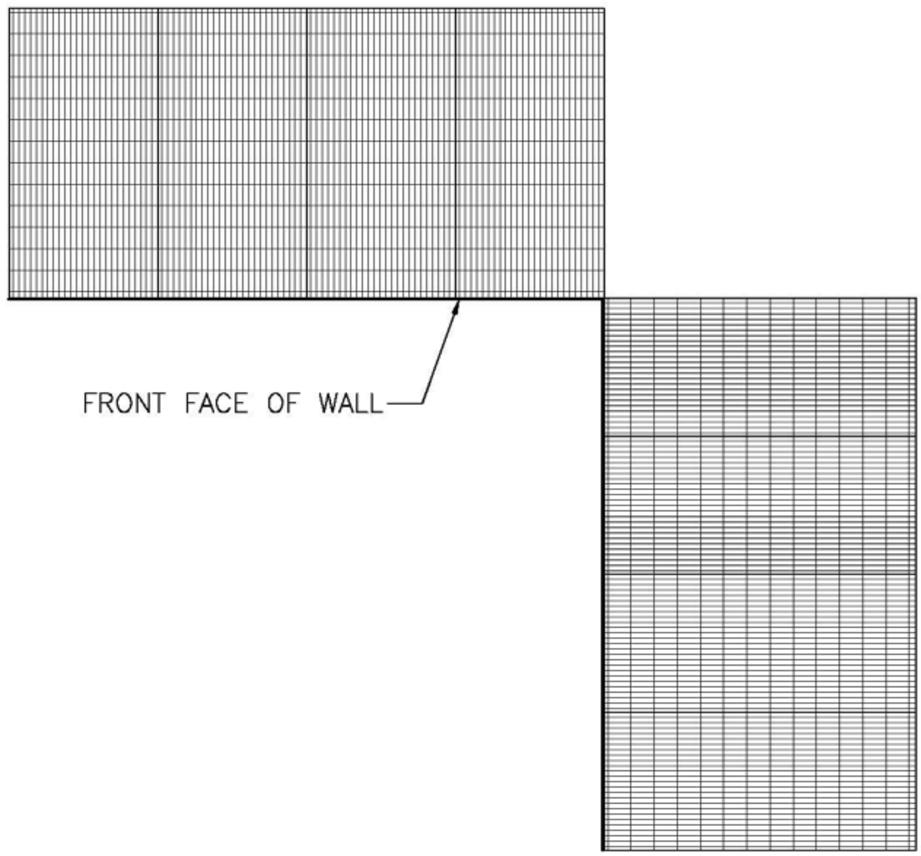
TYPICAL DETAIL



GEOGRID 90° CORNER DETAIL

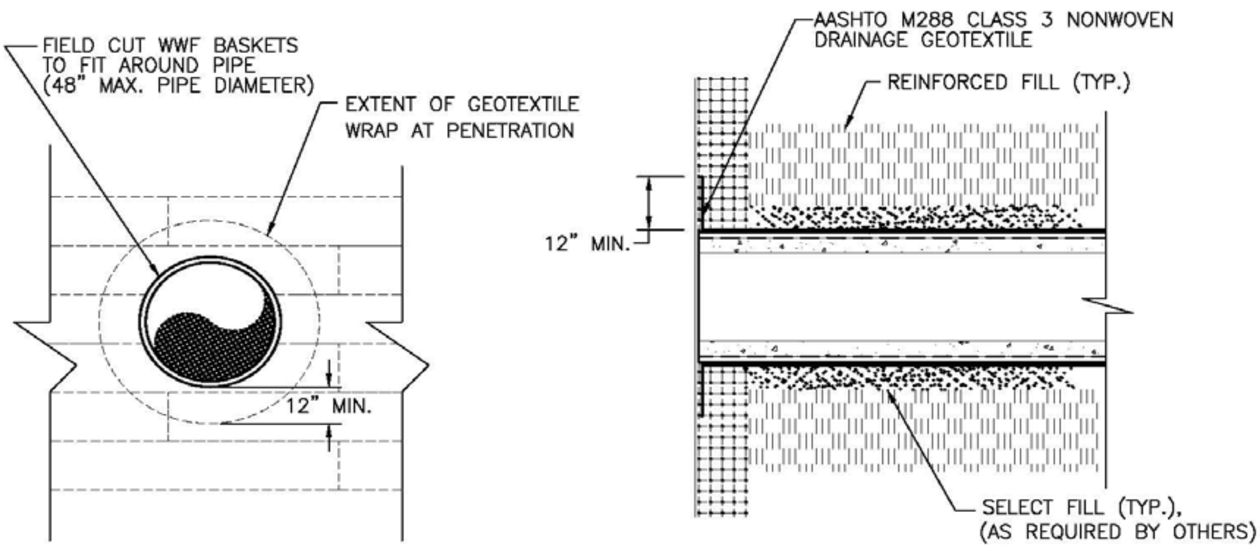
NOT TO SCALE

DESCRIPTION	GEOGRID 90° CORNER DETAIL
FILE NAME:	GPOW5.DWG

Tensar Earth Technologies Inc.

Tensar

TYPICAL DETAIL



ELEVATION VIEW

PLAN VIEW

NOTES:

1. CUT WIRE FACING AS CLOSE AS POSSIBLE TO PIPE PENETRATION.
2. CUT OR TERMINATE GEOGRIDS 3 INCHES OR LESS FROM PIPE.
3. WRAP ENTIRE PIPE WITH AASHTO M288 CLASS 3 NON-WOVEN DRAINAGE GEOTEXTILE. ENSURE THAT WRAP EXTENDS AT LEAST 12 INCHES BEHIND WIRE FACING AT PENETRATION TO ENSURE NO LOSS OF FILL.
4. FOR GEOGRID LAYOUT REFER TO ELEVATION VIEW FOR LENGTH, TYPE AND LOCATION

PIPE PENETRATION DETAIL AT WELDED WIRE FACE SYSTEM

NOT TO SCALE

DESCRIPTION	PIPE PENETRATION DETAIL AT WELDED WIRE FACE SYSTEM
FILE NAME:	GPP10.DWG

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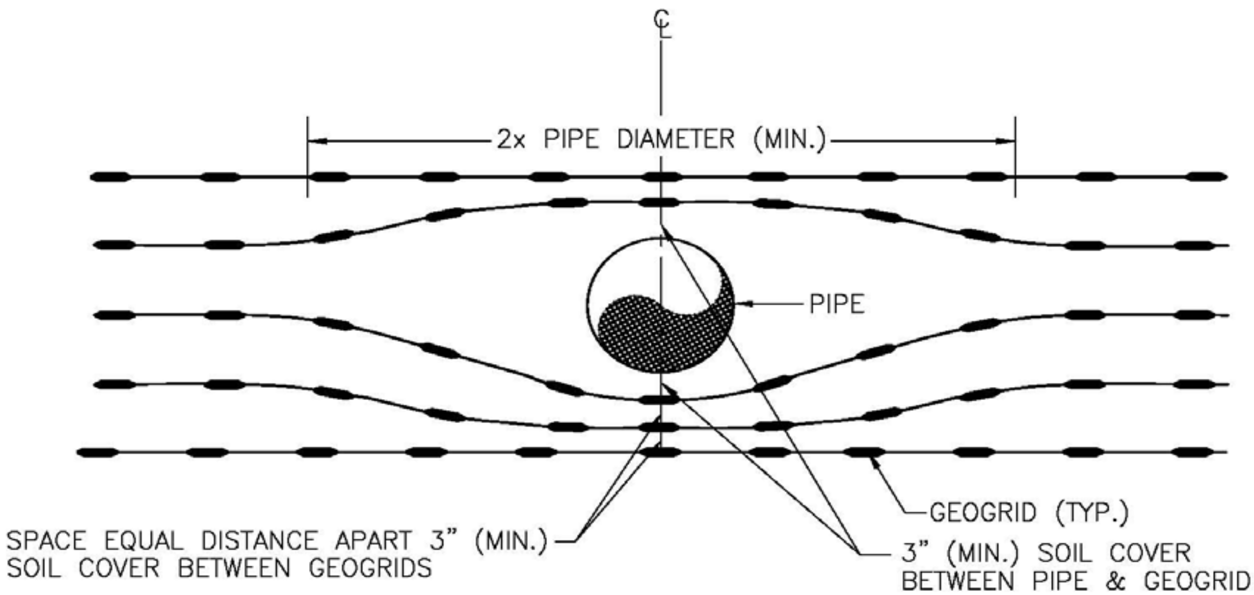
TYPICAL DETAIL



GEOGRID PLACEMENT AT PIPE

NOT TO SCALE

DESCRIPTION	GEOGRID PLACEMENT AT PIPE
FILE NAME:	GPP1.DWG

Tensar Earth Technologies Inc.

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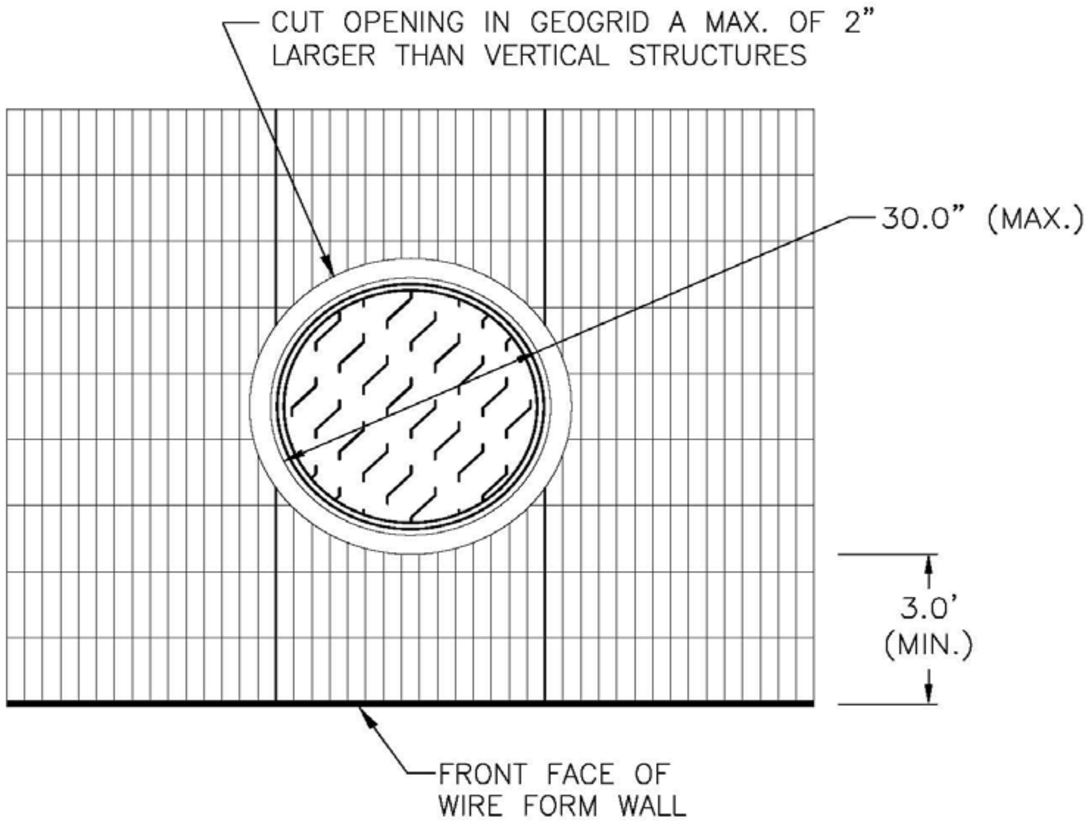
TYPICAL DETAIL



NOTE:

FOR OTHER CONDITIONS APPLY THE PROVISIONS OF ARTICLE 11.10.10.4 OF AASHTO LRFD SPECIFICATIONS.

GEOGRID PENETRATION

NOT TO SCALE

DESCRIPTION	GEOGRID PENETRATION
FILE NAME:	GP2.DWG

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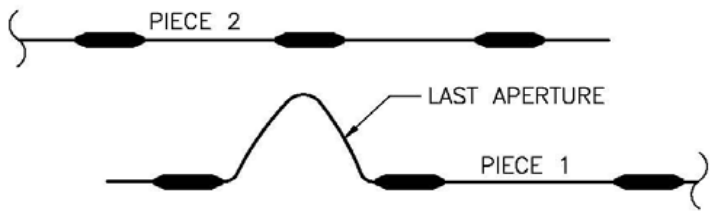
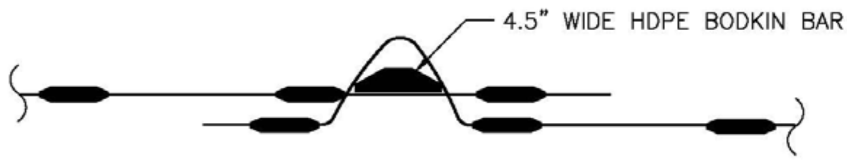
TYPICAL DETAIL

TO FORM A BODKIN CONNECTION:

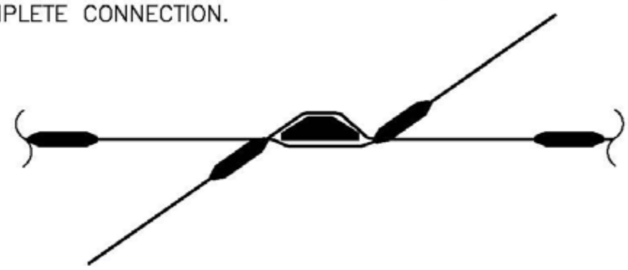
1. BEND THE LAST APERTURE OF ONE PIECE OF GEOGRID IN HALF.



2. PASS THE BENT APERTURES THROUGH THE APERTURES OF THE SECOND PIECE OF GEOGRID AND INSERT THE BODKIN BAR INTO THE CAVITY BETWEEN THE GEOGRIDS.



3. PULL BOTH PIECES OF GEOGRID IN OPPOSITE DIRECTIONS TO COMPLETE CONNECTION.



NOTE:

THE SPLICED GEOGRID PIECE ON EITHER SIDE OF THE BODKIN CONNECTION BE AT LEAST 6 FEET LONG UNLESS THE GEOGRID TERMINATES IN A FIXED CONNECTION.

BODKIN CONNECTION

NOT TO SCALE

DESCRIPTION	BODKIN CONNECTION
FILE NAME:	BC.DWG

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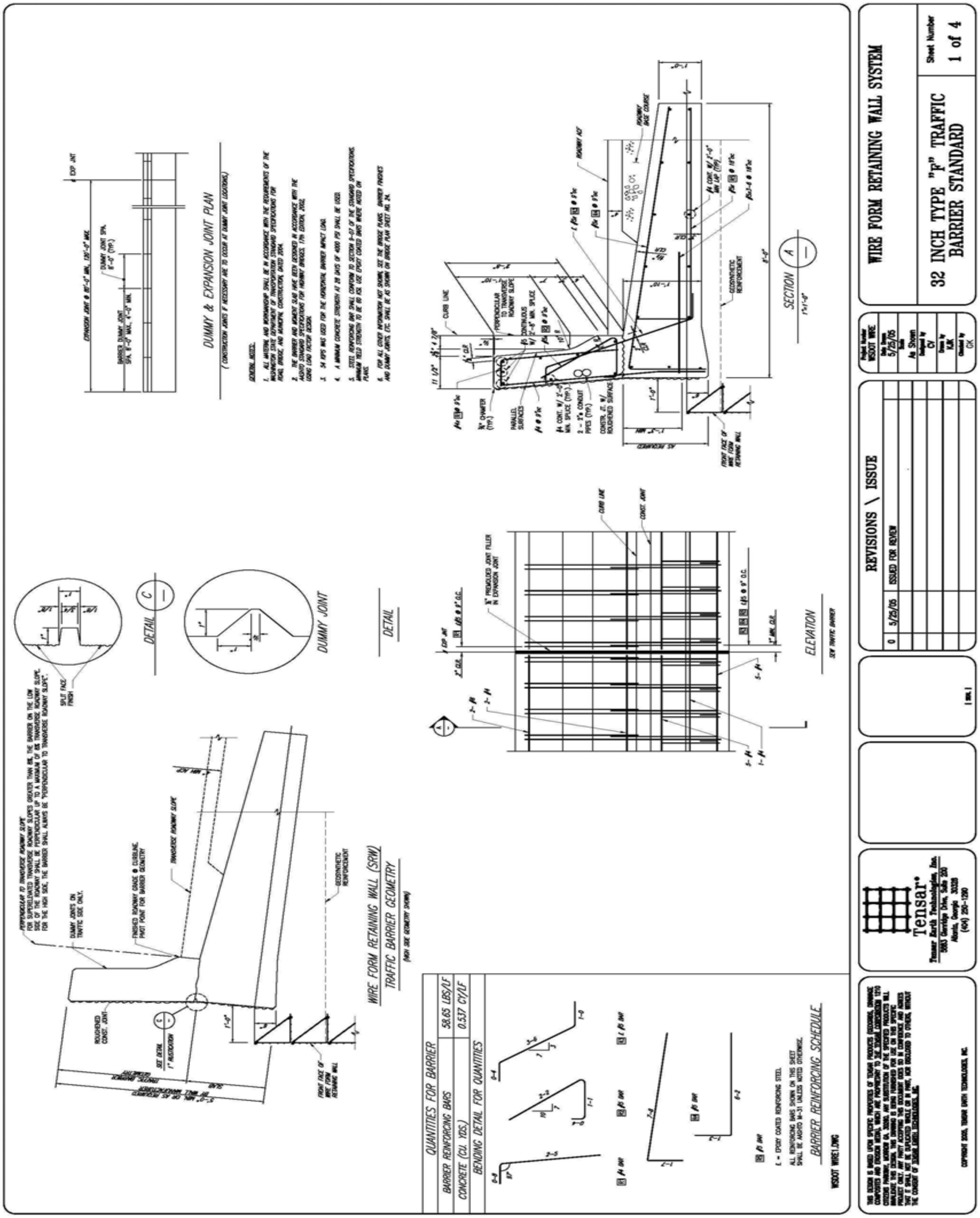
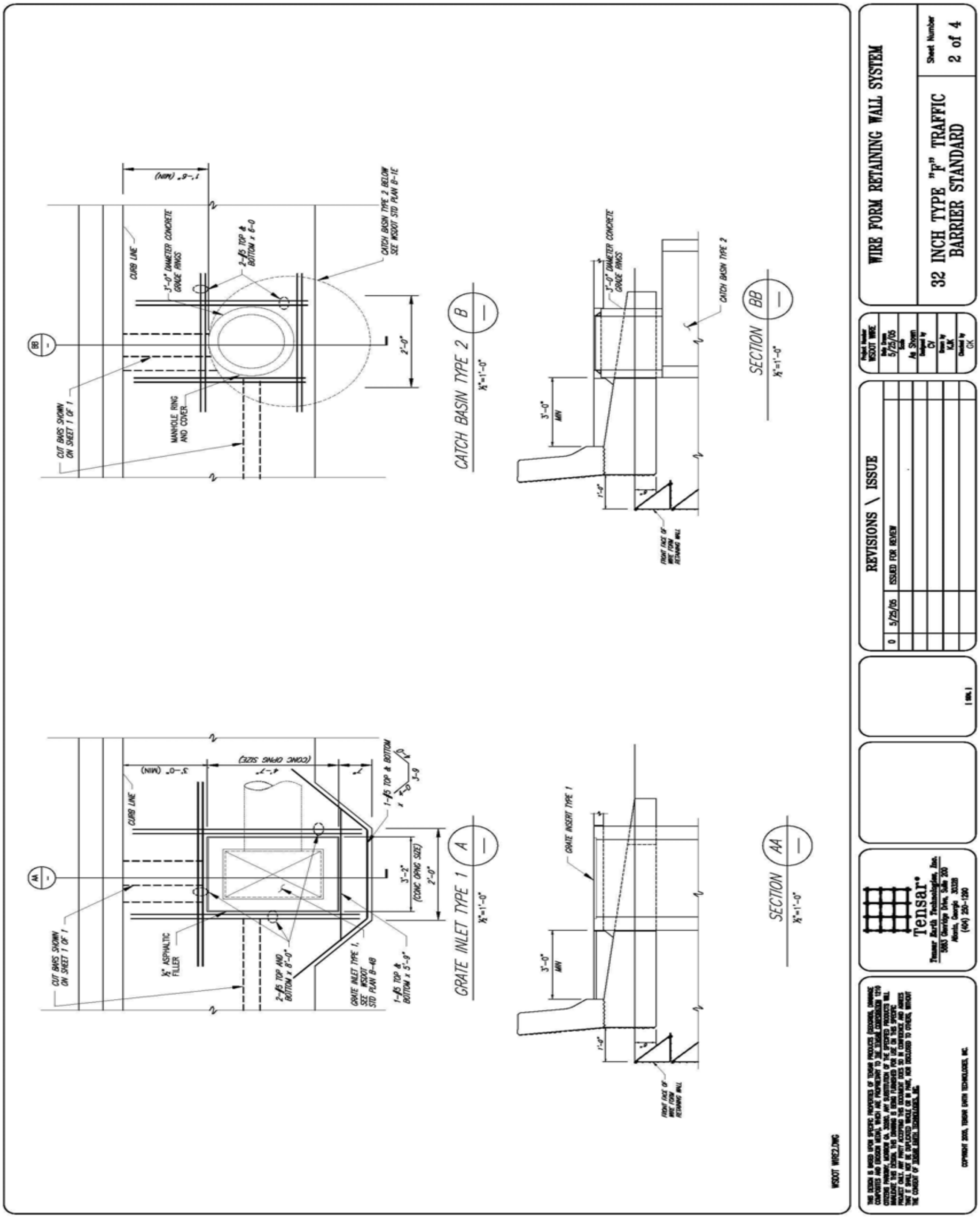
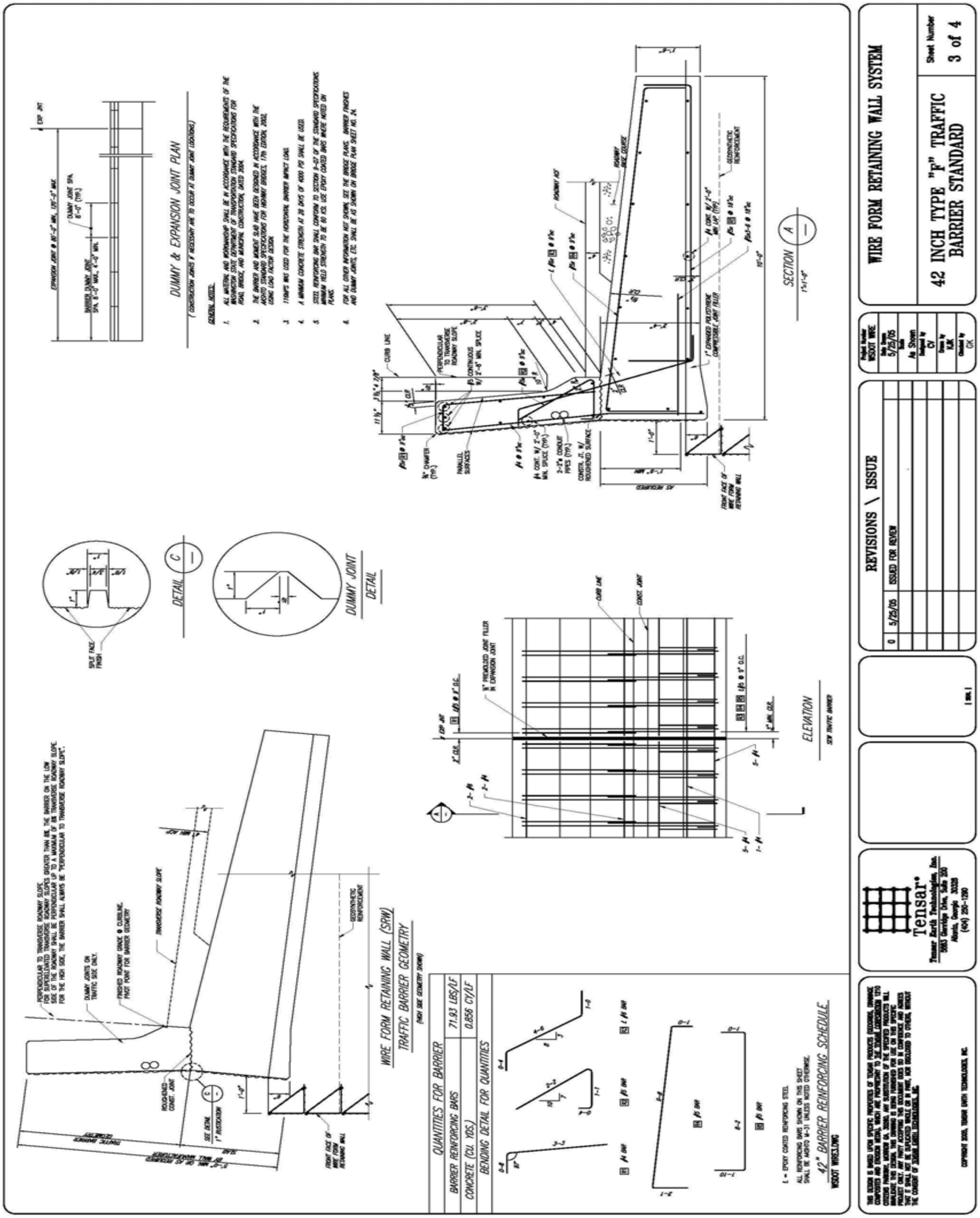
Tensar

TYPICAL DETAIL


WIRE FORM RETAINING WALL SYSTEM
32 INCH TYPE "F" TRAFFIC BARRIER STANDARD
Sheet Number
1 of 4
REVISIONS \ ISSUE
0 5/25/05 ISSUED FOR REVIEW
WIRE FORM RETAINING WALL (SRW) TRAFFIC BARRIER GEOMETRY
DUMMY & EXPANSION JOINT PLAN
QUANTITIES FOR BARRIER
BARRIER REINFORCING BARS 58.65 LBS/LF
CONCRETE (CU. YDS.) 0.537 CY/LF
BENDING DETAIL FOR QUANTITIES
BARRIER REINFORCING SCHEDULE
ELEVATION
SECTION A
DETAIL C
DUMMY JOINT
DETAIL
Tensar


WIRE FORM RETAINING WALL SYSTEM
32 INCH TYPE "F" TRAFFIC BARRIER STANDARD
Sheet Number
2 of 4
REVISIONS \ ISSUE
0 5/25/05 ISSUED FOR REVIEW
CATCH BASIN TYPE 2
SECTION BB
GRATE INLET TYPE 1
SECTION AA
CURB LINE
CUT BARS SHOWN ON SHEET 1 OF 1
MANHOLE RING AND COVER
GRATE INSERT TYPE 1
CATCH BASIN TYPE 2
Tensar
WSDOT WIREZLDWG


WIRE FORM RETAINING WALL SYSTEM
42 INCH TYPE "F" TRAFFIC BARRIER STANDARD
Sheet Number
3 of 4
REVISIONS \ ISSUE
0 5/25/05 ISSUED FOR REVIEW
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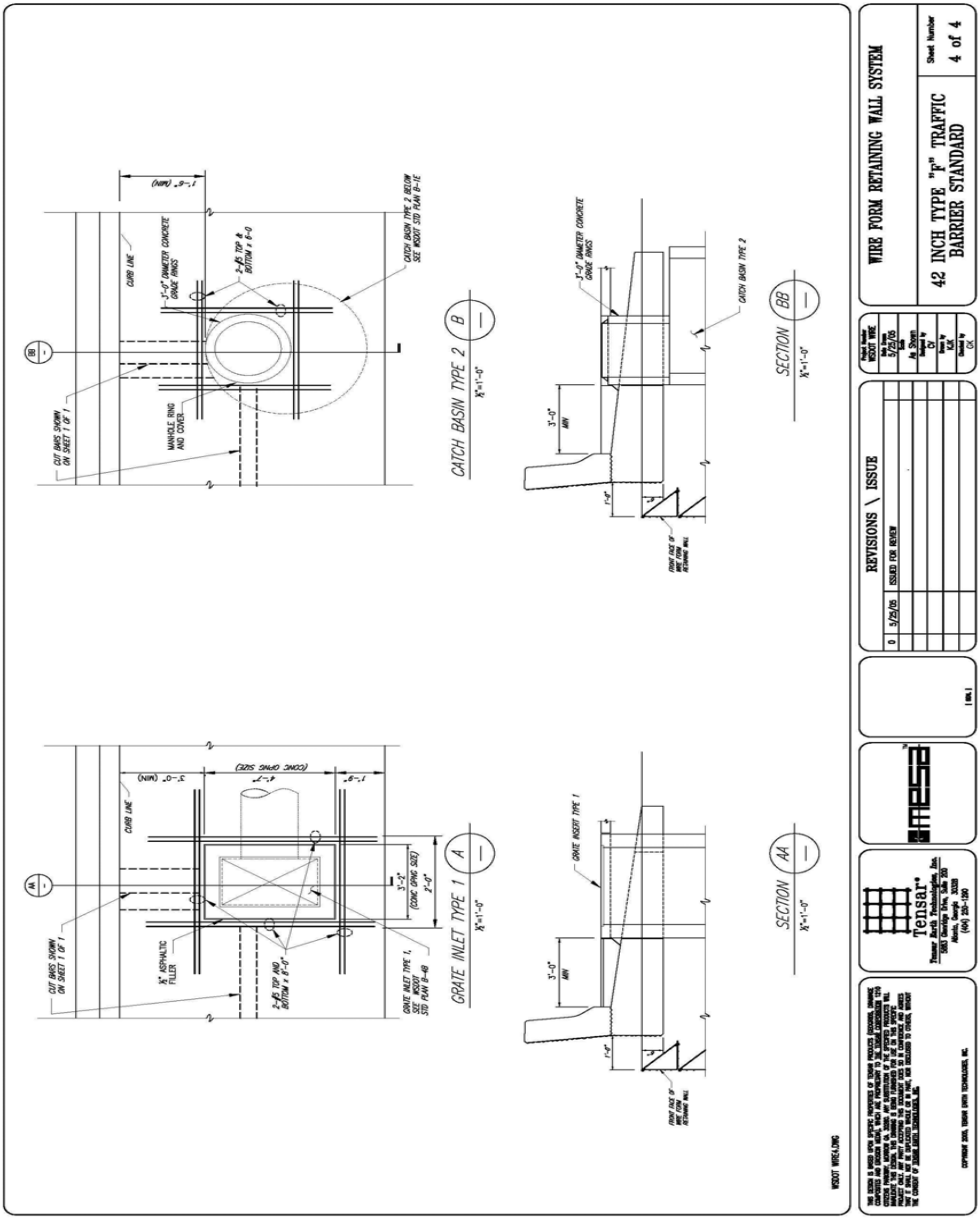
DUMMY & EXPANSION JOINT PLAN
GENERAL NOTES:
SECTION A
DETAIL C
DUMMY JOINT DETAIL
ELEVATION
WIRE FORM RETAINING WALL (SRW) TRAFFIC BARRIER GEOMETRY
QUANTITIES FOR BARRIER
BARRIER REINFORCING BARS 71.93 LBS/LF
CONCRETE (CU. YDS.) 0.856 CY/LF
BENDING DETAIL FOR QUANTITIES
42" BARRIER REINFORCING SCHEDULE


WIRE FORM RETAINING WALL SYSTEM
42 INCH TYPE "F" TRAFFIC BARRIER STANDARD
Sheet Number
4 of 4
REVISIONS \ ISSUE
0 5/25/05 ISSUED FOR REVIEW
CATCH BASIN TYPE 2
GRATE INLET TYPE 1
SECTION AA
SECTION BB
CUT BARS SHOWN ON SHEET 1 OF 1
CURB LINE
MANHOLE RING AND COVER
GRATE INSERT TYPE 1
CATCH BASIN TYPE 2
Tensar

Appendix 15-N Preapproved Wall Appendix: Specific Requirements and Details for SSL Concrete Panel Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply to the design of the SSL MSE Plus™ Retaining Wall:

The welded wire steel soil reinforcement shall be comprised of W11, W20, or W24 smooth wire as shown and noted in the preapproved SSL MSEPlus wall system drawings. Deformed bars shall not be used for soil reinforcement. As SSL has committed to always supply soil reinforcement steel with a minimum yield strength of 75 ksi, the soil reinforcement steel shall be designed for a yield strength, Fy, of 75 ksi, which is greater than the minimum yield strength specified in ASTM A82. Because the yield strength is greater than the minimum yield strength allowed by ASTM A82, as a minimum, the yield strength of the steel shipped to the project site will be verified that it meets the minimum Fy of 75 ksi through the tensile test results for the as delivered material, and WSDOT reserves the right to conduct its own tensile tests to verify the steel yield strength.

The design of the connection between the facing panels and the soil reinforcement shall meet the AASHTO LRFD Bridge Design Specification requirements. To determine the connection strength, the following values of the short-term (i.e., uncorroded) connection strength ratio CRu shall be used:

Welded Wire Soil Reinforcement Wire Size Short-Term Connection Strength Ratio, CR_u

Welded Wire Soil Reinforcement Wire Size	Short-Term Connection Strength Ration, CR_u
W11	0.98
W20	0.87
W24	0.96

Minimum bend radii for the welded wire soil reinforcement shall be as shown in the preapproved plans (sheet 4 of 15 titled “Standard Details 3 of 3”).

Reinforcement pullout shall be calculated based on the default values for steel grid reinforcement provided in the AASHTO Specifications. If, at some future time product and soil specific pullout data is provided to support use of non-default pullout interaction coefficients, it should be noted that LRFD pullout resistance design using these product and soil specific interaction coefficients has not been calibrated using product specific data statistics and reliability theory. Therefore, the specified resistance factors in the GDM and AASHTO LRFD Specifications should not be considered applicable to product specific pullout interaction coefficients.

Approved details for the SSL MSE PlusTM wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- In plan sheet 4 of 10, regarding the filter fabric shown, the use *Standard Specification* materials as specified in *Standard Specification* M 41-10 Section 9-33 that are similar to those specified in this plan sheet shall be used.
- In plan sheets 4 of 15, 2 of 10, and 5 of 10, there should be a minimum cover of 4 inches of soil between the steel grid and the traffic barrier reaction slab.

Quality control of the materials used in the SSL MSEPlus wall system shall meet the requirements in the *SSL Quality Control Manual*, Revision 4, dated 5/31/2012.

SPECIALIZING IN CONSTRUCTION PRODUCTS

SSL

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4740 Scotts Valley Drive, Suite "E"
Scotts Valley, California 95066
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FAX 831/430-9340

INDEX OF DRAWINGS

SHEET	DESCRIPTION
SD-01	TITLE & INDEX
SD-02	STANDARD DETAILS: 1 OF 3
SD-03	STANDARD DETAILS: 2 OF 3
SD-04	STANDARD DETAILS: 3 OF 3
SD-05	CORNER PANEL DETAILS
SD-06	DRAINAGE DETAILS: 1 OF 2
SD-07	DRAINAGE DETAILS: 2 OF 2
SD-08	PENETRATION DETAILS
SD-09	HANDLING AND UNLOADING DETAILS
SD-10	ERECTION SEQUENCE
SD-11	PANEL REBAR DETAILS: 1 OF 5
SD-12	PANEL REBAR DETAILS: 2 OF 5
SD-13	PANEL REBAR DETAILS: 3 OF 5
SD-14	PANEL REBAR DETAILS: 4 OF 5
SD-15	PANEL REBAR DETAILS: 5 OF 5

CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES

DANIEL MITCHELL STATE OF WASHINGTON 47993 REGISTERED PROFESSIONAL ENGINEER

DANIEL MITCHELL
05/13/13

SPECIALIZING IN CONSTRUCTION PRODUCTS SSL
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THE DESIGN CONTAINED ON THESE DRAWINGS IS BASED ON INFORMATION PROVIDED BY THE OWNER. ON THE BASIS OF THIS INFORMATION, SSL HAS DESIGNED THE STRUCTURE(S) SHOWN AND IS RESPONSIBLE FOR INTERNAL STABILITY ONLY. THE OWNER REMAINS RESPONSIBLE FOR EXTERNAL STABILITY INCLUDING FOUNDATION (BEARING CAPACITY AND SETTLEMENT) AND SLOPE STABILITY (SLIDING AND ROTATION).

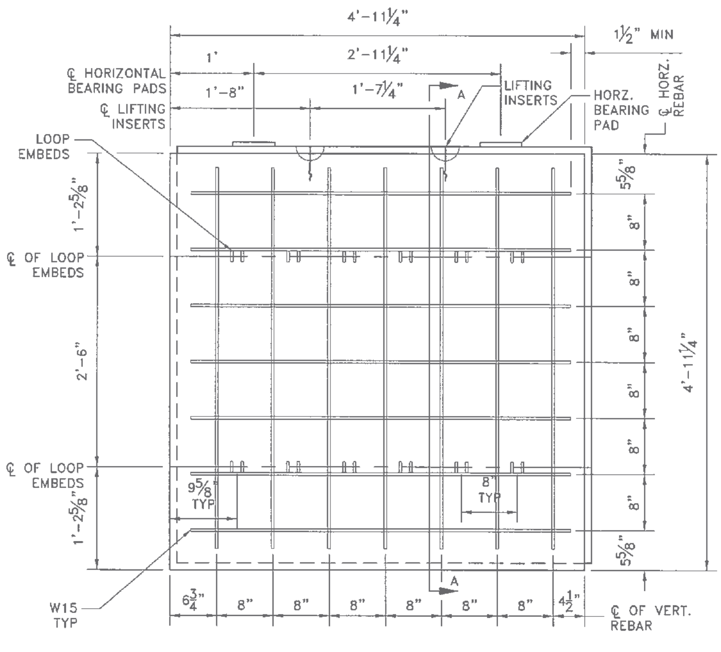
DRAWN BY: AJM	08/13/12
CHECK BY: GTF	08/13/12
DESIGN BY: DSM	08/13/12
CHECK BY: AJM	08/13/12
PROJ. ENGR: DSM	

NO.	DATE	REVISION DESCRIPTION	BY

TITLE & INDEX

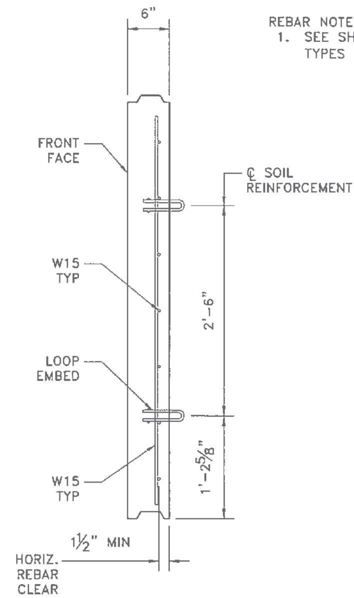
5X5 PANEL WITH TONGUE AND GROOVE
STANDARD DETAILS AND NOTES
STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION

Scale: NTS
Job:
Sheet: SD-01
Sheet 01 of 15



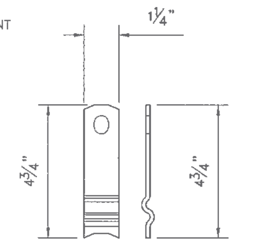
STANDARD "A" PANEL

SHOWN FROM BACK FACE



SECTION A–A

REBAR NOTE:
1. SEE SHEET SD–04 FOR USAGE OF PANEL TYPES "A", "X" AND "Y".



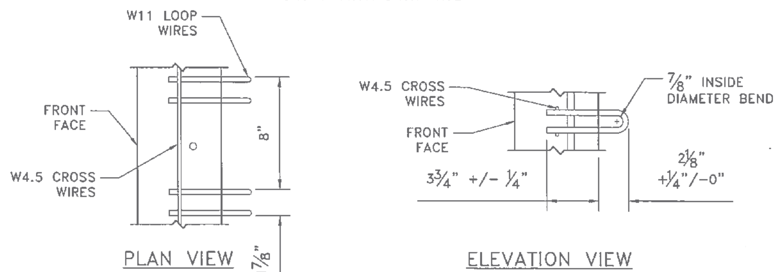
NOTE: LIFTING INSERT SHOWN IS TYPICAL. OTHER LIFTING INSERTS WITH CAPACITY EQUAL OR GREATER THAN WHAT IS SPECIFIED IN THE CASTING DRAWINGS MAY BE SUBSTITUTED.

LIFTING INSERT DETAIL

PANEL REINFORCEMENT TABLE

Panel Type	Minimum Vert. Area	Minimum Horz. Area
A	0.88 in²	0.99 in²
A2	0.99 in²	0.55 in²
X	1.40 in²	1.40 in²
X2	1.40 in²	0.77 in²
Y	1.76 in²	1.10 in²
Y2	1.76 in²	0.55 in²

9" MAX SPACING BETWEEN BARS
PANEL REINFORCEMENT MAY BE DEFORMED BARS, SMOOTH WIRE OR DEFORMED WIRE



LOOP EMBED DETAILS

PANEL TOLERANCES:

OVERALL DIMENSIONS:
- STANDARD PANEL
  ± 3/16" VERTICAL
  ± 3/16" HORIZONTAL
- TOP AND SPECIAL PANELS
  ± 3/16" VERTICAL
  ± 3/16" HORIZONTAL

CONNECTION DEVICE LOCATIONS:
- EMBEDS
  ± 1" VERTICAL
  ± 1" HORIZONTAL

PANEL SQUARENESS:
- 90° PANEL CORNERS
  ± 3⁄16" USING 2' SQUARE
  (MEASURE 3 PANEL CORNERS)

PANEL DIAGONAL:
- PANELS WITH 90° CORNERS
  ½" MAX. DIFFERENCE BETWEEN DIAGONALS

SURFACE FINISH:
- FINISH AT FRONT FACE
  ± ⅛" IN 5'

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DANIEL MITCHELL
STATE OF WASHINGTON
47993
REGISTERED PROFESSIONAL ENGINEER

DANIEL MITCHELL
05/13/13

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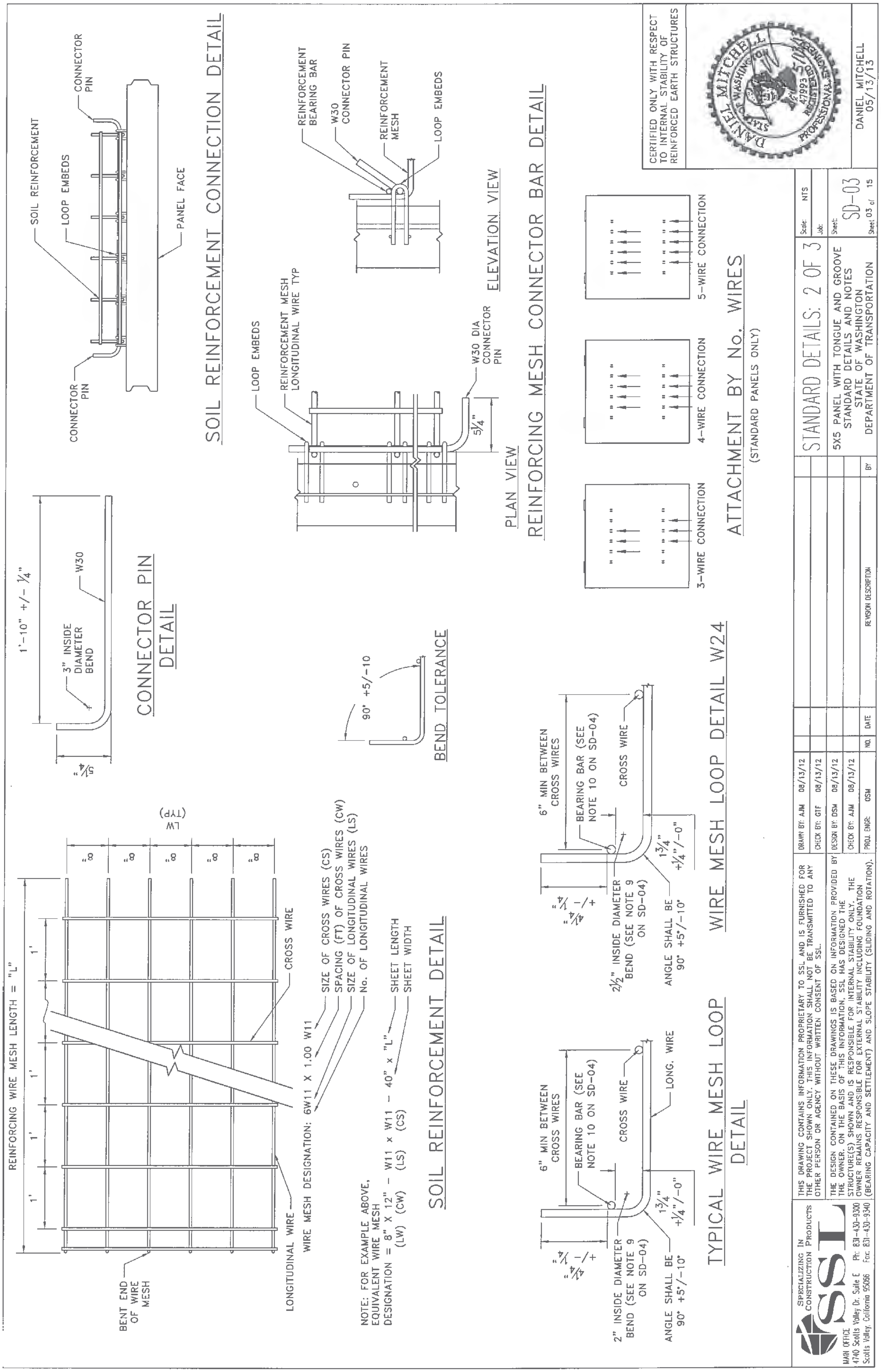
STANDARD DETAILS: 1 OF 3

5X5 PANEL WITH TONGUE AND GROOVE
STANDARD DETAILS AND NOTES
STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION

Scale: NTS
Job:
Sheet: SD–02
Sheet 02 of 15



SOIL REINFORCEMENT CONNECTION DETAIL
SOIL REINFORCEMENT
LOOP EMBEDS
PANEL FACE
CONNECTOR PIN
REINFORCING WIRE MESH LENGTH = "L"
BENT END OF WIRE MESH
LONGITUDINAL WIRE
CROSS WIRE
WIRE MESH DESIGNATION: 6W11 X 1.00 W11
SIZE OF CROSS WIRES (CS)
SPACING (FT) OF CROSS WIRES (CW)
SIZE OF LONGITUDINAL WIRES (LS)
No. OF LONGITUDINAL WIRES
NOTE: FOR EXAMPLE ABOVE, EQUIVALENT WIRE MESH DESIGNATION = 8" X 12" - W11 x W11 - 40" x "L"
(LW) (CW) (LS) (CS)
SHEET LENGTH
SHEET WIDTH
LW (TYP)
SOIL REINFORCEMENT DETAIL
1'-10" +/- 1/4"
3" INSIDE DIAMETER BEND
W30
5 1/4"
CONNECTOR PIN DETAIL
90° +5/-10
BEND TOLERANCE
LOOP EMBEDS
REINFORCEMENT MESH LONGITUDINAL WIRE TYP
W30 DIA CONNECTOR PIN
PLAN VIEW
REINFORCEMENT BEARING BAR
W30 CONNECTOR PIN
REINFORCEMENT MESH
ELEVATION VIEW
REINFORCING MESH CONNECTOR BAR DETAIL
3-WIRE CONNECTION
4-WIRE CONNECTION
5-WIRE CONNECTION
ATTACHMENT BY No. WIRES
(STANDARD PANELS ONLY)
6" MIN BETWEEN CROSS WIRES
BEARING BAR (SEE NOTE 10 ON SD-04)
CROSS WIRE
LONG. WIRE
2" INSIDE DIAMETER BEND (SEE NOTE 9 ON SD-04)
ANGLE SHALL BE 90° +5°/-10°
1 3/4" +1/4"/-0"
TYPICAL WIRE MESH LOOP DETAIL
2 1/2" INSIDE DIAMETER BEND (SEE NOTE 9 ON SD-04)
WIRE MESH LOOP DETAIL W24
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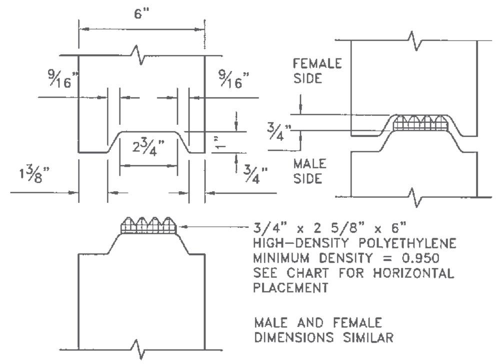
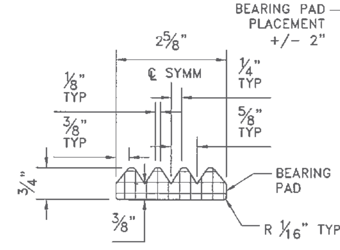
STANDARD DETAILS: 2 OF 3
5X5 PANEL WITH TONGUE AND GROOVE STANDARD DETAILS AND NOTES
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION
Scale: NTS
Job:
Sheet: SD-03
Sheet 03 of 15

MATERIAL PROPERTIES NOTES:

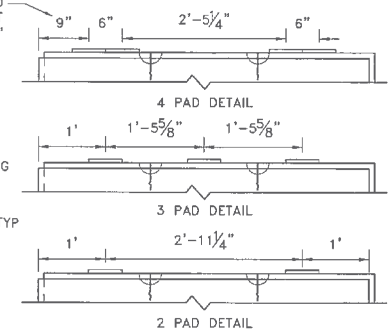
1. PANEL REINFORCEMENT BARS SHALL BE DEFORMED BILLET STEEL BARS FOR CONCRETE REINFORCEMENT CONFORMING TO THE SPECIFICATION OF ASTM DESIGNATION A615, GRADE 60, INCLUDING SUPPLEMENTARY REQUIREMENT S1 OR LOW ALLOY STEEL DEFORMED BARS CONFORMING TO THE SPECIFICATIONS OF ASTM DESIGNATION A706. STRUCTURAL WELDED WIRE REINFORCEMENT THAT CONFORMS TO ASTM A185/A497 OR SMOOTH SPECIFICATIONS MAY BE SUBSTITUTED FOR ASTM DESIGNATION A615.
2. W11, W20 & W24 STEEL WIRE FOR SOIL REINFORCEMENT SHALL CONFORM TO THE ASTM DESIGNATION A82. W4.5 & W11 FOR THE LOOP EMBEDS SHALL CONFORM TO THE ASTM DESIGNATION A82. W30 FOR THE CONNECTION PINS SHALL CONFORM TO THE ASTM DESIGNATION A82. THE WELDED WIRE SOIL REINFORCEMENT AND LOOP EMBED SHALL BE WELDED IN ACCORDANCE WITH ASTM DESIGNATION A185. ALL SOIL REINFORCEMENT MESH SHALL BE COMPOSED OF SMOOTH WIRE, DEFORMED WIRE SHALL NOT BE USED FOR SOIL REINFORCEMENT, LOOP EMBEDS AND CONNECTION PIN.
3. THE LOOP EMBEDS, SOIL REINFORCEMENT AND CONNECTION PINS SHALL BE GALVANIZED IN ACCORDANCE WITH ASTM DESIGNATION A123 AFTER BENDING.
4. CONCRETE PANELS TO HAVE A 28-DAY COMPRESSIVE STRENGTH OF 4000 PSI.
5. ALL PANEL REINFORCEMENT MUST HAVE A MINIMUM OF 1 1/2" COVERAGE WITH CONCRETE ON ALL SIDES.
6. FOR PANELS WITH W11 SOIL REINFORCEMENT USE "A" PANEL REINFORCEMENT. PANELS WITH W20 SOIL REINFORCEMENT USE "X" PANEL REINFORCEMENT. PANELS WITH W24 SOIL REINFORCEMENT USE "Y" PANEL REINFORCEMENT.
7. THE MOLDED PLASTIC PANEL PADS SHALL BE COMPOSED OF A HIGH DENSITY POLYETHYLENE MATERIAL, SHALL HAVE A MINIMUM TENSILE STRENGTH OF 4 KSI AND A MINIMUM TENSILE ELONGATION OF 600 PERCENT.
8. FILTER FABRIC IS A NON-WOVEN GEOTEXTILE COMPOSED OF POLYPROPYLENE FIBERS, WHICH ARE FORMED INTO A STABLE NETWORK SUCH THAT THE FIBERS RETAIN THEIR RELATIVE POSITION. FILTER FABRIC IS INERT TO BIOLOGICAL DEGRADATION AND RESISTS NATURALLY ENCOUNTERED CHEMICALS, ALKALIS AND ACIDS.
9. THE MINIMUM INSIDE BEND DIAMETER FOR W11 AND W20 WIRE USED FOR SOIL REINFORCEMENT SHALL BE NO LESS THAN TWICE THE NOMINAL DIAMETER OF THE WIRE SIZE AND IN NO INSTANCE BE LESS THAN 1 INCH. THE INSIDE BEND DIAMETER FOR W24 WIRE USED FOR SOIL REINFORCEMENT SHALL BE 2 1/2 INCHES.
10. THE BEARING BAR AND THE CROSS WIRES SHALL BE THE SAME SIZE.



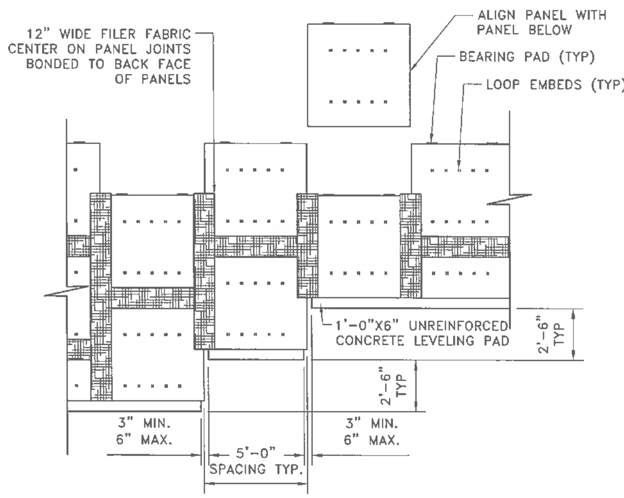
HORIZONTAL PANEL JOINT DETAIL



BEARING PAD DETAIL

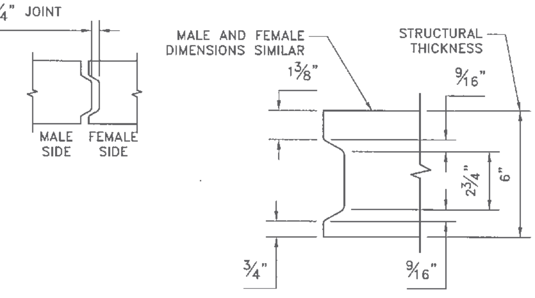


BEARING PAD PLACEMENT



FILTER FABRIC DETAIL AND PANEL PLACEMENT

(BACK VIEW)



VERTICAL PANEL JOINT DETAIL

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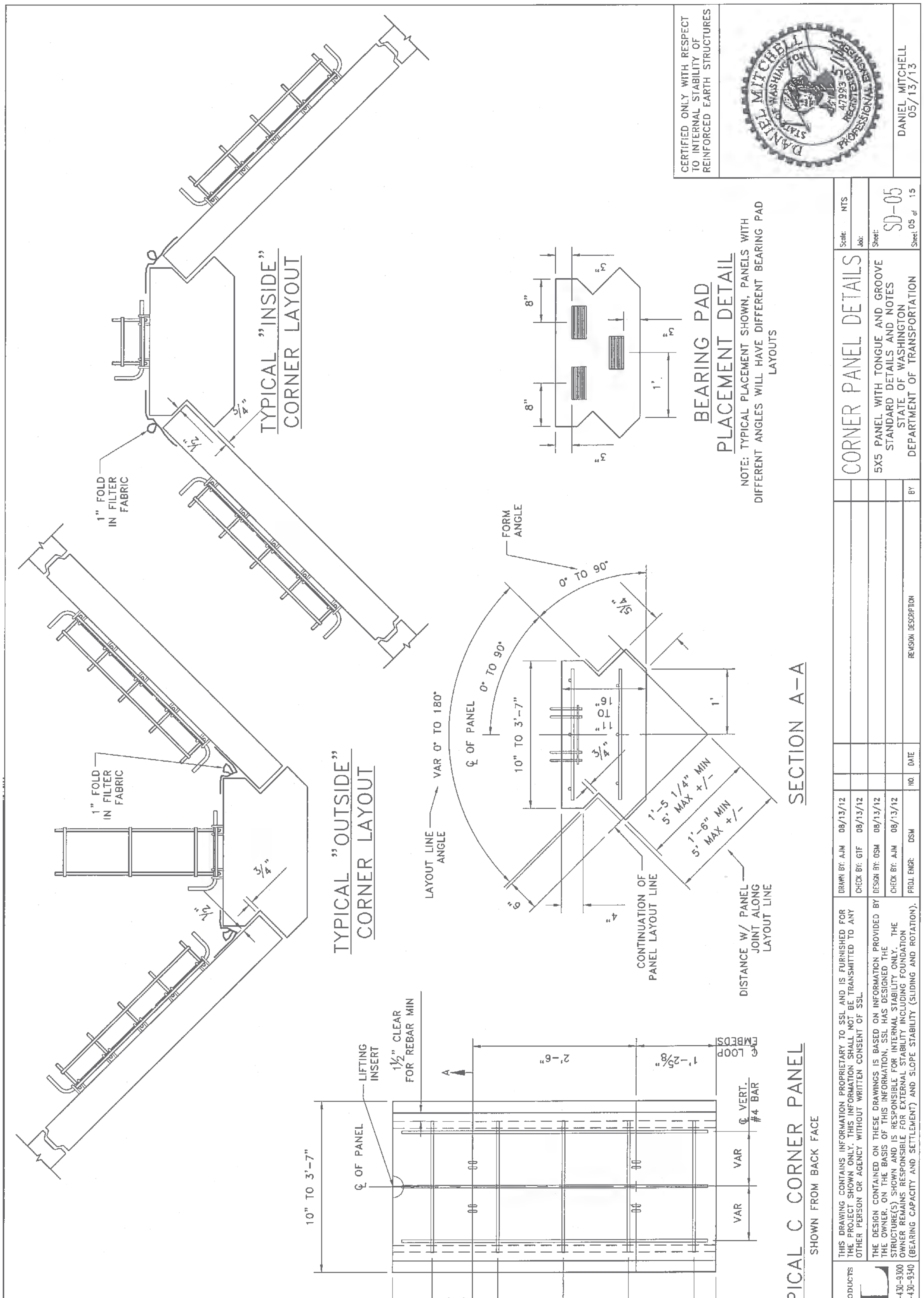
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TYPICAL "INSIDE" CORNER LAYOUT
TYPICAL "OUTSIDE" CORNER LAYOUT
1" FOLD IN FILTER FABRIC
BEARING PAD PLACEMENT DETAIL
NOTE: TYPICAL PLACEMENT SHOWN, PANELS WITH DIFFERENT ANGLES WILL HAVE DIFFERENT BEARING PAD LAYOUTS
SECTION A–A
FORM ANGLE
LAYOUT LINE ANGLE
VAR 0° TO 180°
0° TO 90°
℄ OF PANEL
10" TO 3'-7"
CONTINUATION OF PANEL LAYOUT LINE
DISTANCE W/ PANEL JOINT ALONG LAYOUT LINE
1'-5 1/4" MIN 5' MAX +/-
1'-6" MIN 5' MAX +/-
TYPICAL C CORNER PANEL
SHOWN FROM BACK FACE
LIFTING INSERT
1½" CLEAR FOR REBAR MIN
℄ LOOP EMBEDS
℄ VERT. #4 BAR
℄ HORZ. #4 BAR
VAR
4'-11¼"
CORNER PANEL DETAILS
5X5 PANEL WITH TONGUE AND GROOVE STANDARD DETAILS AND NOTES
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION
Scale: NTS
SD-05
Sheet 05 of 15
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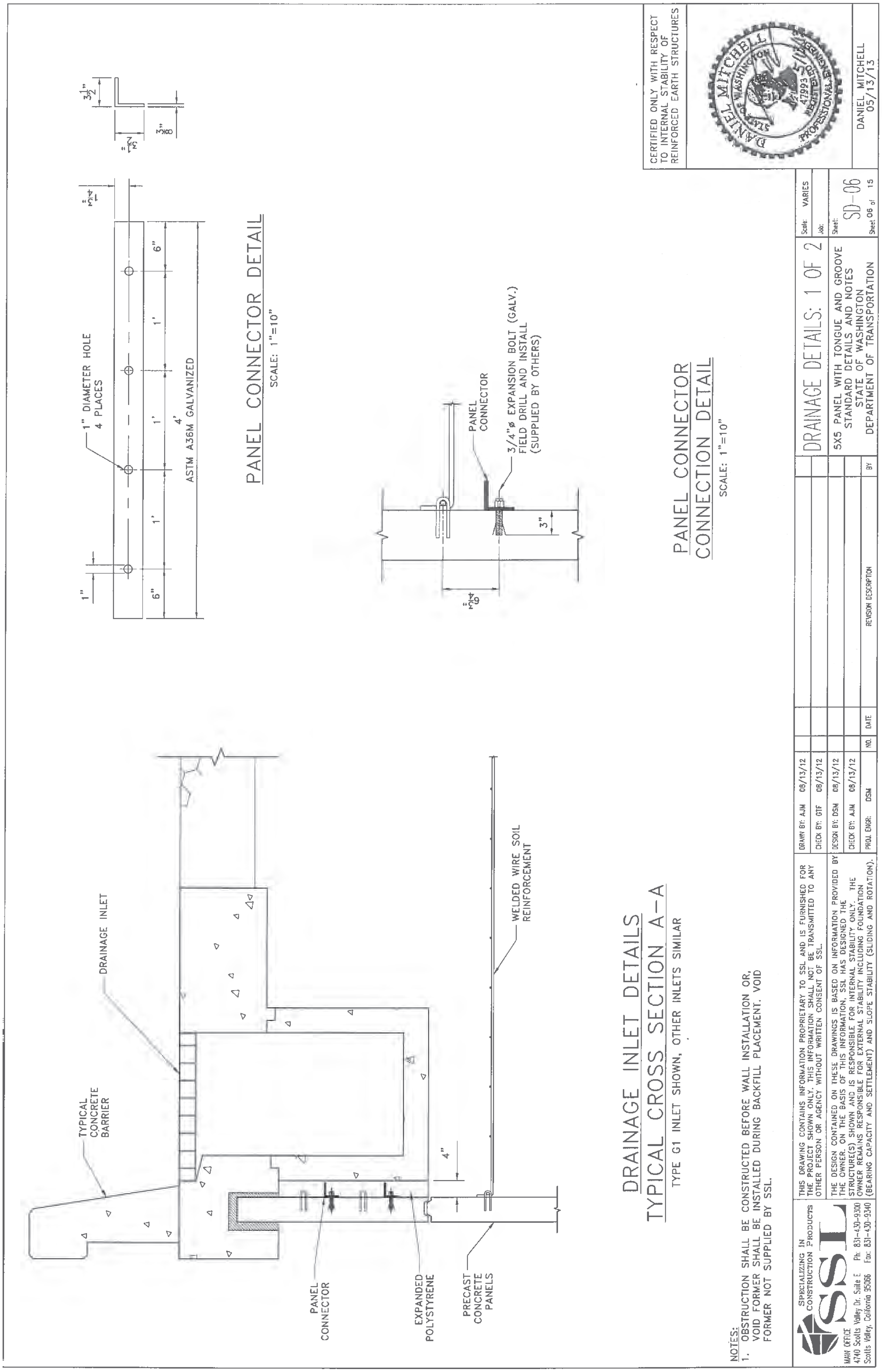
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PANEL CONNECTOR DETAIL
SCALE: 1"=10"
1" DIAMETER HOLE 4 PLACES
ASTM A36M GALVANIZED
PANEL CONNECTOR
3/4"ø EXPANSION BOLT (GALV.) FIELD DRILL AND INSTALL (SUPPLIED BY OTHERS)
PANEL CONNECTOR CONNECTION DETAIL
SCALE: 1"=10"
TYPICAL CONCRETE BARRIER
DRAINAGE INLET
WELDED WIRE SOIL REINFORCEMENT
PANEL CONNECTOR
EXPANDED POLYSTYRENE
PRECAST CONCRETE PANELS
DRAINAGE INLET DETAILS
TYPICAL CROSS SECTION A-A
TYPE G1 INLET SHOWN, OTHER INLETS SIMILAR
NOTES:
1. OBSTRUCTION SHALL BE CONSTRUCTED BEFORE WALL INSTALLATION OR, VOID FORMER SHALL BE INSTALLED DURING BACKFILL PLACEMENT. VOID FORMER NOT SUPPLIED BY SSL.
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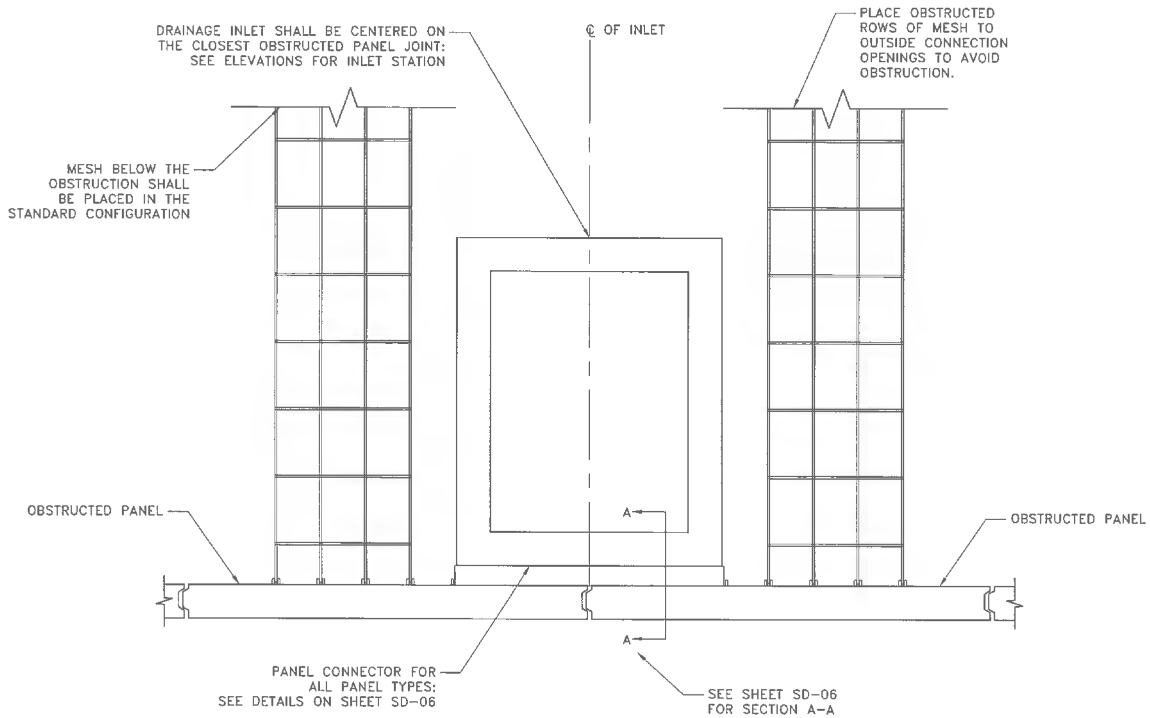
5X5 PANEL WITH TONGUE AND GROOVE STANDARD DETAILS AND NOTES STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION
Scale: VARIES
Sheet: SD-06
Sheet 06 of 15



TYPICAL DRAINAGE DETAILS

(CENTER LINE OF INLET TO BE RELOCATED TO PANEL JOINT)

SCALE: 1"=20"

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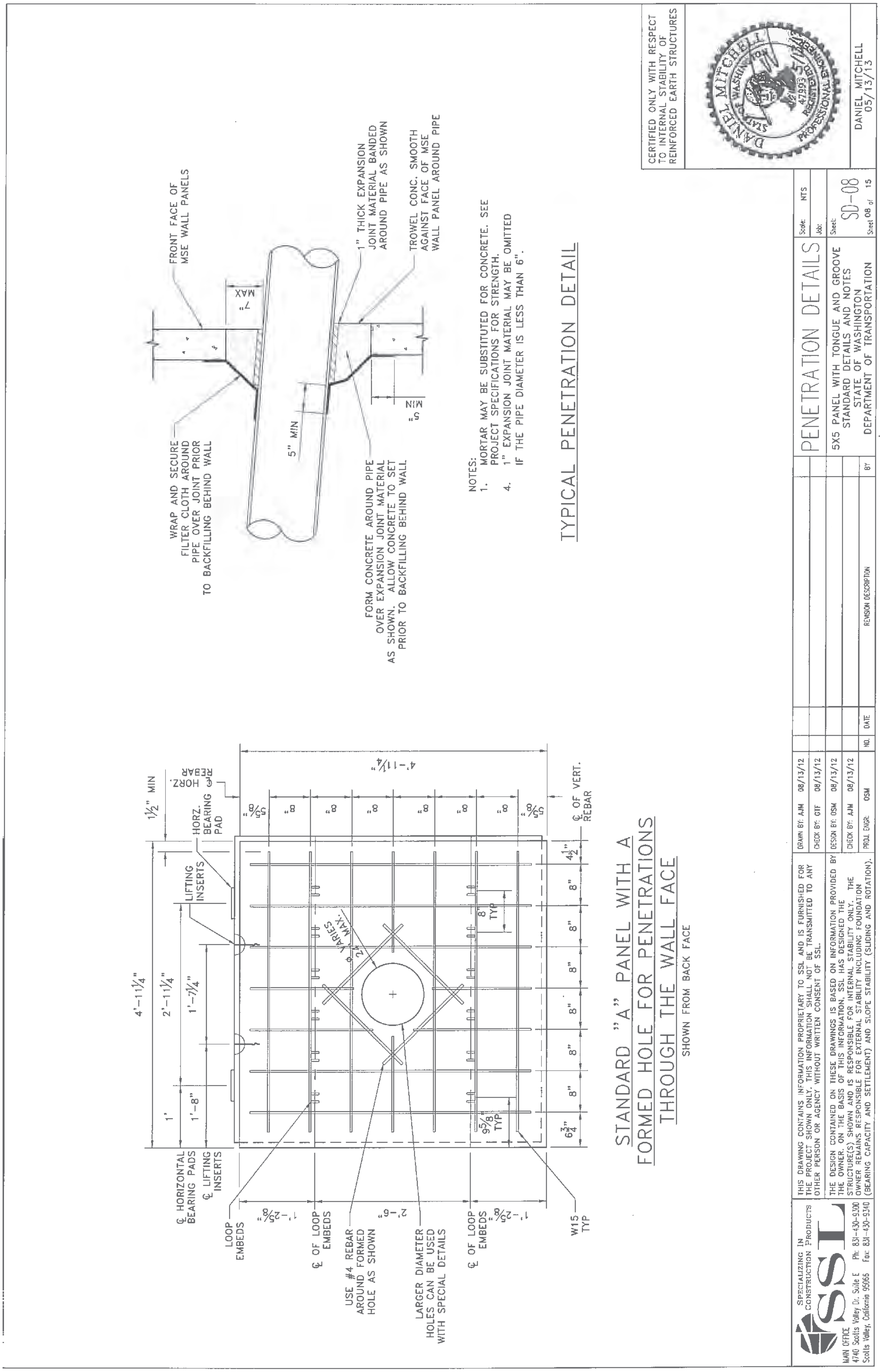
CHECK BY: AJM	08/13/12				
PROJ. ENGR:	DSM	NO.	DATE	REVISION DESCRIPTION	BY

DRAINAGE DETAILS: 2 OF 2

5X5 PANEL WITH TONGUE AND GROOVE STANDARD DETAILS AND NOTES
STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION

Scale: 1"=20"
Job:
Sheet: SD-07
Sheet 07 of 15



STANDARD "A" PANEL WITH A FORMED HOLE FOR PENETRATIONS THROUGH THE WALL FACE
SHOWN FROM BACK FACE
LOOP EMBEDS
℄ HORIZONTAL BEARING PADS
℄ LIFTING INSERTS
LIFTING INSERTS
HORZ. BEARING PAD
℄ HORZ. REBAR
℄ OF VERT. REBAR
℄ OF LOOP EMBEDS
USE #4 REBAR AROUND FORMED HOLE AS SHOWN
LARGER DIAMETER HOLES CAN BE USED WITH SPECIAL DETAILS
W15 TYP
VARIES 24" MAX.
8" TYP
9 5/8" TYP
4'-11 1/4"
2'-11 1/4"
1'-7/4"
1'-8"
1'
1 1/2" MIN
1'-2 5/8"
2'-6"
TYPICAL PENETRATION DETAIL
WRAP AND SECURE FILTER CLOTH AROUND PIPE OVER JOINT PRIOR TO BACKFILLING BEHIND WALL
FRONT FACE OF MSE WALL PANELS
1" THICK EXPANSION JOINT MATERIAL BANDED AROUND PIPE AS SHOWN
TROWEL CONC. SMOOTH AGAINST FACE OF MSE WALL PANEL AROUND PIPE
FORM CONCRETE AROUND PIPE OVER EXPANSION JOINT MATERIAL AS SHOWN. ALLOW CONCRETE TO SET PRIOR TO BACKFILLING BEHIND WALL
5" MIN
7" MAX
NOTES:
1. MORTAR MAY BE SUBSTITUTED FOR CONCRETE. SEE PROJECT SPECIFICATIONS FOR STRENGTH.
4. 1" EXPANSION JOINT MATERIAL MAY BE OMITTED IF THE PIPE DIAMETER IS LESS THAN 6".
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PENETRATION DETAILS
5X5 PANEL WITH TONGUE AND GROOVE STANDARD DETAILS AND NOTES
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION
Scale: NTS
Sheet: SD-08
Sheet 08 of 15
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PROPER STORAGE AND HANDLING OF PANELS

1. THE PANELS SHOULD BE STACKED ONE ON ONE, SEPARATED BY NON-STAINING DUNNAGE, WITH A WIDTH GREATER THAN OR EQUAL TO 2.5 INCHES (OR THE HEIGHT OF THE EMBED, WHICHEVER IS GREATER). THE AMOUNT OF PANELS PER STACK VARIES.
2. DUNNAGE SHOULD BE ALIGNED IN THE VERTICAL DIRECTION. CARE SHOULD BE TAKEN NOT TO DAMAGE THE EDGES OR FACE OF THE PANELS DURING UNLOADING, STORAGE OR SETTING. THE PANELS MAY BE UNLOADED SUPPORTED BY THE PROVIDED PALLETS (SHOWN IN FIGURE 1).
3. DURING PANEL ERECTION, PANELS SHALL BE LIFTED AND SET BY THE USE OF THE TWO LIFTING ANCHORS LOCATED IN THE TOP OF EACH PANEL (SHOWN IN FIGURE 1).
4. WHEN LIFTING PANELS FROM THE STACK, MAKE SURE THAT AN ADDITIONAL PIECE OF DUNNAGE IS BELOW THE BOTTOM EDGE OF THE PANEL TO PREVENT DAMAGE WHEN ROTATING PANELS FROM HORIZONTAL TO VERTICAL (SHOWN IN FIGURES 2).
5. LIFTING LINE MUST BE VERTICAL TO AVOID DAMAGE TO PANEL.

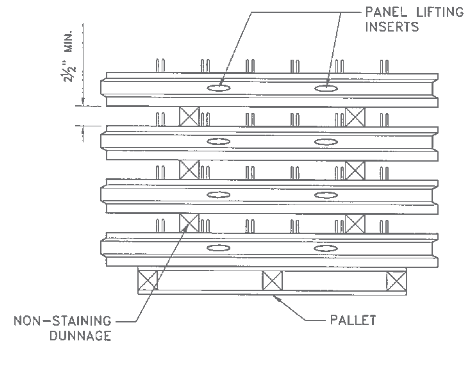

FIGURE 1

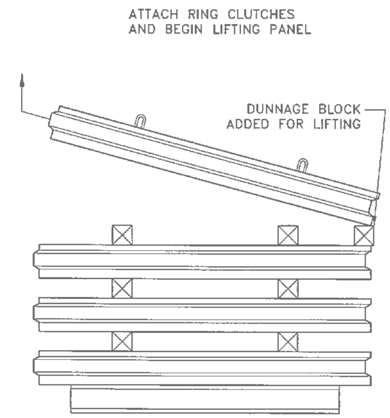

FIGURE 2

PROPER STORAGE AND HANDLING OF WELDED WIRE SOIL REINFORCING

1. SOIL REINFORCEMENT ARRIVES TO THE SITE ON A FLATBED TRUCK WITH DUNNAGE SEPARATING THE DIFFERENT BUNDLES OF SOIL REINFORCING (SEE FIGURE 3).
2. OFF-LOAD THE SOIL REINFORCING CAREFULLY, USING AT LEAST TWO BALANCED PICK POINTS SPACED NO MORE THAN 7 FEET APART (SEE FIGURE 3).
3. PLACE SOIL REINFORCING ON THE DUNNAGE BEFORE SETTING ON THE GROUND, MAKING SURE THAT THE SOIL REINFORCING DOES NOT CONTACT THE GROUND. DO NOT PLACE THE SOIL REINFORCING DIRECTLY ON THE GROUND.
4. ENSURE THAT THE DUNNAGE UNDER THE STACKED BUNDLES OF SOIL REINFORCEMENT ARE ALIGNED VERTICALLY AND ARE NOT SPACED MORE 7 FEET APART HORIZONTALLY (SEE FIGURE 3). NOTE THAT THE PLACEMENT OF THE DUNNAGE IN FIGURE 3 ARE SHOWN FOR CLARIFICATION PURPOSES ONLY. DUNNAGE MAY NEED TO BE ADDED OR REMOVED BASED ON THE LENGTH OF THE SOIL REINFORCEMENT BEING PLACED INTO STORAGE.

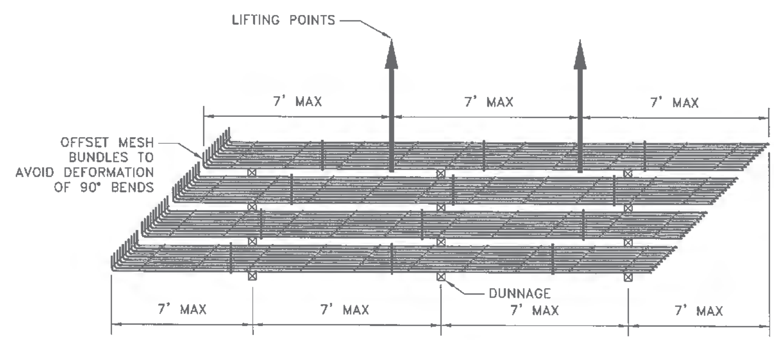

FIGURE 3

CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES

DANIEL MITCHELL 05/13/13

SPECIALIZING IN CONSTRUCTION PRODUCTS
SSL
MAIN OFFICE 4740 Scotts Valley Dr. Suite E Scotts Valley, California 95066 Ph: 831-430-9300 Fax: 831-430-9340

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DESIGN BY: DSM	08/13/12
CHECK BY: AJM	08/13/12
PROJ. ENGR: DSM	

NO.	DATE	REVISION DESCRIPTION	BY

HANDLING AND UNLOADING DETAILS

5X5 PANEL WITH TONGUE AND GROOVE STANDARD DETAILS AND NOTES STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION

Scale: NTS
Job:
Sheet: SD-09
Sheet 09 of 15

6.0 ERECTION SEQUENCE

SITE PREPARATION
THE AREA WHERE THE MSE PLUS™ WALL IS TO BE PLACED SHOULD BE GRADED LEVEL, TO A WIDTH EQUAL TO THE LENGTH OF THE REINFORCING MESH PLUS 6", OR TO CONTRACT SPECIFICATIONS. ANY UNSUITABLE MATERIAL SHOULD BE REMOVED AND REPLACED WITH SELECT BACKFILL AND COMPACTED ACCORDING TO THE CONTRACT SPECIFICATIONS.

CONSTRUCTION OF THE CONCRETE LEVELING PAD
CONSTRUCT THE LEVELING PADS ALONG THE LINES AND GRADES SHOWN ON SSL'S WORKING DRAWINGS. CURE THE CONCRETE AT LEAST 12 HOURS PRIOR TO THE SETTING OF THE FACING PANELS.

ERECTION OF THE BASE LINE OF PANELS
IT MUST BE EMPHASIZED THAT CARE SHOULD BE EXERCISED IN THE ALIGNMENT AND LEVELING OF THE FIRST COURSE OF PANELS.

THIS WILL INSURE THE MOST AESTHETICALLY PLEASING RESULT AND ALSO INCREASE PRODUCTION OF THE ERECTION OF SUBSEQUENT COURSES.

A) THE FIRST STEP IN THE ERECTION OF THE BASE LINE IS TO ESTABLISH THE STARTING POINT. IF A WORKING POINT IS SHOWN ON THE DRAWINGS, OR IF THE WALL MATCHES AN EXISTING STRUCTURE, THIS WOULD BE THE STARTING POINT. IF NEITHER OF THESE ARE THE CASE, THEN BEGIN THE ERECTION ON THE FOOTING WITH THE LOWEST ELEVATION.

B) THE SECOND STEP IS TO ESTABLISH A FACE OF WALL LINE. THIS LINE SHOULD BE SNAPPED ON THE CONCRETE LEVELING PAD WITH A CHALK LINE. THE METHOD FOR ESTABLISHING REFERENCE POINTS FOR THIS WILL BE LEFT TO THE DISCRETION OF THE CONTRACTOR.

C) THIRD, PLACE THE BEGINNING FIRST PANEL WITH THE OUTSIDE FACE ON THE WALL LINE SHOWN IN THE APPROVED WORKING DRAWINGS. THE PANEL IS THEN LEVELED WITH A 5 FT LEVEL AND BATTERED TO THE FILL SIDE APPROXIMATELY 5/8" IN 5 FEET. THIS BATTER OF 5/8" IS TO COMPENSATE FOR BACKFILL COMPACTION EFFORT AND WILL VARY WITH THE NATURE OF THE BACKFILL MATERIAL. THE ACTUAL AMOUNT OF BATTER WILL BE DETERMINED ONLY BY TRIAL AND ERROR.

D) THE SECOND PANEL IS THEN PLACED ADJACENT TO THE BEGINNING HALF PANEL. THE THIRD PANEL IS THEN SET. CHECK THE VERTICAL JOINT SPACING BETWEEN THE TONGUE AND GROOVE OF THE PANELS AND ADJUST TO BETWEEN 5/8" AND 3/4". THE VERTICAL JOINT SHALL BE MEASURED INSIDE THE TONGUE AND GROOVE WITHIN STRUCTURAL THICKNESS OF THE PANELS. LEVEL AND CHECK THE HEIGHT BETWEEN THE PANELS.

E) ADJUST THE PANELS WITH WEDGES OR SPACERS TO ACHIEVE ±3/8" BATTER FROM POINT OF FULL PANEL TO TOP FACE OF PARTIAL PANEL.

F) CHECK BATTER AND LEVEL OF THESE PANELS BEFORE PROCEEDING. CONTINUE PANEL ERECTION.

G) BRACE ALL OF THE FULL PANELS WITH APPROVED BRACING ON THE OUTSIDE FACE. THE BATTER SHOULD BE CHECKED BEFORE SECURING. CLAMP THE SMALLER PANELS TO THE LARGER PANELS. THE BASE COURSE MUST BE CLAMPED AND BRACED TO MAINTAIN ALIGNMENT AND BATTER. NO BRACING WILL BE REQUIRED ON SUBSEQUENT COURSES. SEE DETAILS IN FIGURE 1.

H) GLUE THE FILTER CLOTH TO THE BACK FACE OF THE PANELS OVER THE JOINTS. USING NO MORE THAN A 1/4" ADHESIVE BEAD TO HOLD THE FILTER FABRIC TO THE PANEL WHILE BACKFILL IS BEING PLACED. (FIG. 2)

I) PLACE APPROVED BACKFILL TO THE LEVEL OF THE FIRST ROW OF LOOP CONNECTORS. (FIG. 3) DO NOT PLACE THE FIRST LIFT OF BACKFILL DIRECTLY AGAINST THE PANELS. COMPACT THE ZONE WITHIN 914MM OF THE BACK OF THE WALL FACING PANEL WITHOUT CAUSING DAMAGE TO OR DISTORTION TO THE WALL FACING PANELS BY USING LIGHT MECHANICAL TAMPERS APPROVED BY THE ENGINEER.

J) ATTACH MESH TO PANELS WITH THE PIN CONNECTORS PROVIDED. ALIGN THE LOOP ON THE MESH INTO THE EMBEDS IN THE PANEL. INSERT THE PIN CONNECTOR ROD THROUGH THE PIN WINDOW AND INTO LOOPS THE 3 EMBEDS. REPEAT THIS STEP FOR THE OTHER 3 EMBEDS.

K) THE NEXT LIFTS OF BACKFILL ARE THEN PLACED (THE THICKNESS DETERMINED BY SPECIFICATIONS). FILL AND COMPACT UP TO THE TOP OF THE SMALLER PANEL AS SHOWN IN FIGURE 4. OVERLAPPING MESH REQUIRES 3" OF FILL BETWEEN LAYERS. COMPACTION TESTS SHOULD NOT BE TAKEN WITHIN 3 FEET OF THE PANELS.

ERECTION OF SUBSEQUENT COURSES
A) GLUE THE 3/4" PREFORMED BEARING PADS ON TOP OF THE SMALLER PANELS AND THEN PLACE THE SECOND ROW OF PANELS. (FIG. 5)

B) IF CARE WAS TAKEN ON THE BOTTOM COURSE OF PANELS, LITTLE IF ANY LEVELING SHOULD BE REQUIRED. THE BATTER MAY BE SET BY USING WEDGES BETWEEN THE CLAMP AND PANEL OR IN THE HORIZONTAL PANEL JOINT.

C) PLACE FILTER CLOTH ON BACK FACE OF PANELS. (FIG. 5)

D) PLACE THE BACKFILL UP TO THE NEXT LEVEL OF MESH, INSTALL THE MESH AS ABOVE, AND REPEAT THIS SEQUENCE FOR THE REMAINING WALL HEIGHT. FILL MAY BE SLOPED DOWNWARD TOWARD THE PANELS WITHIN 610MM OF THE BACK FACE IF NECESSARY TO PREVENT EXCESSIVE OUTWARD MOVEMENT WHEN COMPACTING THE BACKFILL.

E) INSTALL END AND TOP PANELS AS REQUIRED.

F) CONTINUE THE ABOVE PROCEDURES UNTIL ALL FACING PANELS ARE INSTALLED.

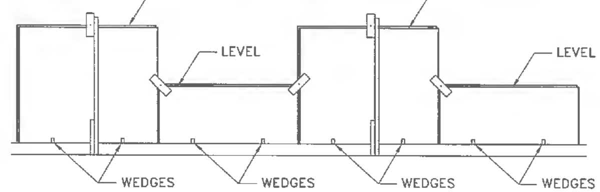
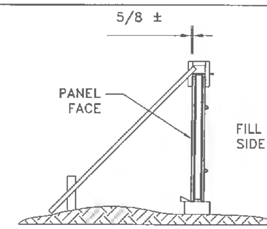
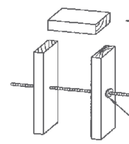


FIGURE 1

FRONT VIEW



SIDE VIEW



CLAMP DETAIL

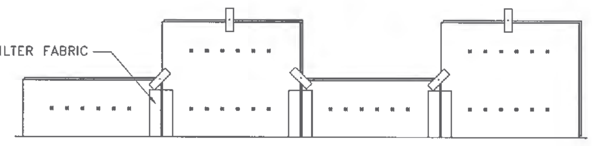


FIGURE 2

BACK VIEW

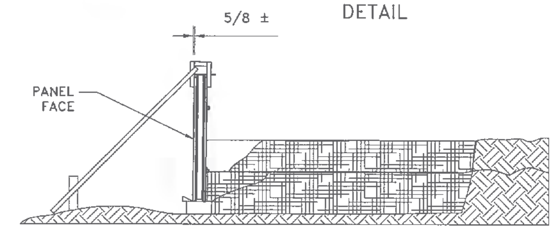


FIGURE 4

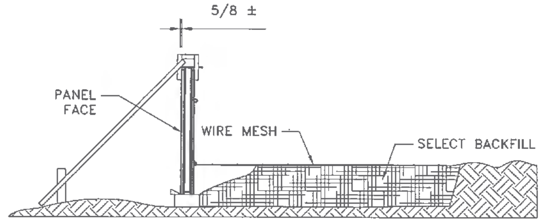


FIGURE 3

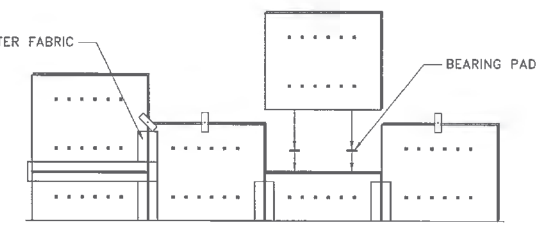


FIGURE 5

BACK VIEW

CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES

DANIEL MITCHELL
STATE OF WASHINGTON
47993
REGISTERED
PROFESSIONAL ENGINEER

DANIEL MITCHELL
05/13/13

SSL, LLC. IS SUPPLYING PRECAST CONCRETE PANELS AND ACCESSORIES TO BE USED IN CONJUNCTION WITH OTHER MATERIALS IN THE CONSTRUCTION OF MSE PLUS RETAINING WALLS DETAILED TO CONFORM TO THE REQUIREMENTS OF THE OWNER'S CONTRACT DOCUMENTS. THE MSE PLUS WALL ERECTION MANUAL FURNISHED TO THE OWNER AND THE WALL CONTRACTOR PROVIDES ONLY A GENERAL EXPLANATION OF THE SYSTEM. THE CONTRACTOR IS SOLELY RESPONSIBLE TO DEVISE AND CARRY OUT A PLAN FOR ERECTION AND ASSEMBLY OF COMPONENTS, WHICH IS PROJECT SPECIFIC. SAID PLAN SHALL PROVIDE FOR UNLOADING, HANDLING, PLACING, AND BRACING COMPONENT MATERIALS WITHOUT RELYING ON THE CONCEPTUAL DRAWINGS AND GUIDELINES PROVIDED BY SSL. ACKNOWLEDGMENT OF THE RECEIPT AND REVIEW OF THE SSL WALL ERECTION MANUAL DOES NOT RELIEVE THE WALL CONTRACTOR OF RESPONSIBILITY TO COMPLY WITH THE PROJECT SPECIFICATIONS AND DRAWINGS OR STRICT COMPLIANCE WITH ALL FALL PROTECTION, LAWS, SAFETY STANDARDS, AND PROCEDURES DURING THE PERFORMANCE OF JOB SITE WORK. PRECAUTIONS SHALL BE TAKEN TO PROTECT INSPECTORS, SUPERVISION AND TRADESMAN FROM INJURY FROM SHIFTING OR FALLING PANELS DURING ERECTION.

SPECIALIZING IN CONSTRUCTION PRODUCTS
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Scotts Valley, California 95066 Fax: 831-430-9340

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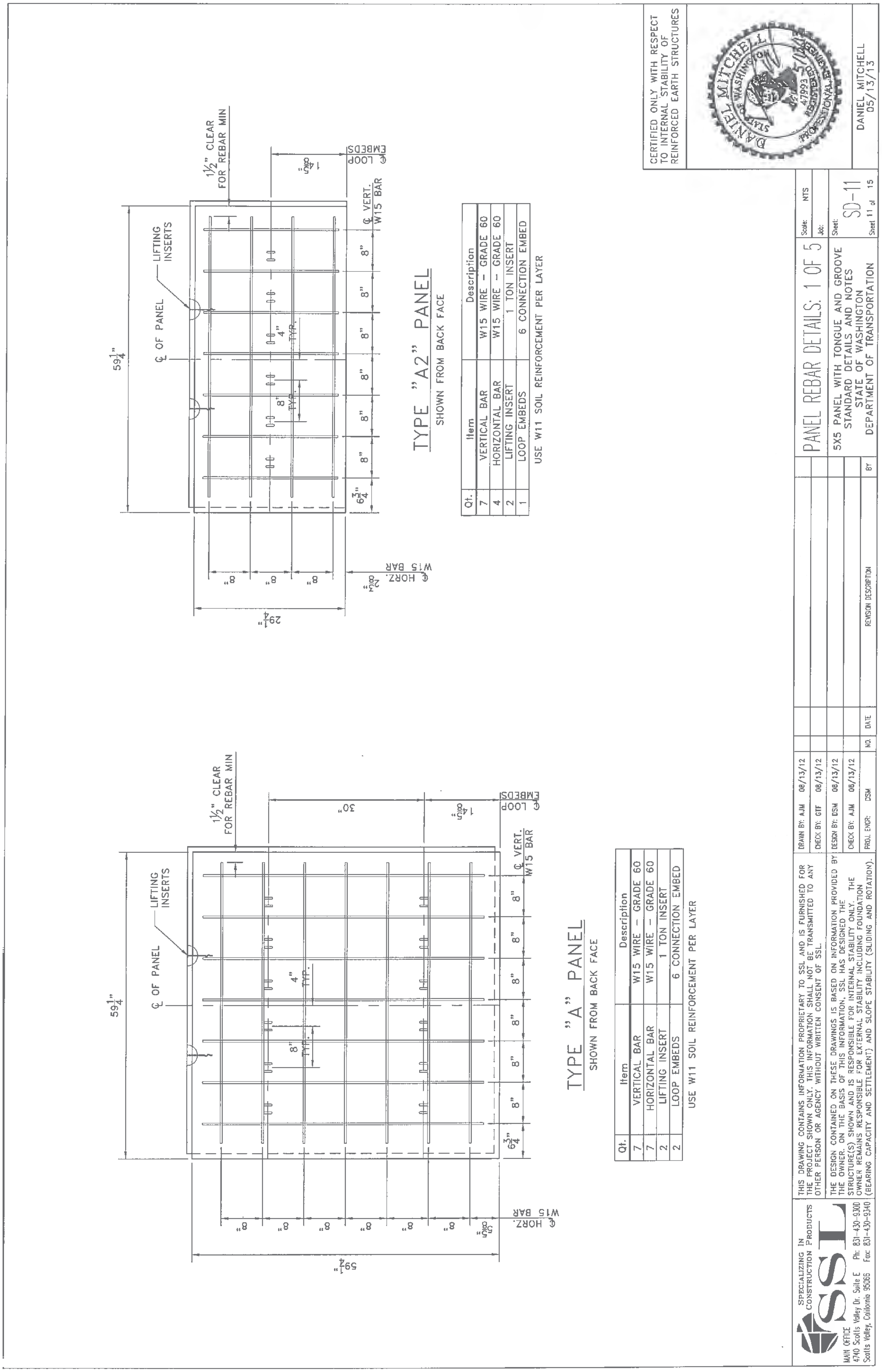
DESIGN BY: DSM 08/13/12				
CHECK BY: AJM 08/13/12				
PROJ. ENGR: DSM	NO.	DATE	REVISION DESCRIPTION	BY

ERECTION SEQUENCE

5X5 PANEL WITH TONGUE AND GROOVE
STANDARD DETAILS AND NOTES
STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION

Scale: NTS
Job:
Sheet: SD-10
Sheet 10 of 15



TYPE "A2" PANEL
SHOWN FROM BACK FACE
Qt. | Item | Description
7 | VERTICAL BAR | W15 WIRE – GRADE 60
4 | HORIZONTAL BAR | W15 WIRE – GRADE 60
2 | LIFTING INSERT | 1 TON INSERT
1 | LOOP EMBEDS | 6 CONNECTION EMBED
USE W11 SOIL REINFORCEMENT PER LAYER
TYPE "A" PANEL
SHOWN FROM BACK FACE
Qt. | Item | Description
7 | VERTICAL BAR | W15 WIRE – GRADE 60
7 | HORIZONTAL BAR | W15 WIRE – GRADE 60
2 | LIFTING INSERT | 1 TON INSERT
2 | LOOP EMBEDS | 6 CONNECTION EMBED
USE W11 SOIL REINFORCEMENT PER LAYER
LIFTING INSERTS
℄ OF PANEL
1½" CLEAR FOR REBAR MIN
℄ LOOP EMBEDS
℄ VERT. W15 BAR
℄ HORZ. W15 BAR
59¼"
29¼"
30"
CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES
DANIEL MITCHELL 05/13/13
PANEL REBAR DETAILS: 1 OF 5
5X5 PANEL WITH TONGUE AND GROOVE STANDARD DETAILS AND NOTES
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION
Scale: NTS
Sheet: SD-11
Sheet 11 of 15
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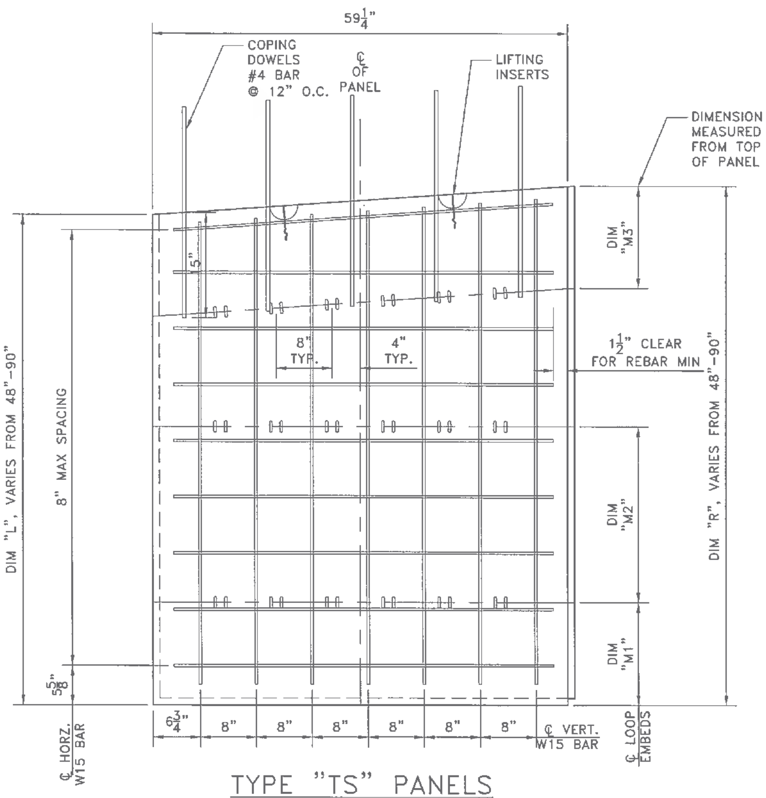
PROJ. ENGR: DSM



TYPE "TS" PANELS WITH 2-3 EMBEDS

SHOWN FROM BACK FACE

Qt.	Item	Description
7	VERTICAL BAR	W15 WIRE – GRADE 60
VAR	HORIZONTAL BAR	W15 WIRE – GRADE 60
2	LIFTING INSERT	1 TON INSERT
VAR	LOOP EMBEDS	6 CONNECTION EMBED

USE W11 SOIL REINFORCEMENT PER LAYER

NOTE:

FOR REBAR PANEL REINFORCEMENT:
TOP PANELS ABOVE 88" NEED 12 HORIZONTAL W15 BARS
TOP PANELS 81" TO 88" NEED 11 HORIZONTAL W15 BARS
TOP PANELS 73" TO 80" NEED 10 HORIZONTAL W15 BARS
TOP PANELS 65" TO 72" NEED 9 HORIZONTAL W15 BARS
TOP PANELS 57" TO 64" NEED 8 HORIZONTAL W15 BARS
TOP PANELS 49" TO 56" NEED 7 HORIZONTAL W15 BARS
TOP PANELS 41" TO 48" NEED 6 HORIZONTAL W15 BARS
TOP PANELS 33" TO 40" NEED 5 HORIZONTAL W15 BARS
TOP PANELS 24" TO 32" NEED 4 HORIZONTAL W15 BARS

DOWELS SHALL #4 BARS BE PLACED 12" MAX O.C. WITH 15" MIN IN THE PANEL AND 15" MIN OUT OF THE PANEL AS NEEDED. IF DOWELS ARE NEEDED THERE WILL BE A "YES" IN THE DOWEL COLUMN, IF DOWELS ARE NOT NEEDED THERE WILL BE A "NO" IN THE COLUMN

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DANIEL MITCHELL
05/13/13

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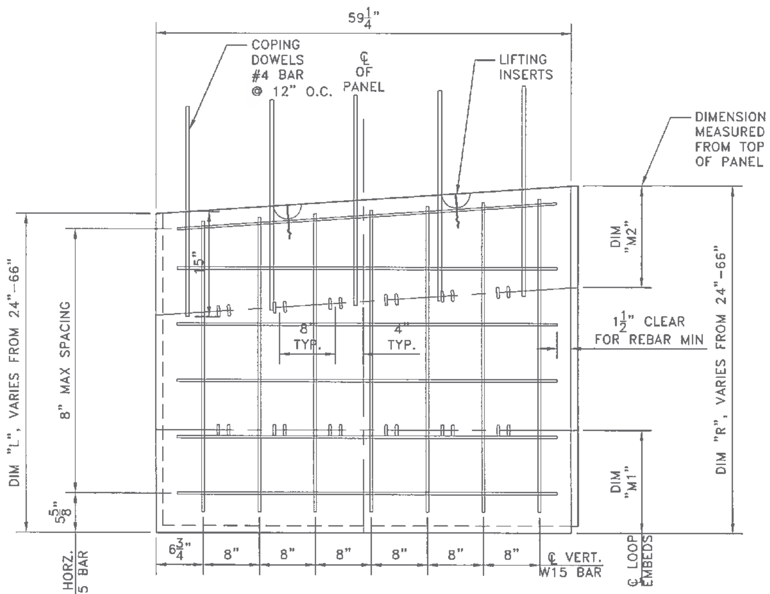
PANEL REBAR DETAILS: 2 OF 5

5X5 PANEL WITH TONGUE AND GROOVE
STANDARD DETAILS AND NOTES
STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION

Scale: NTS
Job:
Sheet: SD-12
Sheet 12 of 15



TYPE "TS" PANELS
WITH 2-3 EMBEDS
SHOWN FROM BACK FACE

Qt.	Item	Description
7	VERTICAL BAR	W15 WIRE – GRADE 60
VAR	HORIZONTAL BAR	W15 WIRE – GRADE 60
2	LIFTING INSERT	1 TON INSERT
VAR	LOOP EMBEDS	6 CONNECTION EMBED

USE W11 SOIL REINFORCEMENT PER LAYER

NOTE:

FOR REBAR PANEL REINFORCEMENT:
TOP PANELS ABOVE 88" NEED 12 HORIZONTAL W15 BARS
TOP PANELS 81" TO 88" NEED 11 HORIZONTAL W15 BARS
TOP PANELS 73" TO 80" NEED 10 HORIZONTAL W15 BARS
TOP PANELS 65" TO 72" NEED 9 HORIZONTAL W15 BARS
TOP PANELS 57" TO 64" NEED 8 HORIZONTAL W15 BARS
TOP PANELS 49" TO 56" NEED 7 HORIZONTAL W15 BARS
TOP PANELS 41" TO 48" NEED 6 HORIZONTAL W15 BARS
TOP PANELS 33" TO 40" NEED 5 HORIZONTAL W15 BARS
TOP PANELS 24" TO 32" NEED 4 HORIZONTAL W15 BARS

DOWELS SHALL #4 BARS BE PLACED 12" MAX O.C. WITH 15" MIN IN THE PANEL AND 15" MIN OUT OF THE PANEL AS NEEDED. IF DOWELS ARE NEEDED THERE WILL BE A "YES" IN THE DOWEL COLUMN, IF DOWELS ARE NOT NEEDED THERE WILL BE A "NO" IN THE COLUMN

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05/13/13

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CHECK BY: AJM	08/13/12				
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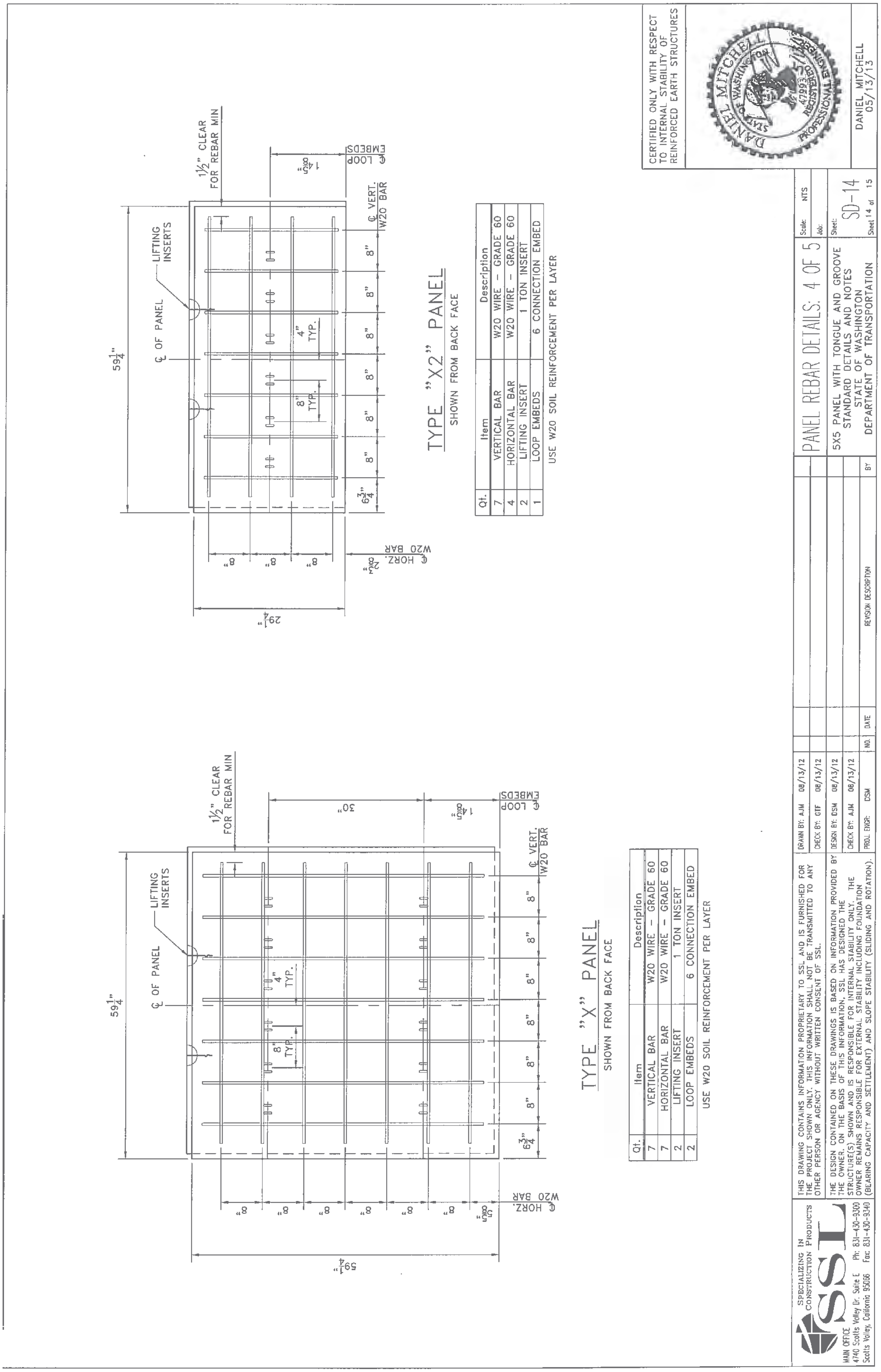
PANEL REBAR DETAILS: 3 OF 5

5X5 PANEL WITH TONGUE AND GROOVE
STANDARD DETAILS AND NOTES
STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION

Scale: NTS
Job:
Sheet: SD-13
Sheet 13 of 15


CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES
DANIEL MITCHELL
05/13/13
TYPE "X2" PANEL
SHOWN FROM BACK FACE
Qt. | Item | Description
7 | VERTICAL BAR | W20 WIRE - GRADE 60
4 | HORIZONTAL BAR | W20 WIRE - GRADE 60
2 | LIFTING INSERT | 1 TON INSERT
1 | LOOP EMBEDS | 6 CONNECTION EMBED
USE W20 SOIL REINFORCEMENT PER LAYER
TYPE "X" PANEL
SHOWN FROM BACK FACE
Qt. | Item | Description
7 | VERTICAL BAR | W20 WIRE - GRADE 60
7 | HORIZONTAL BAR | W20 WIRE - GRADE 60
2 | LIFTING INSERT | 1 TON INSERT
2 | LOOP EMBEDS | 6 CONNECTION EMBED
USE W20 SOIL REINFORCEMENT PER LAYER
1½" CLEAR FOR REBAR MIN
LIFTING INSERTS
℄ OF PANEL
℄ LOOP EMBEDS
℄ VERT. W20 BAR
℄ HORZ. W20 BAR
59¼"
29¼"
30"
8" TYP.
4" TYP.
6¾"
PANEL REBAR DETAILS: 4 OF 5
5X5 PANEL WITH TONGUE AND GROOVE STANDARD DETAILS AND NOTES
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION
Scale: NTS
Sheet: SD-14
Sheet 14 of 15
DRAWN BY: AJM 08/13/12
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DESIGN BY: DSM 08/13/12
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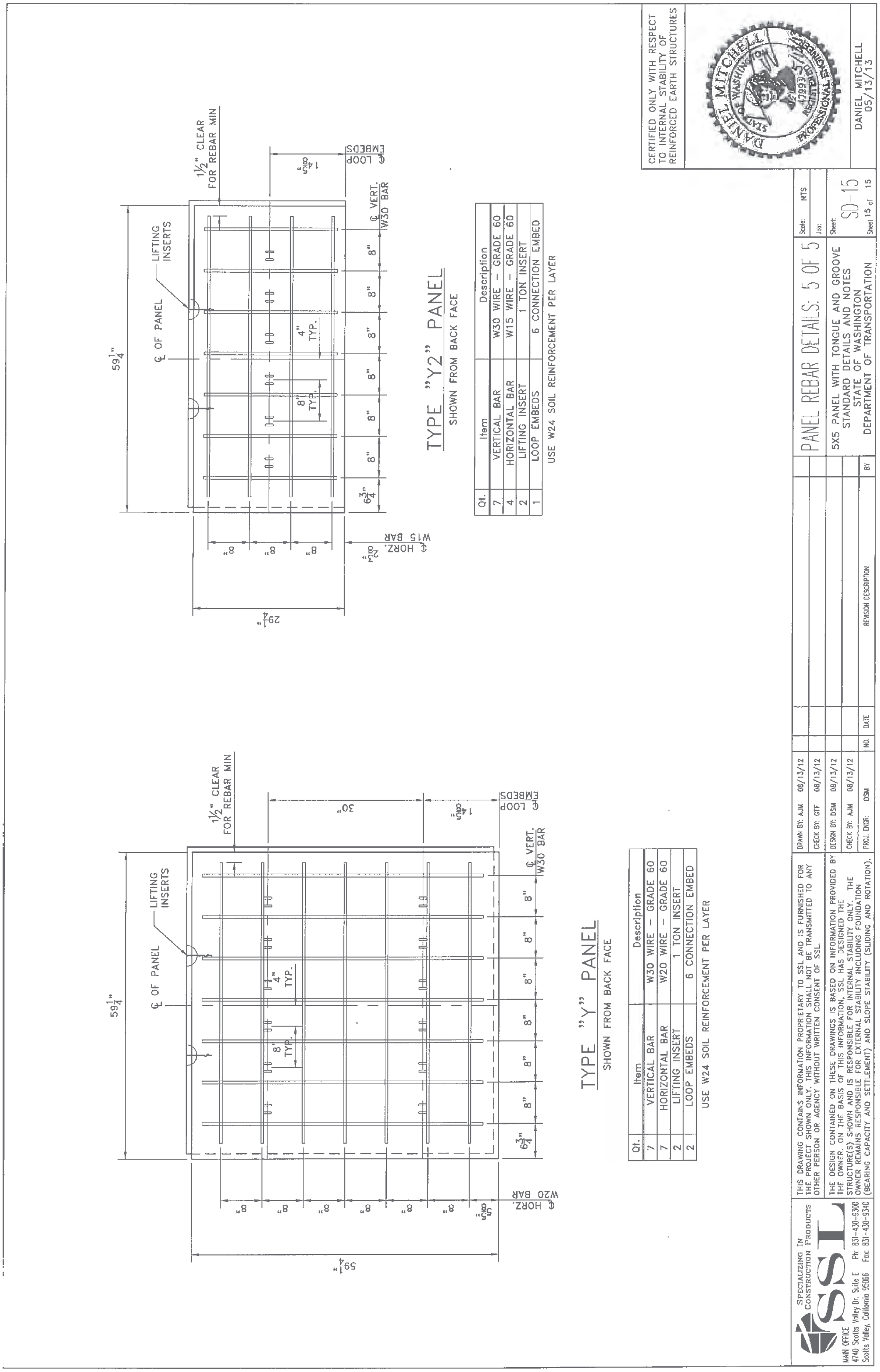

TYPE "Y" PANEL

Qt.	Item	Description
7	VERTICAL BAR	W30 WIRE – GRADE 60
7	HORIZONTAL BAR	W20 WIRE – GRADE 60
2	LIFTING INSERT	1 TON INSERT
2	LOOP EMBEDS	6 CONNECTION EMBED

TYPE "Y2" PANEL

Qt.	Item	Description
7	VERTICAL BAR	W30 WIRE – GRADE 60
4	HORIZONTAL BAR	W15 WIRE – GRADE 60
2	LIFTING INSERT	1 TON INSERT
1	LOOP EMBEDS	6 CONNECTION EMBED

SPECIALIZING IN CONSTRUCTION PRODUCTS

SSL

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INDEX OF DRAWINGS

SHEET	DESCRIPTION
DT-01	TITLE & INDEX
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DT-03	CORNER DETAILS
DT-04	CURVED LAYOUT LINE DETAILS
DT-05	TRAFFIC BARRIER SYSTEMS
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DT-09	COPING SAMPLE DETAILS
DT-10	SLIP JOINT DETAILS

CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES

DANIEL MITCHELL STATE OF WASHINGTON 47993 REGISTERED PROFESSIONAL ENGINEER

DANIEL S. MITCHELL
5/13/13

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TITLE & INDEX

STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION
SSL LLC. MSE PLUS
PANEL WALL SYSTEM

Scale: NTS
Job: ----
Sheet: DT-01
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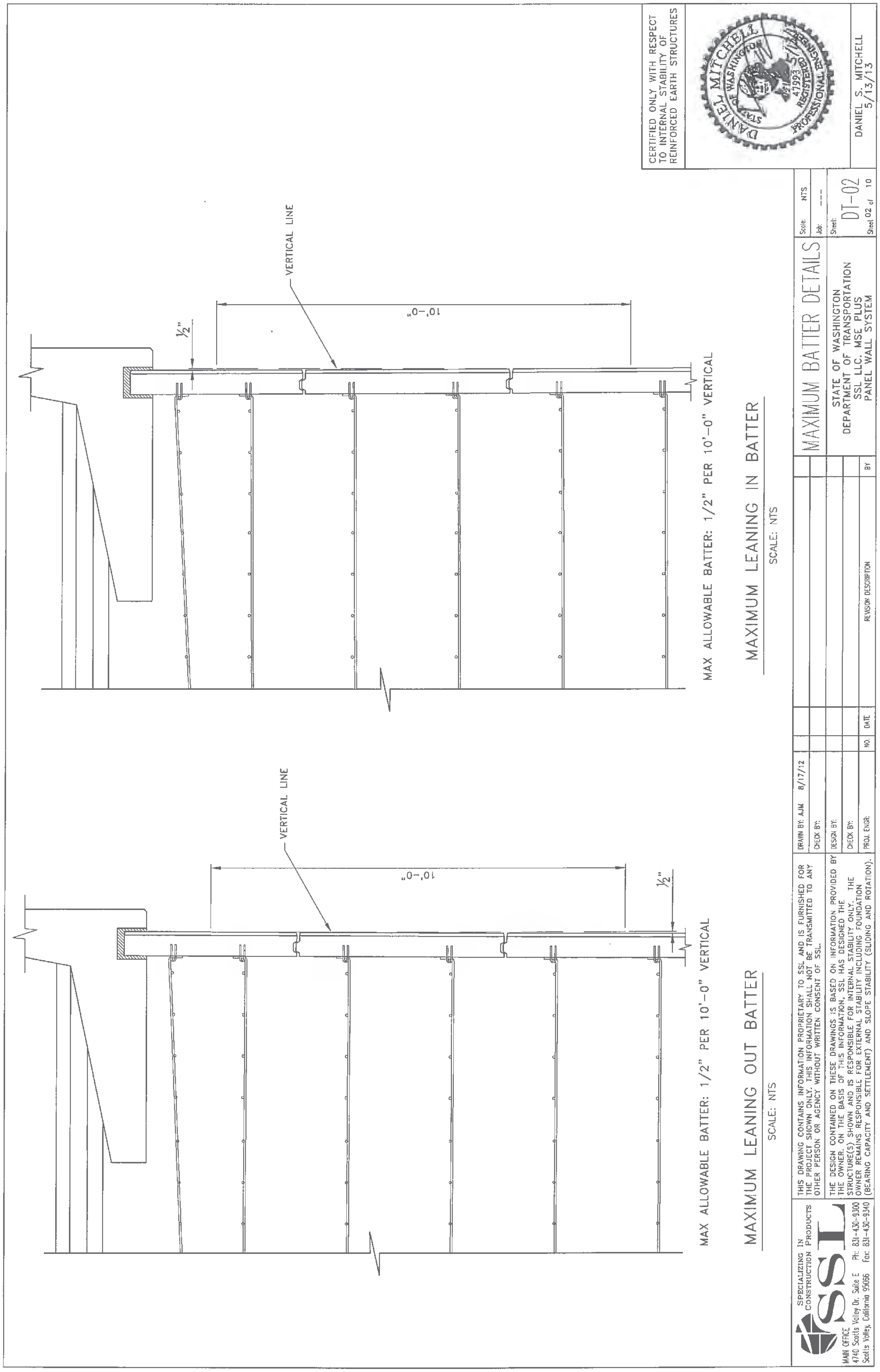
CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES
DANIEL S. MITCHELL 5/13/13
VERTICAL LINE
10'-0"
½"
MAX ALLOWABLE BATTER: 1/2" PER 10'-0" VERTICAL
MAXIMUM LEANING IN BATTER
SCALE: NTS
MAXIMUM LEANING OUT BATTER
SCALE: NTS
MAXIMUM BATTER DETAILS
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION SSL LLC. MSE PLUS PANEL WALL SYSTEM
Scale: NTS
Sheet: DT-02
Sheet 02 of 10
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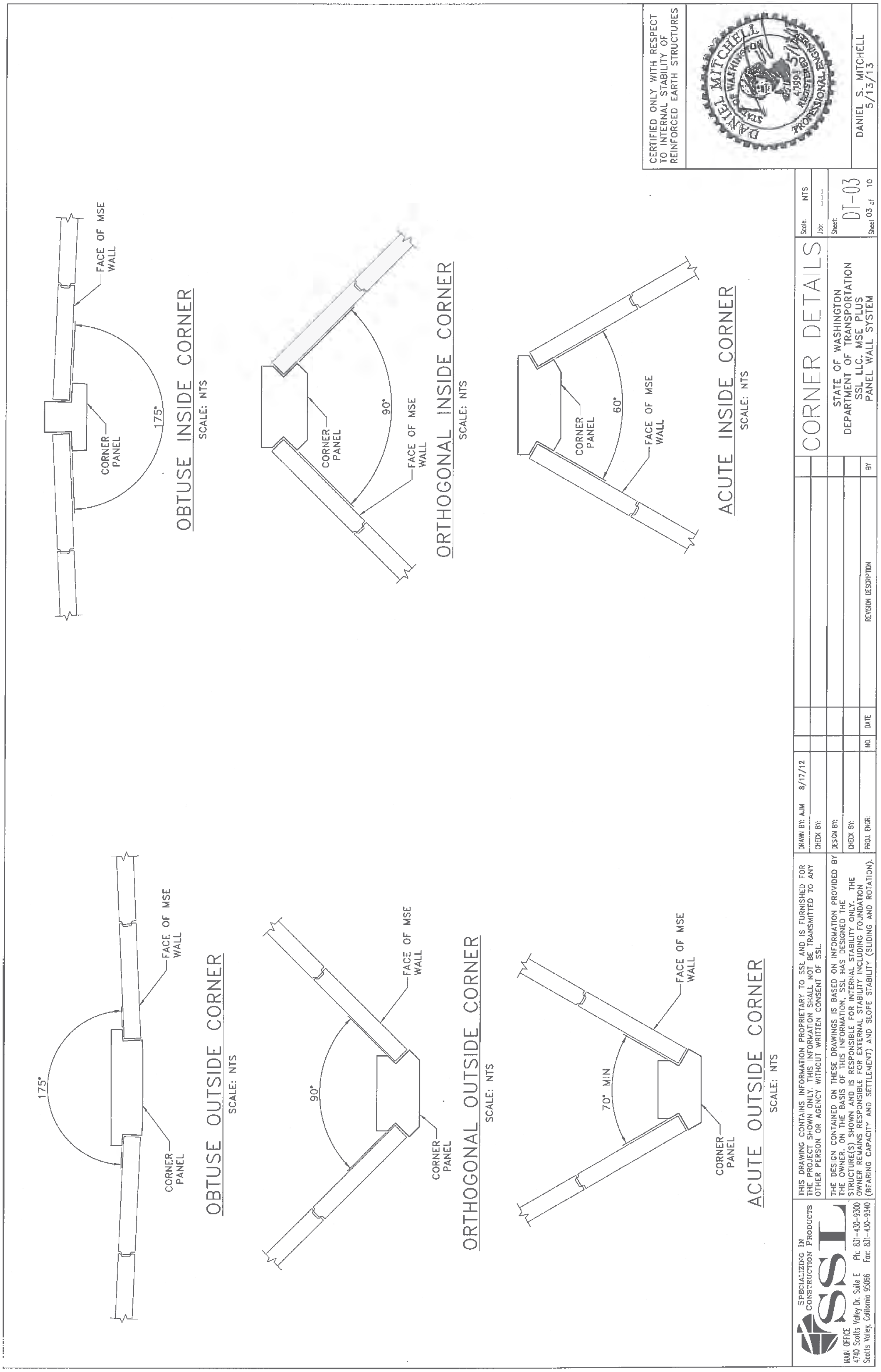
OBTUSE INSIDE CORNER
SCALE: NTS
175°
CORNER PANEL
FACE OF MSE WALL
ORTHOGONAL INSIDE CORNER
SCALE: NTS
90°
CORNER PANEL
FACE OF MSE WALL
ACUTE INSIDE CORNER
SCALE: NTS
60°
CORNER PANEL
FACE OF MSE WALL
OBTUSE OUTSIDE CORNER
SCALE: NTS
175°
CORNER PANEL
FACE OF MSE WALL
ORTHOGONAL OUTSIDE CORNER
SCALE: NTS
90°
CORNER PANEL
FACE OF MSE WALL
ACUTE OUTSIDE CORNER
SCALE: NTS
70° MIN
CORNER PANEL
FACE OF MSE WALL
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SSL, LLC. MSE PLUS
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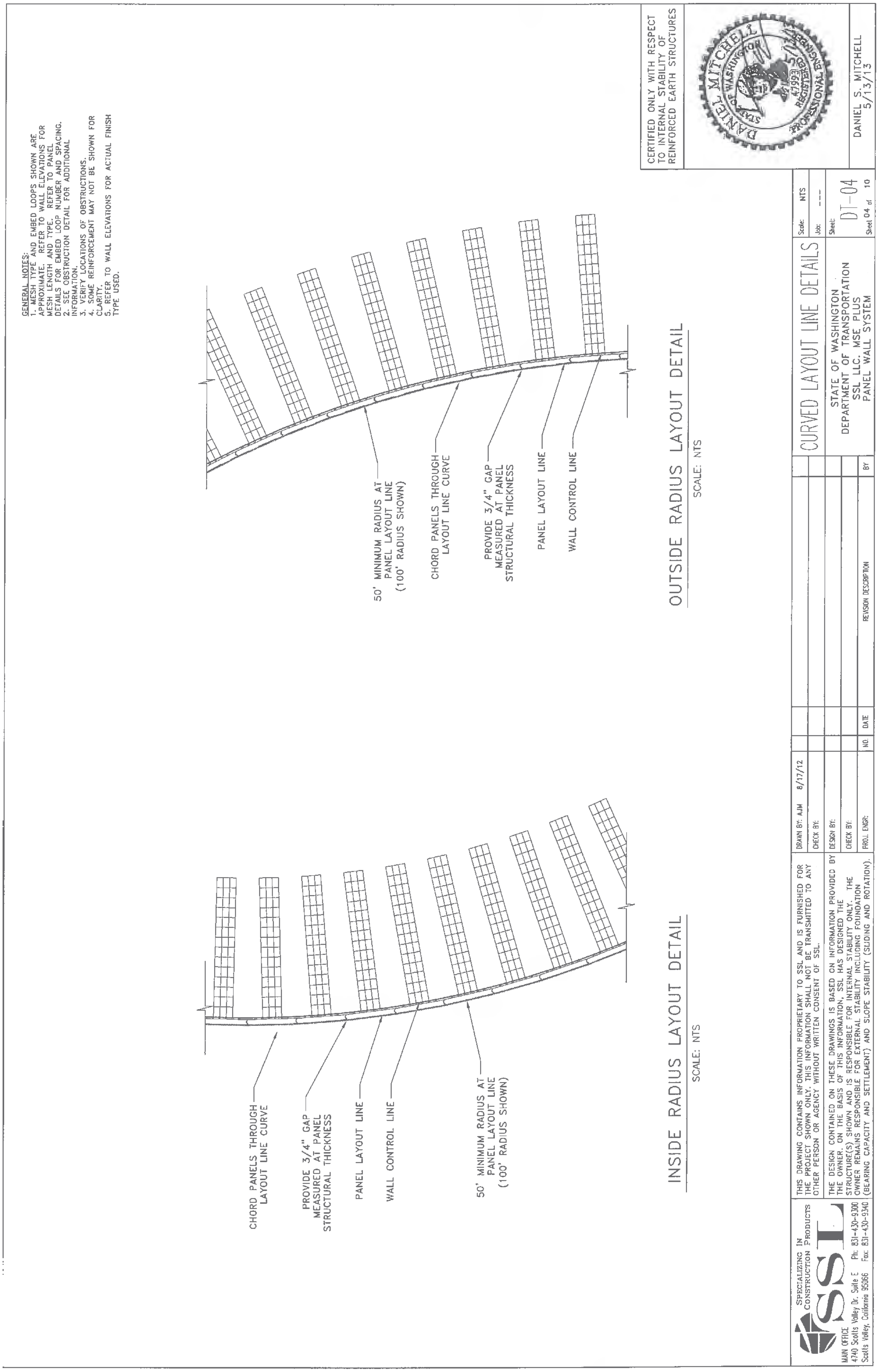
Scale: NTS
Job:
Sheet:
DT-03
Sheet 03 of 10


GENERAL NOTES:
1. MESH TYPE AND EMBED LOOPS SHOWN ARE APPROXIMATE. REFER TO WALL ELEVATIONS FOR MESH LENGTH AND TYPE. REFER TO PANEL DETAILS FOR EMBED LOOP NUMBER AND SPACING.
2. SEE OBSTRUCTION DETAIL FOR ADDITIONAL INFORMATION.
3. VERIFY LOCATIONS OF OBSTRUCTIONS.
4. SOME REINFORCEMENT MAY NOT BE SHOWN FOR CLARITY.
5. REFER TO WALL ELEVATIONS FOR ACTUAL FINISH TYPE USED.
50' MINIMUM RADIUS AT PANEL LAYOUT LINE (100' RADIUS SHOWN)
CHORD PANELS THROUGH LAYOUT LINE CURVE
PROVIDE 3/4" GAP MEASURED AT PANEL STRUCTURAL THICKNESS
PANEL LAYOUT LINE
WALL CONTROL LINE
OUTSIDE RADIUS LAYOUT DETAIL
SCALE: NTS
CHORD PANELS THROUGH LAYOUT LINE CURVE
PROVIDE 3/4" GAP MEASURED AT PANEL STRUCTURAL THICKNESS
PANEL LAYOUT LINE
WALL CONTROL LINE
50' MINIMUM RADIUS AT PANEL LAYOUT LINE (100' RADIUS SHOWN)
INSIDE RADIUS LAYOUT DETAIL
SCALE: NTS
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CURVED LAYOUT LINE DETAILS
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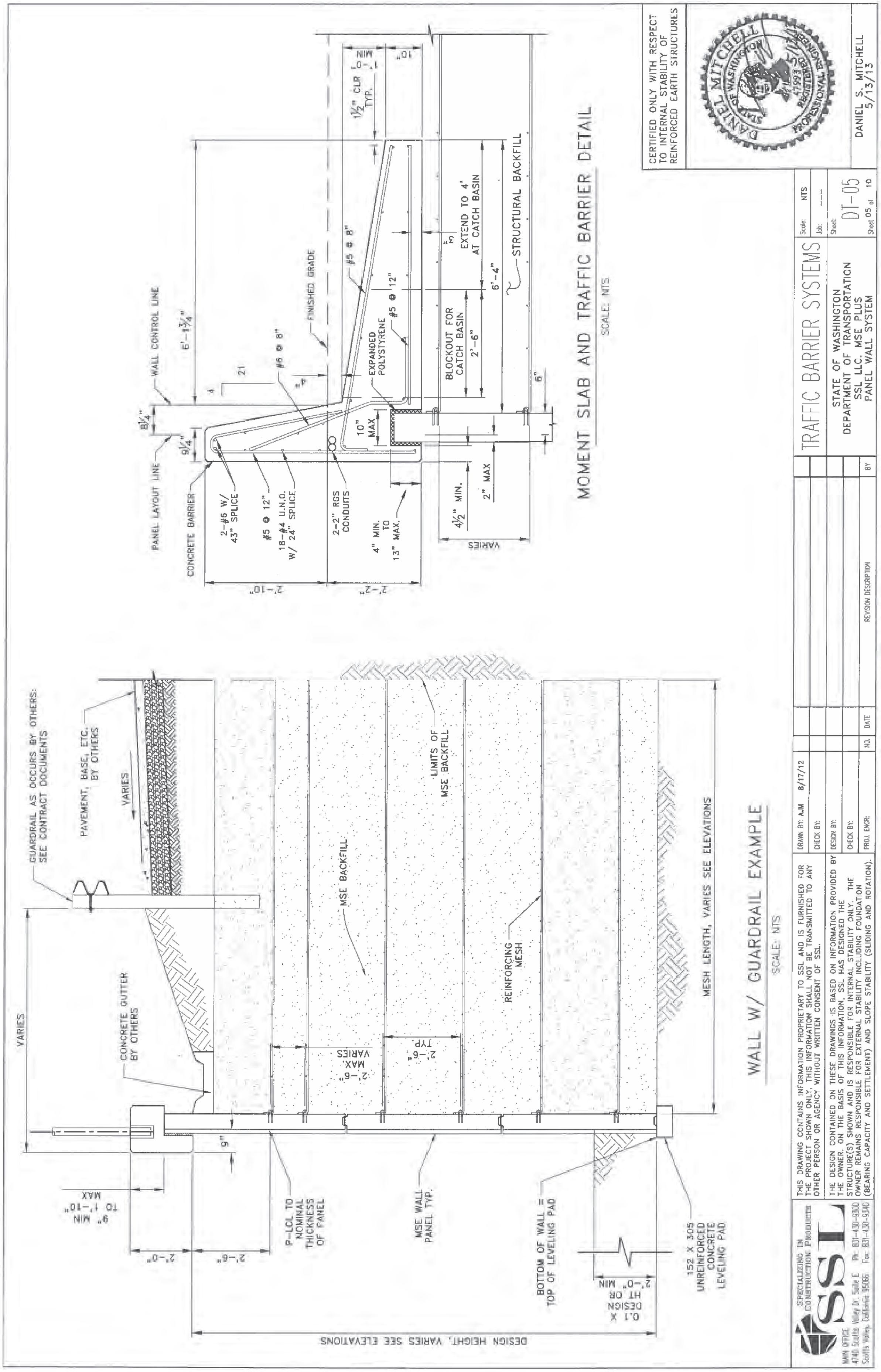
Scale: NTS
Job: ---
Sheet: DT-04
Sheet 04 of 10


MOMENT SLAB AND TRAFFIC BARRIER DETAIL
SCALE: NTS
WALL W/ GUARDRAIL EXAMPLE
SCALE: NTS
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TRAFFIC BARRIER SYSTEMS
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION SSL LLC. MSE PLUS PANEL WALL SYSTEM
DT-05
WALL CONTROL LINE
PANEL LAYOUT LINE
CONCRETE BARRIER
FINISHED GRADE
EXPANDED POLYSTYRENE
BLOCKOUT FOR CATCH BASIN
EXTEND TO 4' AT CATCH BASIN
STRUCTURAL BACKFILL
2-2" RGS CONDUITS
GUARDRAIL AS OCCURS BY OTHERS: SEE CONTRACT DOCUMENTS
PAVEMENT, BASE, ETC. BY OTHERS
CONCRETE GUTTER BY OTHERS
MSE BACKFILL
LIMITS OF MSE BACKFILL
REINFORCING MESH
MESH LENGTH, VARIES SEE ELEVATIONS
P-LOL TO NOMINAL THICKNESS OF PANEL
MSE WALL PANEL TYP.
BOTTOM OF WALL = TOP OF LEVELING PAD
152 X 305 UNREINFORCED CONCRETE LEVELING PAD
DESIGN HEIGHT, VARIES SEE ELEVATIONS
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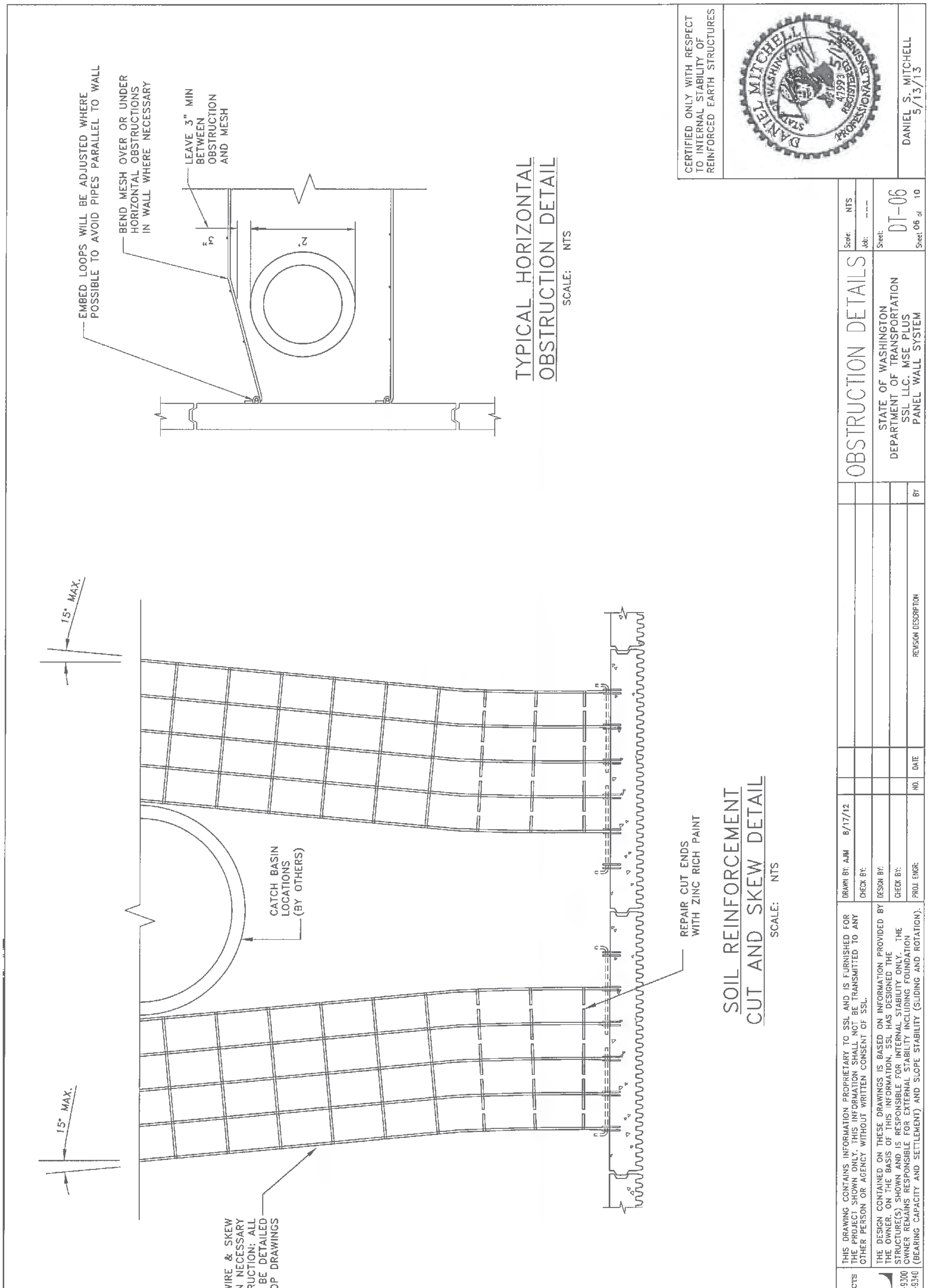

EMBED LOOPS WILL BE ADJUSTED WHERE POSSIBLE TO AVOID PIPES PARALLEL TO WALL
BEND MESH OVER OR UNDER HORIZONTAL OBSTRUCTIONS IN WALL WHERE NECESSARY
LEAVE 3" MIN BETWEEN OBSTRUCTION AND MESH
3"
2'
TYPICAL HORIZONTAL OBSTRUCTION DETAIL
SCALE: NTS
15° MAX.
15° MAX.
CATCH BASIN LOCATIONS (BY OTHERS)
REPAIR CUT ENDS WITH ZINC RICH PAINT
CUT CROSSWIRE & SKEW MESH ONLY WHEN NECESSARY TO AVOID OBSTRUCTION: ALL SKEWED MESH TO BE DETAILED IN SE WALL SHOP DRAWINGS
SOIL REINFORCEMENT CUT AND SKEW DETAIL
SCALE: NTS
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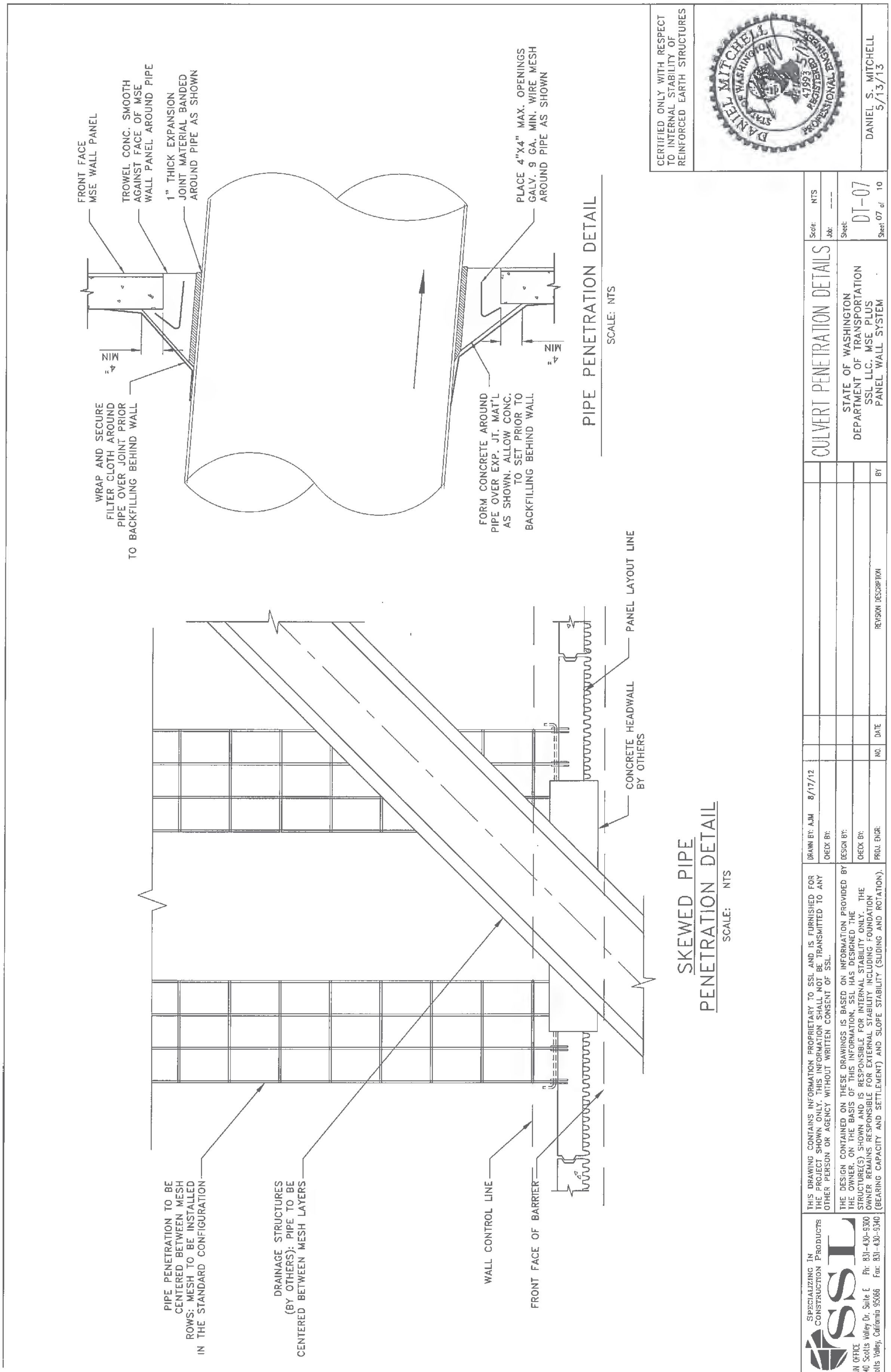
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION
SSL LLC. MSE PLUS PANEL WALL SYSTEM
Scale: NTS
Job: ---
Sheet: DT-06
Sheet 06 of 10


FRONT FACE
MSE WALL PANEL
TROWEL CONC. SMOOTH
AGAINST FACE OF MSE
WALL PANEL AROUND PIPE
1" THICK EXPANSION
JOINT MATERIAL BANDED
AROUND PIPE AS SHOWN
PLACE 4"X4" MAX. OPENINGS
GALV. 9 GA. MIN. WIRE MESH
AROUND PIPE AS SHOWN
WRAP AND SECURE
FILTER CLOTH AROUND
PIPE OVER JOINT PRIOR
TO BACKFILLING BEHIND WALL
4" MIN
FORM CONCRETE AROUND
PIPE OVER EXP. JT. MAT'L
AS SHOWN. ALLOW CONC.
TO SET PRIOR TO
BACKFILLING BEHIND WALL.
PIPE PENETRATION DETAIL
SCALE: NTS
CERTIFIED ONLY WITH RESPECT
TO INTERNAL STABILITY OF
REINFORCED EARTH STRUCTURES
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PANEL LAYOUT LINE
CONCRETE HEADWALL
BY OTHERS
SKEWED PIPE
PENETRATION DETAIL
SCALE: NTS
PIPE PENETRATION TO BE
CENTERED BETWEEN MESH
ROWS; MESH TO BE INSTALLED
IN THE STANDARD CONFIGURATION
DRAINAGE STRUCTURES
(BY OTHERS): PIPE TO BE
CENTERED BETWEEN MESH LAYERS
WALL CONTROL LINE
FRONT FACE OF BARRIER
SPECIALIZING IN
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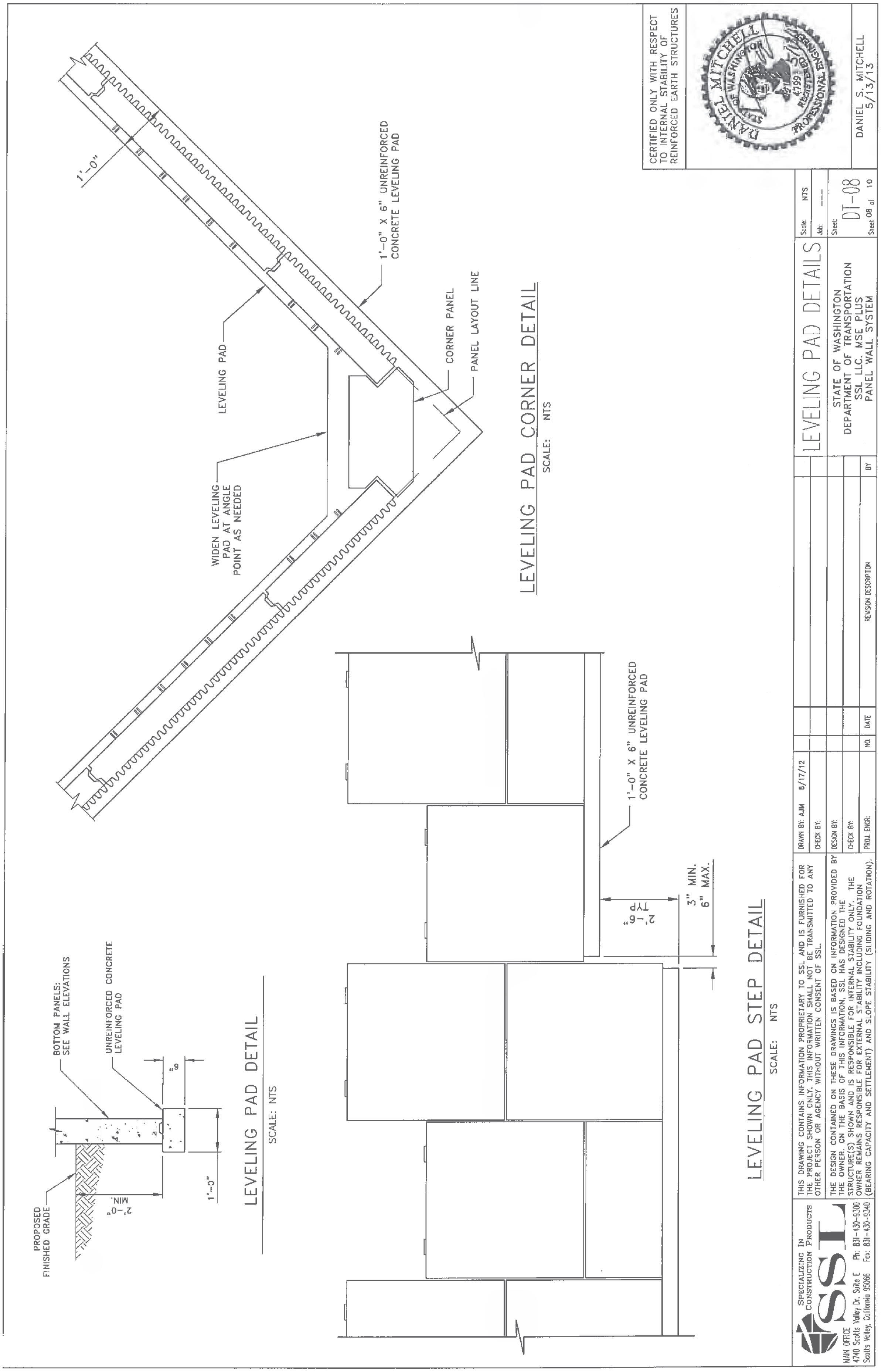
SSL LLC. MSE PLUS
PANEL WALL SYSTEM
Scale: NTS
Job: ---
Sheet:
DT-07
Sheet 07 of 10


LEVELING PAD CORNER DETAIL
SCALE: NTS
1'-0"
1'-0" X 6" UNREINFORCED CONCRETE LEVELING PAD
LEVELING PAD
CORNER PANEL
PANEL LAYOUT LINE
WIDEN LEVELING PAD AT ANGLE POINT AS NEEDED
LEVELING PAD DETAIL
SCALE: NTS
PROPOSED FINISHED GRADE
BOTTOM PANELS: SEE WALL ELEVATIONS
UNREINFORCED CONCRETE LEVELING PAD
6"
2'-0" MIN.
1'-0"
LEVELING PAD STEP DETAIL
SCALE: NTS
1'-0" X 6" UNREINFORCED CONCRETE LEVELING PAD
2'-6" TYP
3" MIN.
6" MAX.
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LEVELING PAD DETAILS
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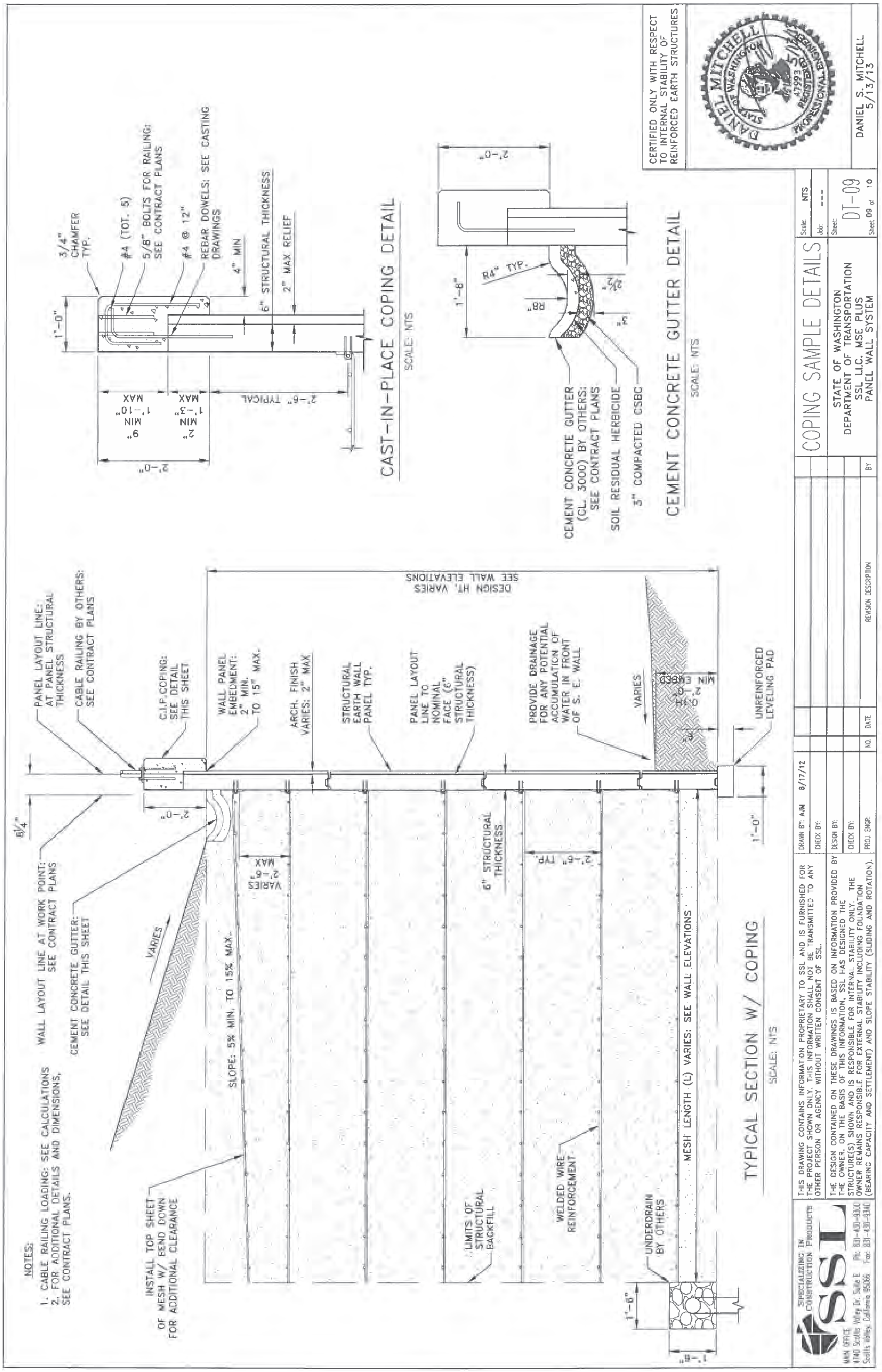
Scale: NTS
Job: ---
Sheet: DT-08
Sheet 08 of 10


NOTES:
1. CABLE RAILING LOADING: SEE CALCULATIONS
2. FOR ADDITIONAL DETAILS AND DIMENSIONS, SEE CONTRACT PLANS.
WALL LAYOUT LINE AT WORK POINT: SEE CONTRACT PLANS
CEMENT CONCRETE GUTTER: SEE DETAIL THIS SHEET
PANEL LAYOUT LINE: AT PANEL STRUCTURAL THICKNESS
CABLE RAILING BY OTHERS: SEE CONTRACT PLANS
C.I.P. COPING: SEE DETAIL THIS SHEET
WALL PANEL EMBEDMENT: 2" MIN. TO 15" MAX.
ARCH. FINISH VARIES: 2" MAX
STRUCTURAL EARTH WALL PANEL TYP.
PANEL LAYOUT LINE TO NOMINAL FACE (6" STRUCTURAL THICKNESS)
PROVIDE DRAINAGE FOR ANY POTENTIAL ACCUMULATION OF WATER IN FRONT OF S. E. WALL
VARIES
UNREINFORCED LEVELING PAD
SEE WALL ELEVATIONS DESIGN HT. VARIES
INSTALL TOP SHEET OF MESH W/ BEND DOWN FOR ADDITIONAL CLEARANCE
SLOPE: 5% MIN. TO 15% MAX.
VARIES 2'-6" MAX
6" STRUCTURAL THICKNESS
2'-6" TYP.
LIMITS OF STRUCTURAL BACKFILL
WELDED WIRE REINFORCEMENT
UNDERDRAIN BY OTHERS
MESH LENGTH (L) VARIES: SEE WALL ELEVATIONS
TYPICAL SECTION W/ COPING
SCALE: NTS
3/4" CHAMFER TYP.
#4 (TOT. 5)
5/8" BOLTS FOR RAILING: SEE CONTRACT PLANS
#4 @ 12"
REBAR DOWELS: SEE CASTING DRAWINGS
4" MIN
6" STRUCTURAL THICKNESS
2" MAX RELIEF
CAST-IN-PLACE COPING DETAIL
SCALE: NTS
CEMENT CONCRETE GUTTER (CL. 3000) BY OTHERS: SEE CONTRACT PLANS
SOIL RESIDUAL HERBICIDE
3" COMPACTED CSBC
R4" TYP.
CEMENT CONCRETE GUTTER DETAIL
SCALE: NTS
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COPING SAMPLE DETAILS
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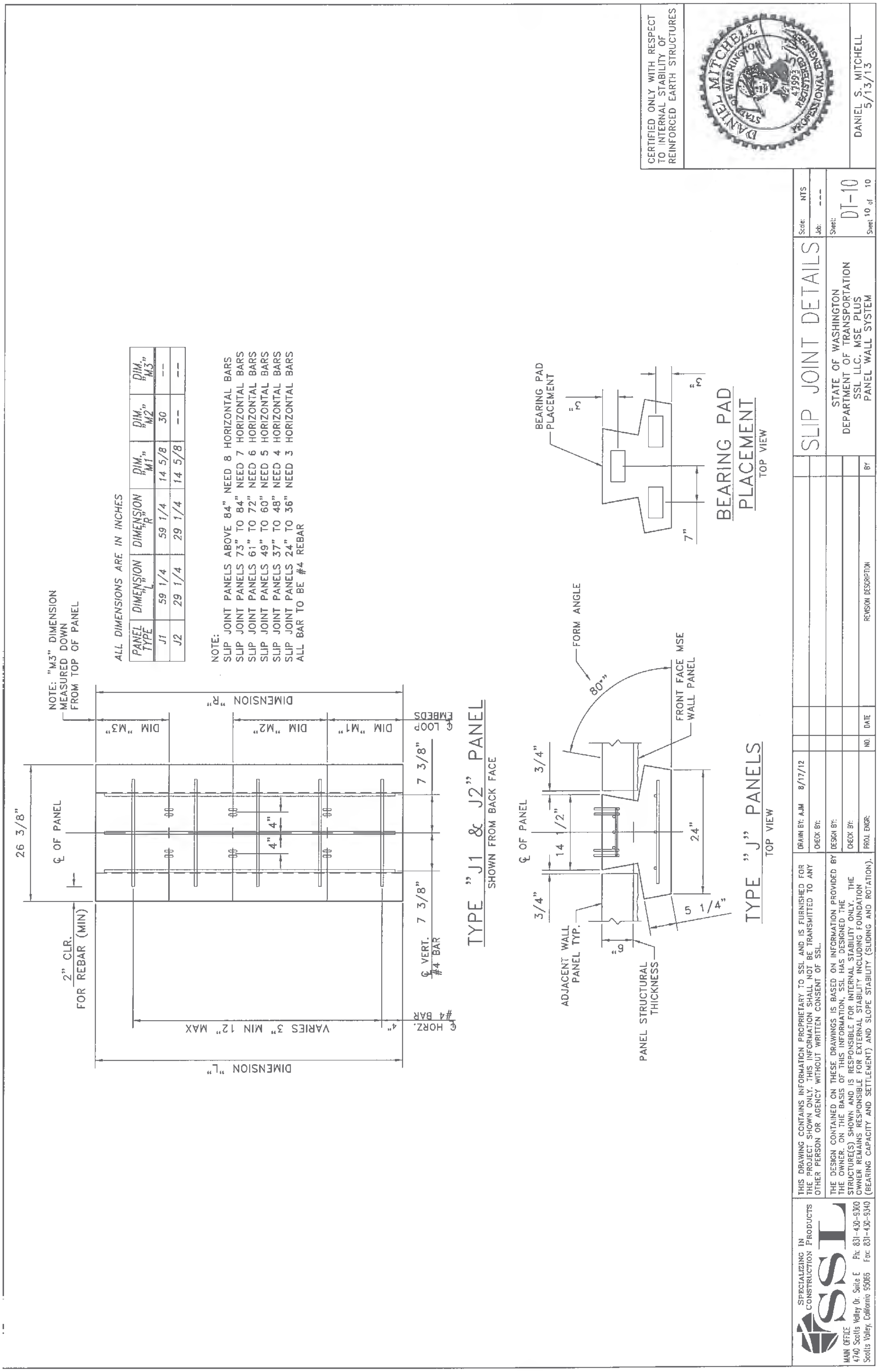
DT-09
Sheet 09 of 10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NOTE: "M3" DIMENSION MEASURED DOWN FROM TOP OF PANEL
ALL DIMENSIONS ARE IN INCHES
PANEL TYPE | DIMENSION "L" | DIMENSION "R" | DIM. "M1" | DIM. "M2" | DIM. "M3"
J1 | 59 1/4 | 59 1/4 | 14 5/8 | 30 | --
J2 | 29 1/4 | 29 1/4 | 14 5/8 | -- | --
NOTE:
SLIP JOINT PANELS ABOVE 84" NEED 8 HORIZONTAL BARS
SLIP JOINT PANELS 73" TO 84" NEED 7 HORIZONTAL BARS
SLIP JOINT PANELS 61" TO 72" NEED 6 HORIZONTAL BARS
SLIP JOINT PANELS 49" TO 60" NEED 5 HORIZONTAL BARS
SLIP JOINT PANELS 37" TO 48" NEED 4 HORIZONTAL BARS
SLIP JOINT PANELS 24" TO 36" NEED 3 HORIZONTAL BARS
ALL BAR TO BE #4 REBAR
26 3/8"
℄ OF PANEL
2" CLR. FOR REBAR (MIN)
DIMENSION "R"
DIM "M3"
DIM "M2"
DIM "M1"
℄ LOOP EMBEDS
DIMENSION "L"
VARIES 3" MIN 12" MAX
4"
4"
4"
℄ HORZ. #4 BAR
℄ VERT. #4 BAR
7 3/8"
7 3/8"
TYPE "J1 & J2" PANEL
SHOWN FROM BACK FACE
FORM ANGLE
80°
FRONT FACE MSE WALL PANEL
ADJACENT WALL PANEL TYP.
PANEL STRUCTURAL THICKNESS
3/4"
3/4"
14 1/2"
24"
5 1/4"
6"
TYPE "J" PANELS
TOP VIEW
BEARING PAD PLACEMENT
7"
3"
3"
BEARING PAD PLACEMENT
TOP VIEW
SLIP JOINT DETAILS
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION
SSL LLC. MSE PLUS PANEL WALL SYSTEM
Scale: NTS
Job: ---
Sheet: DT-10
Sheet 10 of 10
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Appendix 15-O Preapproved Wall Appendix: Specific Requirements and Details for Landmark Reinforced Soil Wall

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply:

Facing Blocks –Blocks acceptable for use are the Landmark tapered and straight blocks. These blocks can form facing batters of vertical (0 degrees) to 4 degrees. Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2.5 feet.

Soil Reinforcement – Only geosynthetic reinforcement listed in the QPL and which has been evaluated for connection strength with the Landmark wall system shall be used. Therefore, the following specific QPL geosynthetic reinforcement products are approved for use with this wall system:

Miragrid 5XT
Miragrid 8XT
Miragrid 10XT

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications.

Reinforcement/Facing Block Connection Requirements – The connection between Landmark facing units and the geosynthetic reinforcement is essentially a mechanical connection, with the possible exception of the connection when Miragrid 10XT is used. For mechanical connections, the connection resistance is generally not dependent on the normal force between blocks. The connection testing conducted for this wall system demonstrates that the connection is behaving as a mechanical connection for the Miragrid 5XT and 8XT. For the 10XT, the connection strength increases as normal stress increases. Therefore, it is likely that the connection with Miragrid 10XT is at least partially depending on frictional resistance. The design facing/reinforcement connection strength shall be as specified in the following table.

Table 15-O-1 Approved Connection Strength Design Values for Landmark Walls

Block	Geogrid Product	$T_{ultconn}$ (lbs/ft)	T_{lot} (lbs/ft)	CR_u	Creep Reduction Factor applicable to the Connection (use for RF_{CR} in Eq. 1)
Straight Block	Miragrid 5XT	2800+	3844	0.73	1.45*
	Miragrid 8XT	4000	6564	0.61	1.45*
	Miragrid 10XT	3948+N*Tan 16°	9456	$T_{ultconn}$/9456	1.2
Tapered Block	Miragrid 5XT	2837 – N*Tan7°	3844	$T_{ultconn}$/3844	1.45*
	Miragrid 8XT	4250 – N*Tan5°	6564	$T_{ultconn}$/6564	1.45*
	Miragrid 10XT	3770+N*Tan 30° to N = 2850 lbs/ft, and 5400 lbs/ft at N > 2850 lbs/ft	9456	$T_{ultconn}$/9456	1.2

N = normal load at reinforcement layer at facing, in lbs/ft of width parallel to face.

+This is a lower bound value – see connection test results in report by Bathurst, Clarabut Geotechnical Testing, Inc., Project report No. BCGT9930, 9/1/2000.

*Same as the value of RFCR reported in the QPL, Appendix D for these geogrid products.

T_{ac}, the long-term connection strength, shall be calculated as follows:

$$T_{ac} \times \frac{T_{MARV} \times CR_U}{RF_{CR} \times RF_D}$$

where,

T_{MARV} = the minimum average roll value for the ultimate geosynthetic strength T_{ult},
CR_u = the ultimate connection strength $T_{ultconn}$ divided by the lot specific ultimate tensile strength, T_{lot} (i.e., the lot of material specific to the connection testing),
RF_{CR} = creep reduction factor for the geosynthetic, and
RF_D = the durability reduction factor for the geosynthetic.

RF_{CR} and RF_D shall be as provided in the QPL, Appendix D, except as noted in the previous table. Regarding the Miragrid 10XT, the sustained load test results indicate that the connection resistance reduction due to creep is not as large as for the other two Miragrid products, likely due to the fact that at least some of the connection resistance is frictional in nature rather than fully mechanical. Therefore, the lower creep reduction factor for the Miragrid 10XT is acceptable.

It is noted in ASTM C1372 that a dimensional tolerance for the height of the block of ⅛ inch is allowed, but that Section 15.5.3.8 recommends a tighter dimensional tolerance of 1⁄16 inch. Based on WSDOT experience, for walls greater than 25 feet in height, some cracking of facing blocks due to differential vertical stresses tends to occur in the bottom portion of the wall. Therefore, blocks placed at depths below the wall top of 25 feet or more should be cast to a vertical dimensional tolerance of 1⁄16 inch to reduce the risk of significant cracking of facing blocks.

Approved details for the Landmark wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- In plan sheet 5 of 6, the guard rail detail, the guard rail post shall either be installed through precut holes in the geogrid layers that must penetrated, or the geogrid layers shall be cut in a manner that prevents ripping or tearing of the geogrid.
- In plan sheet 3 of 6, regarding the geogrid at wall corner detail, cords in the wall facing alignment to form an angle point or a radius shall be no shorter than the width of the roll to insure good contact between the connectors and the geogrid cross-bar throughout the width of the geogrid. Alternatively, the geogrid roll could be cut longitudinally in half to allow a tighter radius, if necessary.

PROPOSED SEGMENTAL RETAINING WALL PLANS FOR:

PROJECT NAME (1)
PROJECT NAME (2)
CITY, WASHINGTON

PART 1 GENERAL

1.01 SUMMARY

A. Section Includes
Work includes furnishing and installing Anchor Landmark modular retaining wall units to the lines and grades designated on the construction drawings and as specified herein.

1.02 REFERENCES

A. American Society of Testing and Materials
1. ASTM C 140 Standard Test Methods of Sampling and Testing Concrete Masonry Units
2. ASTM D 448 Standard Classification for Sizes of Aggregate for Road and Bridge Construction
3. ASTM C 698 Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb Rammer and 12-in. Drop, (Standard Proctor)
4. ASTM C 1262 Standard Test Method for Evaluating the Freeze-Thaw Durability of Manufactured Concrete Masonry Units and Related Concrete Units
5. ASTM C 1372 Standard Specification for Segmental Retaining Wall Units
6. ASTM D 1556 Standard Test Method for Density of Soil In Place by the Sand Cone Method
7. ASTM D 1557 Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-lb Rammer and 18-in. Drop, (Modified Proctor)
8. ASTM D 2488 Standard Practice for Description and Identification of Soils, Visual-Manual Procedure (USCS Unified Soil Classification System)
9. ASTM D 2922 Standard Test Method for Density of Soil and Soil-Aggregate In Place by Nuclear Methods (Shallow Depth)
10. ASTM D 4354 Practice for Sampling of Geosynthetics for Testing
11. ASTM D 4595 Test Method for Tensile Properties of Geosynthetics by the Wide Width Strip Method
12. ASTM D 4759 Practice for Determining Specification Conformance of Geosynthetics
13. ASTM D 4873 Standard Guide for Identification, Storage and Handling of Geotextiles
14. ASTM D 5262 Creep Limited Strength of Geosynthetics

B. American Association of State Highway Officials
1. AASHTO M 288 Standard Specification for Geotextile Specification for Highway Applications
2. AASHTO M 278 Standard Specification for Class PS46 Polyvinyl Chloride (PVC) Pipe
3. AASHTO M 304 Standard Specification for Poly (Vinyl Chloride) (PVC) Profile Wall Drain Pipe and Fittings Based on Controlled Inside Diameter
4. AASHTO LRFD Bridge Design Specifications, 4th Ed., 2007

C. Geosynthetic Research Institute
1. GRI GG4; Allowable Design Strength of Geosynthetics

D. WSDOT Standard Plans and Special Provisions

1.03 SUBMITTALS

A. Submit the following in accordance with Section _____:
1. Manufacturer's literature: Materials description
2. Shop drawings: Retaining wall system design, including wall heights, geosynthetic reinforcement layout, drainage provisions and other pertinent details. The shop drawings shall be signed by a registered professional engineer licensed in the state of wall installation.
3. Certificate of Compliance letter in accordance with Section ______ (Earth Retaining Structures, Proprietary Earth Retaining Systems) and Section ______ of the Standard Specifications
4. Design calculations demonstrating satisfaction safety factors for
   1) Overall stability
   2) Internal stability
   3) Bearing capacity
   4) Sliding

1.04 DELIVERY, STORAGE AND HANDLING

A. The contractor shall check the materials upon delivery to assure that proper material has been received.
B. Deliver and handle materials in such a manner as to prevent damage. Store above ground on wood pallets or blocking. Remove damaged or otherwise unsuitable material, when so determined, from the site.
C. The contractor shall prevent excessive mud, wet cement, epoxy and like material, from coming in contact with the modular units and reinforcement.
D. Geosynthetic materials shall remain in protective wrapping until placed in the wall. Follow manufacturer's recommendations regarding protection from direct sunlight.
E. Lock bar material shall remain in boxes until placed in the modular units. Lock bar exposed to direct sunlight for a period exceeding 2 months should not be used in the constructed wall.

1.05 DEFINITIONS

A. Geosynthetic Reinforcement is a material specifically fabricated for use as soil reinforcement.
B. Landmark modular retaining wall units are machine made from portland cement, water, mineral aggregates and potentially fly ash and various admixtures.
C. Permeable Material is a free draining material used as a drainage media.
D. Reinforced Backfill Soil is the soil used as fill within the geosynthetic reinforced soil mass.
E. Foundation Soil is the soil mass supporting the leveling pad and reinforced soil zone of the modular retaining wall system.
F. Wall Subdrain is a perforated PVC pipe, generally of 4 inch (100 mm) diameter, used to drain water from soil.
G. Filter Fabric is a non-woven geosynthetic material used for filtration and to allow for the long term passage of water into a subsurface drain system while retaining the in situ soil.
H. The Landmark Lock Bar is a specifically manufactured polymer based material, supplied by Anchor, and used to mechanically connect the reinforcement products to the Landmark units.

1.06 DISCREPANCIES

Should discrepancies exist between the plans and specifications, plans shall take precedence over the specifications.

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Scale: NOT TO SCALE	0 — 1 VERIFY THAT THE BAR ABOVE IS 1". IF NOT, ADJUST THE SCALE ACCORDINGLY.	PROPOSED SEGMENTAL RETAINING WALLS 0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM WSDOT SUBMITTAL DETAILS			
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PART 2 PRODUCTS

2.01 MATERIALS

A. Modular Block Facing Unit: "Landmark" as manufactured under license from Anchor Wall Systems. No substitutions allowed. Landmark units shall meet requirements of ASTM C1372 except as modified by FHWA NHI-00-043, section 8.10.
   1. Unit height dimensions shall not vary more than +/-1/16 inch (1.6mm) from that specified.
   2. Unit length dimensions shall not vary more than +/-1/8 inch (3.2mm) from that specified.
   3. Minimum required compressive strength shall be 4,000 psi (28Mpa).
   4. Maximum water absorption shall be 5%.
   5. Color: _____. Submit a block sample for approval by the resident engineer.
   6. Texture: Offset Split Face.
   7. The concrete units shall include an integral concrete shear flange connection/location device.
B. Geosynthetic reinforcement: Mirafi Miragrid 5XT, 8XT & 10XT. No substitutions allowed.
C. Connectors: The Landmark lock bar as supplied by Anchor Wall Systems shall be manufactured from CPVC and PVC to the dimensions shown on the plans.
D. Permeable material: Shall conform to the provisions in Section ____ (Earth Retaining Structures) of the Special Provisions for Permeable Materials
E. Reinforced Backfill Soil: Shall conform to the provisions in Section ____ (Earth Retaining Structures) of the Special Provisions for Structure Backfill
F. Wall subdrain: Shall conform to the provisions in Section ____ "Underdrains" of the Standard Specifications and the Special Provisions for this project.
G. Filter fabric: Shall conform to the provisions in Section ____ "Filter Fabric" of the Standard Specifications and the Special Provisions for this project.
H. Leveling Pad: A leveling pad of unreinforced concrete shall be placed to facilitate first course placement. Concrete leveling pads shall have a minimum thickness of 6 inches (150mm) and a minimum width of 24 inches (600mm) and shall have a minimum 28 day compressive strength of 3,000 psi (20.7 Mpa).

PART 3 EXECUTION

3.01 EXAMINATION

A. The contractor shall examine the areas and conditions under which the retaining wall is to be erected and notify the owners representative in writing of conditions detrimental to the proper and timely completion of the work. Contractor shall not proceed with the work until unsatisfactory conditions have been corrected. The contractor shall promptly notify the wall design engineer and owners representative of site condition which may affect wall performance or may require a reevaluation of the wall design.
B. Foundation soil and any cut banks shall be examined by the project geotechnical engineer or technician to ensure that the actual retained and foundation soil strengths, meets or exceeds the strength required, as shown on the construction drawings. Examine cut banks for perched or other water seepage.

3.02 EXCAVATION

A. The contractor shall excavate to the lines and grades shown on the construction drawings. Over-excavation not approved by the owner or owner's representative shall not be paid for and replacement with approved compacted fill and/or wall system components will be required at the Contractor's expense. Do not disturb base beyond the lines shown on the plans.

3.03 FOUNDATION PREPARATION

A. Foundation soil shall be excavated as required for the base reinforcement dimension shown on the construction drawings, or as directed by the engineer.
B. The project geotechnical engineer shall examine the foundation and retained soils to ensure that the soil strength and type meets or exceeds that required, as shown on the construction drawings. Foundation soil not meeting the strength required for bearing capacity or settlement, shall be remediated per the direction of the engineer.

3.04 BASE COURSE PREPARATION

A. Leveling pad materials shall be placed as shown on the construction drawings on the pre-approved foundation soils.
B. Leveling pad materials shall be installed upon undisturbed soils, or foundation soils prepared in accordance with Section 3.03.
C. Concrete leveling pads shall be allowed to cure for 12 hours prior to placement of the first course of modular units.
D. Leveling pad materials shall be prepared to provide intimate contact with the modular wall units.
E. Leveling pad materials shall be to the depths and widths shown on the plans.

3.05 WALL ERECTION

A. Foundation units shall be placed on the prepared leveling pad. Units shall be checked for horizontal alignment with a string line placed at the back of the units and vertical alignment front to back and side to side with a level. The top of all units in the base course shall be at the same elevation.
B. Ensure that concrete wall units are in full contact with base. A 1 inch (25 mm) gap between foundations units is allowed, provided a suitable filter fabric is placed behind the foundation units.
C. The foundation course of modular units shall be backfilled and compacted, front and back, then checked for level and alignment prior to placing the next course of wall units.
D. Wall subdrain shall be installed at the lowest elevation possible to maintain gravity flow of water to outside of the reinforced zone. The wall subdrain shall be day-lighted to an appropriate location away from the wall system at each low point and at 50-foot (15 m) intervals along the wall.
E. Remove all excess fill from top of units and from the lock bar channel in the top of the units and install next course.
F. Subsequent courses of modular units shall be placed side by side for full length of wall alignment. A maximum gap of 1/8 inch (3.2 mm) is allowed between units. Alignment should be checked by using a string line at the back of the units. Adjust units as necessary to maintain horizontal alignment.
G. If required, a minimum of 12 inches (300 mm) of angular permeable material shall be placed behind the modular units.
H. A filter fabric may be required between the permeable material and reinforced soil fill depending on the compatibility of the permeable material and reinforced soil zone materials.
I. Ensure permeable material and backfill are compacted before installation of each succeeding course.
J. Install each succeeding course. Backfill as each course is completed and prior to placement of the next course. Pull the units forward until the locating surface of the unit contacts the locating surface of the units in the preceding course.
K. Check unit vertical alignment with a level on each course, adjust units as necessary with reinforcement shims to maintain proper alignment and setback control.
L. Permeable material or reinforced soil fill must be placed level with the top of the modular units at courses where reinforcement is required.
M. Remove all excess fill from top of units and from the lock bar channel in the top of the units prior to reinforcement placement.
N. Install geosynthetic reinforcement at locations and elevations shown on the design drawings.
   1. The geosynthetic reinforcement has a primary strength direction. The primary strength direction must be placed perpendicular to the wall face.
   2. Reinforcement panels shall be continuous. Seams or connections are not permitted. Adjacent panels shall be abutted with less than 4 inch gap between adjacent panels. 100 percent reinforcement coverage is required.
   3. Panels of geosynthetic reinforcement shall be tensioned such that all folds and wrinkles are removed before reinforced soil is placed. Panels shall be staked or anchored as necessary to maintain taut condition.
   4. Tracked vehicles may not operate on geosynthetic reinforcement with less than 6 inches of compacted soil between the reinforcement and the tracks. Turning of tracked vehicles should be kept to a minimum to prevent damage and disturbance to the reinforcement.
   5. Rubber-tired vehicles may operate directly on geosynthetic reinforcement at speeds less than 10 mph if permitted by the reinforcement manufacturer. Sudden braking and turning shall be avoided.
O. The Landmark lock bar shall be placed at each geosynthetic elevation. Lock bar is not required on courses where geosynthetic reinforcement is not placed. Gaps between adjacent sections of lock bar shall be no greater than 3 inches (75 mm). The lock bar shall be placed flat side up, with the angled side to the back of the unit, as shown on the construction drawings. The reinforcement must be maintained within 1" (25 mm) of the face of the smaller Landmark units below.

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Proj. No:	Reviewed by: AWE	REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):	GENERAL NOTES
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3.06 BACKFILL PLACEMENT

A. Special care shall be taken during compaction below the first reinforcement layer to maintain unit level and alignment.
B. At each level of soil reinforcement the backfill material shall be roughly level to an elevation approximately 1" (30mm) above the level of the facing unit before placing the soil reinforcement.
C. Clean any debris off the top of the units and from within the channel in the top of the units and ensure the backfill is graded reasonably flat prior to reinforcement placement.
D. The reinforcement has a primary strength direction, which must be laid perpendicular to the wall face.
E. Prior to placement of backfill and after placement of the lock bar, pull the reinforcement taut and anchor in place with stakes, staples or fill piles at the back of the reinforcement.
F. Place the reinforced backfill onto the reinforcement and spread in a direction parallel to the wall face. Reinforced backfill shall be placed, spread and compacted in a manner that will minimize slack or wrinkles from forming in the reinforcement.
G. Place a minimum of 6" (150mm) of backfill prior to operating equipment above the reinforcement. Avoid sudden braking or turning on fill placed over the reinforcement.
H. Fill in the reinforced soil zone shall be placed and compacted in lifts not to exceed 6 to 8 inches (150 to 200 mm) in loose thickness where hand operated compaction equipment is used, and not exceeding 12 inches (300 mm) in loose thickness where heavy, self-propelled compaction equipment is used.
I. Only lightweight, hand-operated, compaction equipment shall be allowed within 3 feet (0.9 m) of the back of the Landmark units.
J. All fill placed in the reinforced zone must be compacted in accordance with the project specifications and the project engineer.
K. Compaction tests shall be taken in the reinforced soil zone. A minimum frequency of one test within the reinforced soil zone per every 5 feet (1.5 m) of wall height for every 100 ft (30 m) of wall is recommended.
L. Prior to periods of construction inactivity, the reinforced backfill should be graded to drain away from the wall face. Trenches or berms may be needed to control surface drainage in the vicinity of the retained cut slope, reinforced backfill or wall toe area.

3.07 CAP UNIT INSTALLATION (Where required)

A. Brush clean the top of the upper course of units. Place cap units, cutting as necessary on curved wall portions, prior to adhering the cap units.
B. Mortar is the preferred material to adhere the cap units to the upper course of modular units.
C. Apply mortar or an exterior concrete construction adhesive to the top surface of the upper course of units, and place the cap unit into desired position. If mortar is used, place mortar into the channel in the top course of units as well as on the upper surfaces.
D. Use a string line to maintain proper cap alignment.
E. Backfill and compact to finish grade, after mortar or adhesive has set.

3.08 ADJUSTING AND CLEANING

A. Damaged units should be replaced with new units during construction.
B. Contractor shall remove debris caused by construction and leave adjacent areas clean.

3.09 QUALITY CONTROL

A. The wall installation contractor is responsible for the proper installation of all materials. A qualified independent third party shall be enlisted to verify the correct installation of all materials according to these specifications and the construction drawings.
B. Work found to be deficient according to these specifications or the construction drawings must be corrected.
C. The retaining wall will not be considered complete until accepted by the engineer or duly appointed owner's representative.

PART 4 DESIGN PARAMETERS

4.01 SOIL PARAMETERS FROM GENERAL NOTES ON WSDOT PLAN SHEET RW-____

A. Reinforced/Backfill Soil: Angle of Internal Friction = 34-degrees, Unit Weight = 120 pcf
B. Retained Soil: Angle of Internal Friction = 30-degrees, Unit Weight = 120 pcf
C. Designed in accordance with
   1. AASHTO LRFD Bridge Design Specifications, 4th Ed., 2007

General Note: ____________________ will be supplying the retaining wall blocks and lock bar for use with other accessory materials to be used in the construction of walls detailed herein. The Landmark Wall System being provided for this project will have an offset split texture for the facing. Construction and quality control procedures provided by Anchor Wall Systems and its licensees are intended to provide a general explanation of the system. It is the contractor's obligation to unload and store the products delivered to the site, devise and execute a project specific erection sequence, and implement a fall protection system where required. Construction and installation processes and procedures provided by Anchor Wall Systems and its licensees are general in nature and do not account for project specific criteria. Compliance with the guidelines in such documents does not relieve the contractor of its responsibility to adhere to the project plans, specifications and contract documents or compliance with all fall protection, safety laws, standards, and procedures at the jobsite.

END OF SECTION

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Scale: NOT TO SCALE	0 1 VERIFY THAT THE BAR ABOVE IS 1". IF NOT, ADJUST THE SCALE ACCORDINGLY.	PROPOSED SEGMENTAL RETAINING WALLS 0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM WSDOT SUBMITTAL DETAILS		
Proj. No:	Reviewed by: AWE	REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):	GENERAL NOTES	Sheet W3 of 10
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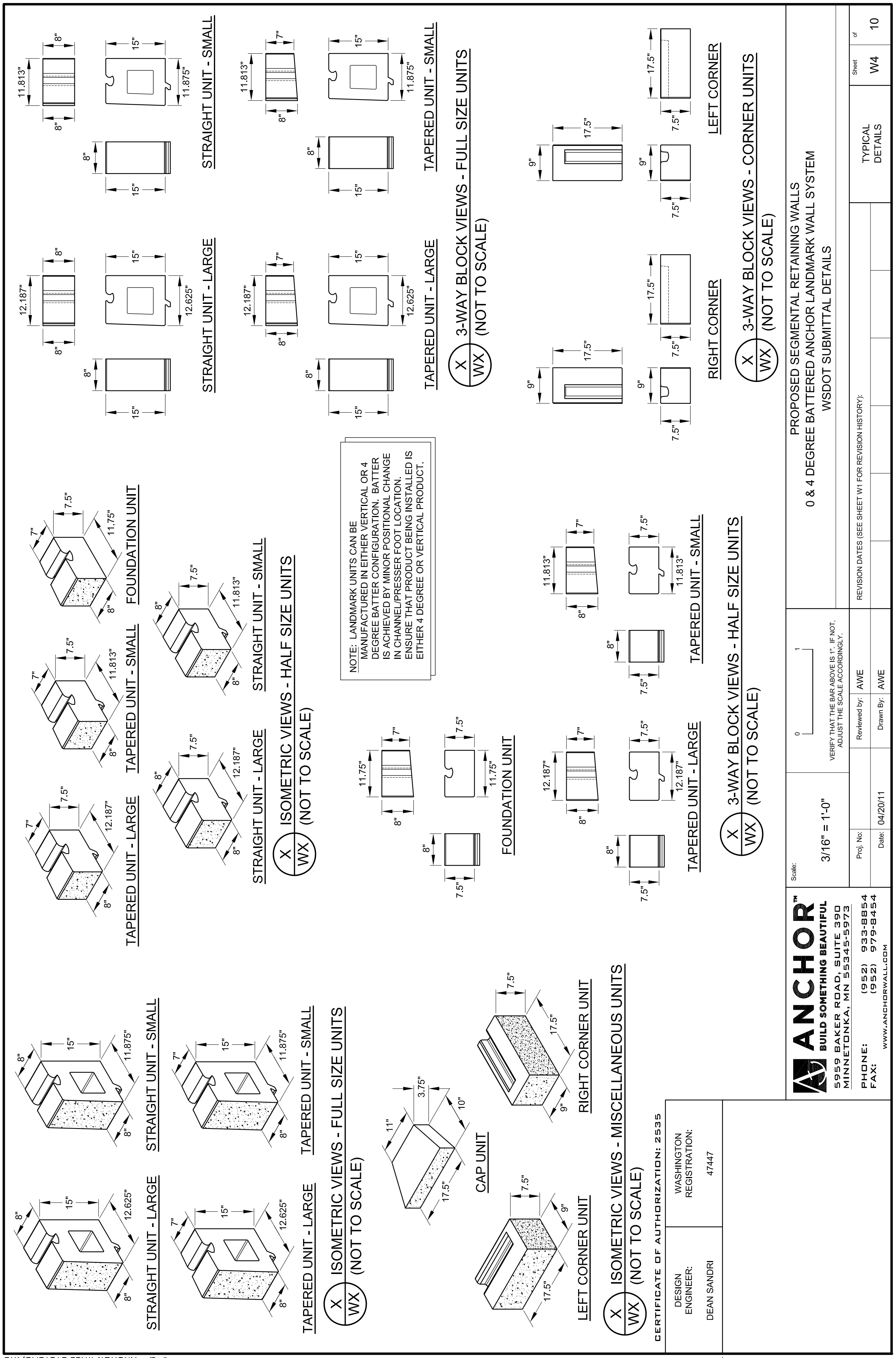
STRAIGHT UNIT - LARGE
STRAIGHT UNIT - SMALL
TAPERED UNIT - LARGE
TAPERED UNIT - SMALL
X / WX ISOMETRIC VIEWS - FULL SIZE UNITS (NOT TO SCALE)
LEFT CORNER UNIT
CAP UNIT
RIGHT CORNER UNIT
X / WX ISOMETRIC VIEWS - MISCELLANEOUS UNITS (NOT TO SCALE)
FOUNDATION UNIT
X / WX ISOMETRIC VIEWS - HALF SIZE UNITS (NOT TO SCALE)
NOTE: LANDMARK UNITS CAN BE MANUFACTURED IN EITHER VERTICAL OR 4 DEGREE BATTER CONFIGURATION. BATTER IS ACHIEVED BY MINOR POSITIONAL CHANGE IN CHANNEL/PRESSER FOOT LOCATION. ENSURE THAT PRODUCT BEING INSTALLED IS EITHER 4 DEGREE OR VERTICAL PRODUCT.
X / WX 3-WAY BLOCK VIEWS - HALF SIZE UNITS (NOT TO SCALE)
X / WX 3-WAY BLOCK VIEWS - FULL SIZE UNITS (NOT TO SCALE)
RIGHT CORNER
LEFT CORNER
X / WX 3-WAY BLOCK VIEWS - CORNER UNITS (NOT TO SCALE)
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VERIFY THAT THE BAR ABOVE IS 1". IF NOT, ADJUST THE SCALE ACCORDINGLY.
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Date: 04/20/11
Reviewed by: AWE
Drawn By: AWE
REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):
PROPOSED SEGMENTAL RETAINING WALLS
0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM
WSDOT SUBMITTAL DETAILS
TYPICAL DETAILS
Sheet W4 of 10
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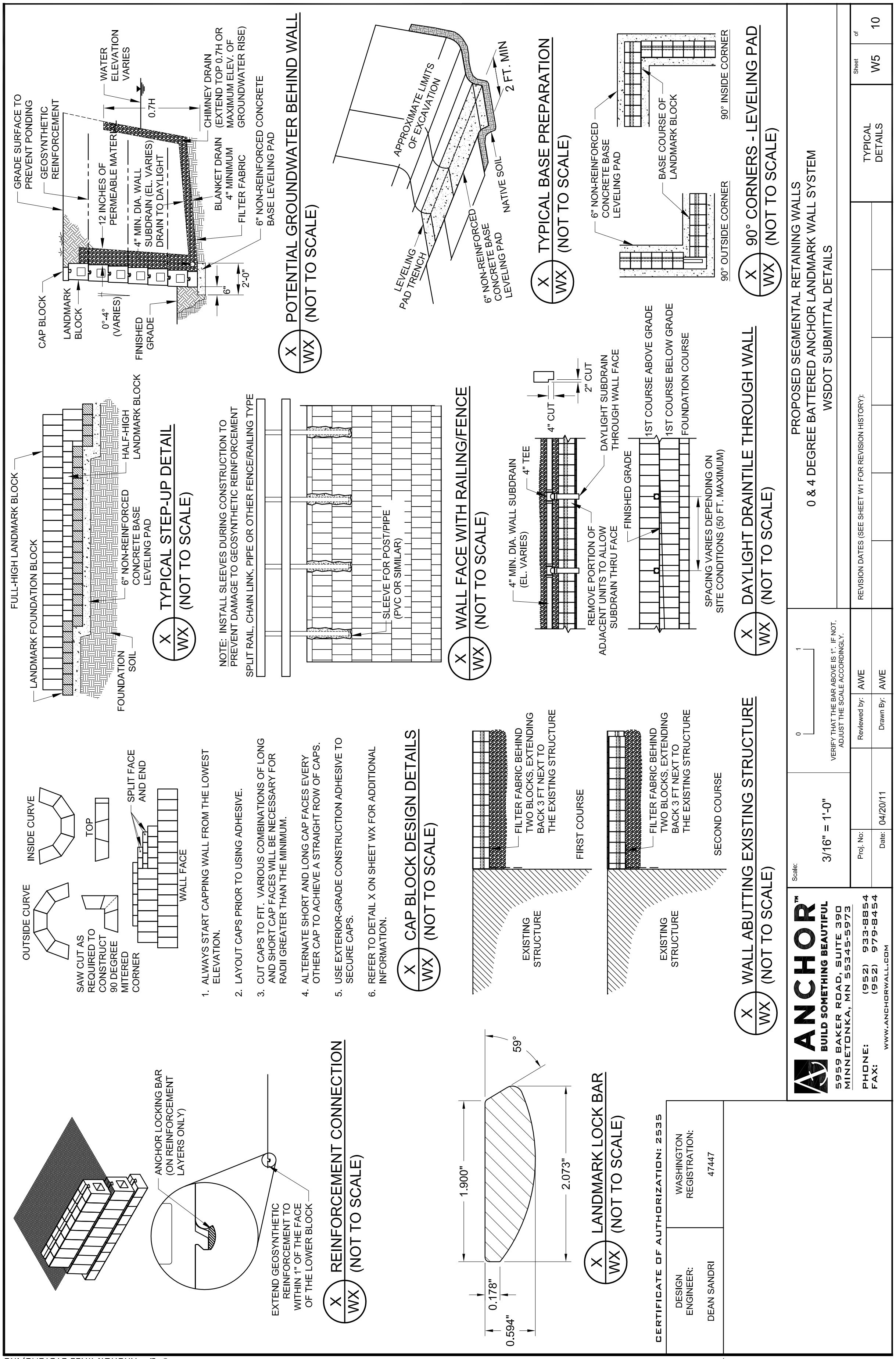
POTENTIAL GROUNDWATER BEHIND WALL
(NOT TO SCALE)
CAP BLOCK
LANDMARK BLOCK
0°-4° (VARIES)
FINISHED GRADE
GRADE SURFACE TO PREVENT PONDING
GEOSYNTHETIC REINFORCEMENT
12 INCHES OF PERMEABLE MATERIAL
4" MIN. DIA. WALL SUBDRAIN (EL. VARIES) DRAIN TO DAYLIGHT
WATER ELEVATION VARIES
0.7H
CHIMNEY DRAIN (EXTEND TOP 0.7H OR MAXIMUM ELEV. OF GROUNDWATER RISE)
BLANKET DRAIN 4" MINIMUM
FILTER FABRIC
6" NON-REINFORCED CONCRETE BASE LEVELING PAD
6"
2'-0"
TYPICAL BASE PREPARATION
(NOT TO SCALE)
APPROXIMATE LIMITS OF EXCAVATION
2 FT. MIN
LEVELING PAD TRENCH
6" NON-REINFORCED CONCRETE BASE LEVELING PAD
NATIVE SOIL
90° CORNERS - LEVELING PAD
(NOT TO SCALE)
6" NON-REINFORCED CONCRETE BASE LEVELING PAD
BASE COURSE OF LANDMARK BLOCK
90° OUTSIDE CORNER
90° INSIDE CORNER
TYPICAL STEP-UP DETAIL
(NOT TO SCALE)
FULL-HIGH LANDMARK BLOCK
LANDMARK FOUNDATION BLOCK
6" NON-REINFORCED CONCRETE BASE LEVELING PAD
HALF-HIGH LANDMARK BLOCK
FOUNDATION SOIL
WALL FACE WITH RAILING/FENCE
(NOT TO SCALE)
NOTE: INSTALL SLEEVES DURING CONSTRUCTION TO PREVENT DAMAGE TO GEOSYNTHETIC REINFORCEMENT
SPLIT RAIL, CHAIN LINK, PIPE OR OTHER FENCE/RAILING TYPE
SLEEVE FOR POST/PIPE (PVC OR SIMILAR)
DAYLIGHT DRAINTILE THROUGH WALL
(NOT TO SCALE)
4" MIN. DIA. WALL SUBDRAIN (EL. VARIES)
4" TEE
4" CUT
2" CUT
DAYLIGHT SUBDRAIN THROUGH WALL FACE
REMOVE PORTION OF ADJACENT UNITS TO ALLOW SUBDRAIN THRU FACE
FINISHED GRADE
1ST COURSE ABOVE GRADE
1ST COURSE BELOW GRADE
FOUNDATION COURSE
SPACING VARIES DEPENDING ON SITE CONDITIONS (50 FT. MAXIMUM)
CAP BLOCK DESIGN DETAILS
(NOT TO SCALE)
OUTSIDE CURVE
INSIDE CURVE
TOP
SPLIT FACE AND END
SAW CUT AS REQUIRED TO CONSTRUCT 90 DEGREE MITERED CORNER
WALL FACE
1. ALWAYS START CAPPING WALL FROM THE LOWEST ELEVATION.
2. LAYOUT CAPS PRIOR TO USING ADHESIVE.
3. CUT CAPS TO FIT. VARIOUS COMBINATIONS OF LONG AND SHORT CAP FACES WILL BE NECESSARY FOR RADII GREATER THAN THE MINIMUM.
4. ALTERNATE SHORT AND LONG CAP FACES EVERY OTHER CAP TO ACHIEVE A STRAIGHT ROW OF CAPS.
5. USE EXTERIOR-GRADE CONSTRUCTION ADHESIVE TO SECURE CAPS.
6. REFER TO DETAIL X ON SHEET WX FOR ADDITIONAL INFORMATION.
WALL ABUTTING EXISTING STRUCTURE
(NOT TO SCALE)
EXISTING STRUCTURE
FILTER FABRIC BEHIND TWO BLOCKS, EXTENDING BACK 3 FT NEXT TO THE EXISTING STRUCTURE
FIRST COURSE
SECOND COURSE
REINFORCEMENT CONNECTION
(NOT TO SCALE)
ANCHOR LOCKING BAR (ON REINFORCEMENT LAYERS ONLY)
EXTEND GEOSYNTHETIC REINFORCEMENT TO WITHIN 1" OF THE FACE OF THE LOWER BLOCK
LANDMARK LOCK BAR
(NOT TO SCALE)
59°
1.900"
2.073"
0.178"
0.594"
X WX
CERTIFICATE OF AUTHORIZATION: 2535
DESIGN ENGINEER: DEAN SANDRI
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Drawn By: AWE
PROPOSED SEGMENTAL RETAINING WALLS
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WSDOT SUBMITTAL DETAILS
REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):
TYPICAL DETAILS
Sheet W5 of 10
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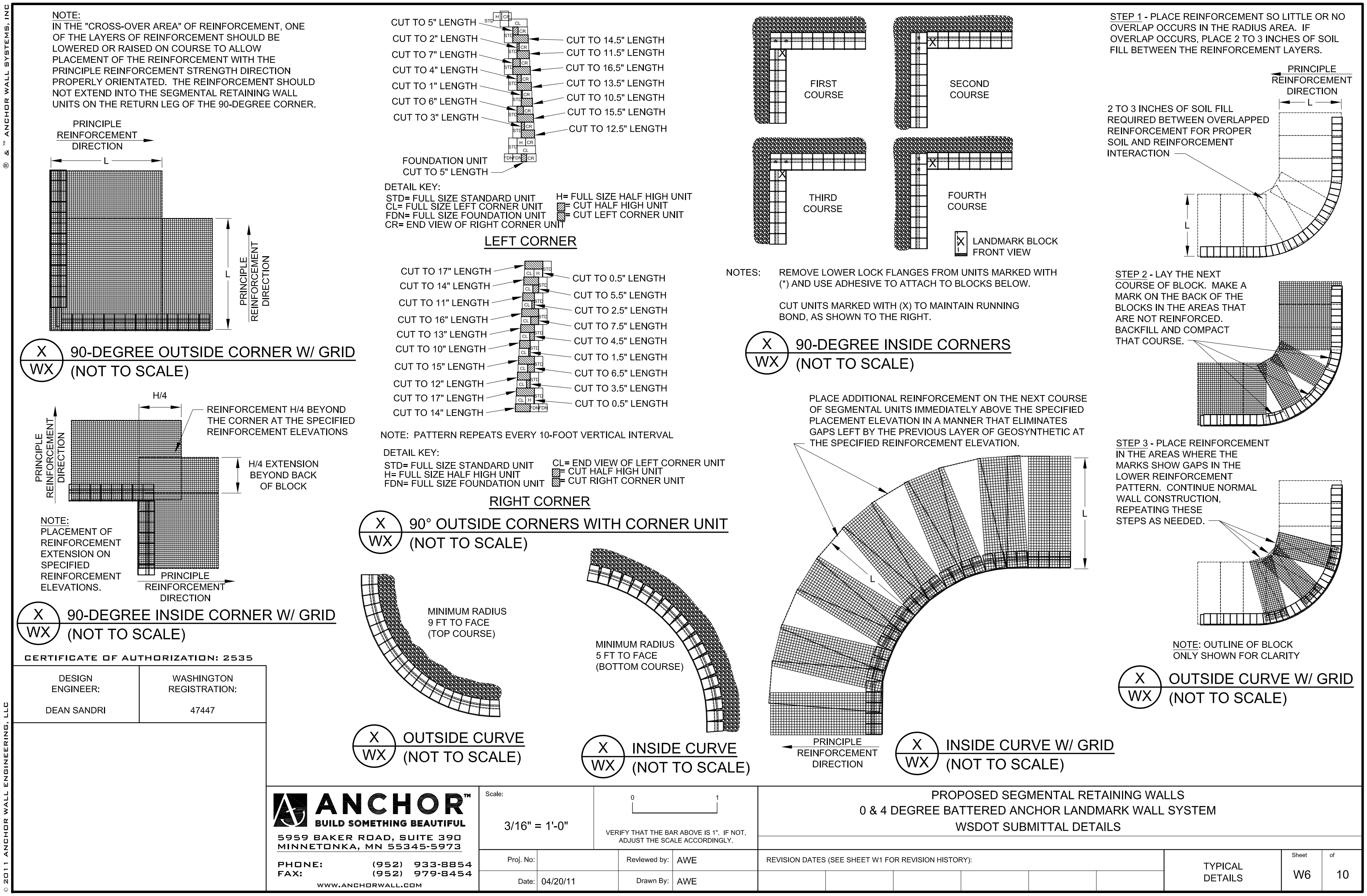

NOTE:
IN THE "CROSS-OVER AREA" OF REINFORCEMENT, ONE OF THE LAYERS OF REINFORCEMENT SHOULD BE LOWERED OR RAISED ON COURSE TO ALLOW PLACEMENT OF THE REINFORCEMENT WITH THE PRINCIPLE REINFORCEMENT STRENGTH DIRECTION PROPERLY ORIENTATED. THE REINFORCEMENT SHOULD NOT EXTEND INTO THE SEGMENTAL RETAINING WALL UNITS ON THE RETURN LEG OF THE 90-DEGREE CORNER.
PRINCIPLE REINFORCEMENT DIRECTION
L
X WX 90-DEGREE OUTSIDE CORNER W/ GRID (NOT TO SCALE)
H/4
REINFORCEMENT H/4 BEYOND THE CORNER AT THE SPECIFIED REINFORCEMENT ELEVATIONS
H/4 EXTENSION BEYOND BACK OF BLOCK
NOTE:
PLACEMENT OF REINFORCEMENT EXTENSION ON SPECIFIED REINFORCEMENT ELEVATIONS.
X WX 90-DEGREE INSIDE CORNER W/ GRID (NOT TO SCALE)
CUT TO 5" LENGTH
CUT TO 2" LENGTH
CUT TO 7" LENGTH
CUT TO 4" LENGTH
CUT TO 1" LENGTH
CUT TO 6" LENGTH
CUT TO 3" LENGTH
CUT TO 14.5" LENGTH
CUT TO 11.5" LENGTH
CUT TO 16.5" LENGTH
CUT TO 13.5" LENGTH
CUT TO 10.5" LENGTH
CUT TO 15.5" LENGTH
CUT TO 12.5" LENGTH
FOUNDATION UNIT CUT TO 5" LENGTH
DETAIL KEY:
STD= FULL SIZE STANDARD UNIT
CL= FULL SIZE LEFT CORNER UNIT
FDN= FULL SIZE FOUNDATION UNIT
CR= END VIEW OF RIGHT CORNER UNIT
H= FULL SIZE HALF HIGH UNIT
= CUT HALF HIGH UNIT
= CUT LEFT CORNER UNIT
LEFT CORNER
CUT TO 17" LENGTH
CUT TO 14" LENGTH
CUT TO 11" LENGTH
CUT TO 16" LENGTH
CUT TO 13" LENGTH
CUT TO 10" LENGTH
CUT TO 15" LENGTH
CUT TO 12" LENGTH
CUT TO 17" LENGTH
CUT TO 14" LENGTH
CUT TO 0.5" LENGTH
CUT TO 5.5" LENGTH
CUT TO 2.5" LENGTH
CUT TO 7.5" LENGTH
CUT TO 4.5" LENGTH
CUT TO 1.5" LENGTH
CUT TO 6.5" LENGTH
CUT TO 3.5" LENGTH
CUT TO 0.5" LENGTH
NOTE: PATTERN REPEATS EVERY 10-FOOT VERTICAL INTERVAL
DETAIL KEY:
STD= FULL SIZE STANDARD UNIT
H= FULL SIZE HALF HIGH UNIT
FDN= FULL SIZE FOUNDATION UNIT
CL= END VIEW OF LEFT CORNER UNIT
= CUT HALF HIGH UNIT
= CUT RIGHT CORNER UNIT
RIGHT CORNER
X WX 90° OUTSIDE CORNERS WITH CORNER UNIT (NOT TO SCALE)
MINIMUM RADIUS 9 FT TO FACE (TOP COURSE)
X WX OUTSIDE CURVE (NOT TO SCALE)
MINIMUM RADIUS 5 FT TO FACE (BOTTOM COURSE)
X WX INSIDE CURVE (NOT TO SCALE)
FIRST COURSE
SECOND COURSE
THIRD COURSE
FOURTH COURSE
LANDMARK BLOCK FRONT VIEW
NOTES: REMOVE LOWER LOCK FLANGES FROM UNITS MARKED WITH (*) AND USE ADHESIVE TO ATTACH TO BLOCKS BELOW.
CUT UNITS MARKED WITH (X) TO MAINTAIN RUNNING BOND, AS SHOWN TO THE RIGHT.
X WX 90-DEGREE INSIDE CORNERS (NOT TO SCALE)
PLACE ADDITIONAL REINFORCEMENT ON THE NEXT COURSE OF SEGMENTAL UNITS IMMEDIATELY ABOVE THE SPECIFIED PLACEMENT ELEVATION IN A MANNER THAT ELIMINATES GAPS LEFT BY THE PREVIOUS LAYER OF GEOSYNTHETIC AT THE SPECIFIED REINFORCEMENT ELEVATION.
X WX INSIDE CURVE W/ GRID (NOT TO SCALE)
STEP 1 - PLACE REINFORCEMENT SO LITTLE OR NO OVERLAP OCCURS IN THE RADIUS AREA. IF OVERLAP OCCURS, PLACE 2 TO 3 INCHES OF SOIL FILL BETWEEN THE REINFORCEMENT LAYERS.
2 TO 3 INCHES OF SOIL FILL REQUIRED BETWEEN OVERLAPPED REINFORCEMENT FOR PROPER SOIL AND REINFORCEMENT INTERACTION
STEP 2 - LAY THE NEXT COURSE OF BLOCK. MAKE A MARK ON THE BACK OF THE BLOCKS IN THE AREAS THAT ARE NOT REINFORCED. BACKFILL AND COMPACT THAT COURSE.
STEP 3 - PLACE REINFORCEMENT IN THE AREAS WHERE THE MARKS SHOW GAPS IN THE LOWER REINFORCEMENT PATTERN. CONTINUE NORMAL WALL CONSTRUCTION, REPEATING THESE STEPS AS NEEDED.
NOTE: OUTLINE OF BLOCK ONLY SHOWN FOR CLARITY
X WX OUTSIDE CURVE W/ GRID (NOT TO SCALE)
CERTIFICATE OF AUTHORIZATION: 2535
DESIGN ENGINEER: DEAN SANDRI
WASHINGTON REGISTRATION: 47447
ANCHOR™ BUILD SOMETHING BEAUTIFUL
5959 BAKER ROAD, SUITE 390
MINNETONKA, MN 55345-5973
PHONE: (952) 933-8854
FAX: (952) 979-8454
WWW.ANCHORWALL.COM
Scale: 3/16" = 1'-0"
0 1
VERIFY THAT THE BAR ABOVE IS 1". IF NOT, ADJUST THE SCALE ACCORDINGLY.
PROPOSED SEGMENTAL RETAINING WALLS
0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM
WSDOT SUBMITTAL DETAILS
Proj. No:
Date: 04/20/11
Reviewed by: AWE
Drawn By: AWE
REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):
TYPICAL DETAILS
Sheet W6 of 10
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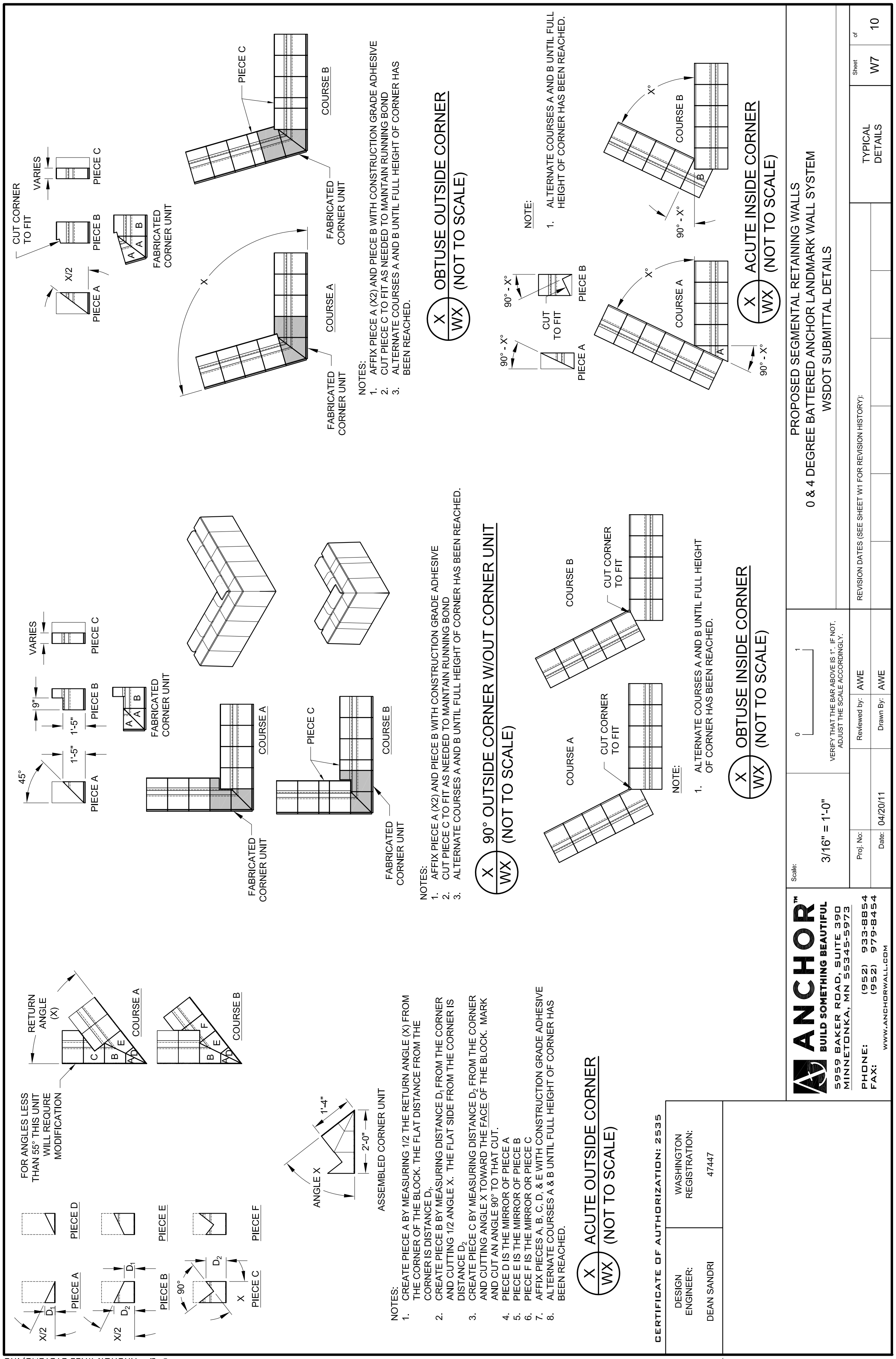

ACUTE OUTSIDE CORNER
(NOT TO SCALE)
PIECE A
PIECE B
PIECE C
PIECE D
PIECE E
PIECE F
X/2
D_1
D_2
90°
X
FOR ANGLES LESS THAN 55° THIS UNIT WILL REQUIRE MODIFICATION
RETURN ANGLE (X)
COURSE A
COURSE B
ANGLE X
1'-4"
2'-0"
ASSEMBLED CORNER UNIT
NOTES:
1. CREATE PIECE A BY MEASURING 1/2 THE RETURN ANGLE (X) FROM THE CORNER OF THE BLOCK. THE FLAT DISTANCE FROM THE CORNER IS DISTANCE D_1.
2. CREATE PIECE B BY MEASURING DISTANCE D_1 FROM THE CORNER AND CUTTING 1/2 ANGLE X. THE FLAT SIDE FROM THE CORNER IS DISTANCE D_2.
3. CREATE PIECE C BY MEASURING DISTANCE D_2 FROM THE CORNER AND CUTTING ANGLE X TOWARD THE FACE OF THE BLOCK. MARK AND CUT AN ANGLE 90° TO THAT CUT.
4. PIECE D IS THE MIRROR OF PIECE A
5. PIECE E IS THE MIRROR OF PIECE B
6. PIECE F IS THE MIRROR OR PIECE C
7. AFFIX PIECES A, B, C, D, & E WITH CONSTRUCTION GRADE ADHESIVE
8. ALTERNATE COURSES A & B UNTIL FULL HEIGHT OF CORNER HAS BEEN REACHED.
90° OUTSIDE CORNER W/OUT CORNER UNIT
(NOT TO SCALE)
45°
1'-5"
1'-5"
9"
VARIES
FABRICATED CORNER UNIT
NOTES:
1. AFFIX PIECE A (X2) AND PIECE B WITH CONSTRUCTION GRADE ADHESIVE
2. CUT PIECE C TO FIT AS NEEDED TO MAINTAIN RUNNING BOND
3. ALTERNATE COURSES A AND B UNTIL FULL HEIGHT OF CORNER HAS BEEN REACHED.
OBTUSE INSIDE CORNER
(NOT TO SCALE)
CUT CORNER TO FIT
NOTE:
1. ALTERNATE COURSES A AND B UNTIL FULL HEIGHT OF CORNER HAS BEEN REACHED.
OBTUSE OUTSIDE CORNER
(NOT TO SCALE)
CUT CORNER TO FIT
NOTES:
1. AFFIX PIECE A (X2) AND PIECE B WITH CONSTRUCTION GRADE ADHESIVE
2. CUT PIECE C TO FIT AS NEEDED TO MAINTAIN RUNNING BOND
3. ALTERNATE COURSES A AND B UNTIL FULL HEIGHT OF CORNER HAS BEEN REACHED.
ACUTE INSIDE CORNER
(NOT TO SCALE)
90° - X°
X°
CUT TO FIT
NOTE:
1. ALTERNATE COURSES A AND B UNTIL FULL HEIGHT OF CORNER HAS BEEN REACHED.
CERTIFICATE OF AUTHORIZATION: 2535
DESIGN ENGINEER: DEAN SANDRI
WASHINGTON REGISTRATION: 47447
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VERIFY THAT THE BAR ABOVE IS 1". IF NOT, ADJUST THE SCALE ACCORDINGLY.
Proj. No:
Reviewed by: AWE
Date: 04/20/11
Drawn By: AWE
REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):
PROPOSED SEGMENTAL RETAINING WALLS
0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM
WSDOT SUBMITTAL DETAILS
TYPICAL DETAILS
Sheet W7 of 10
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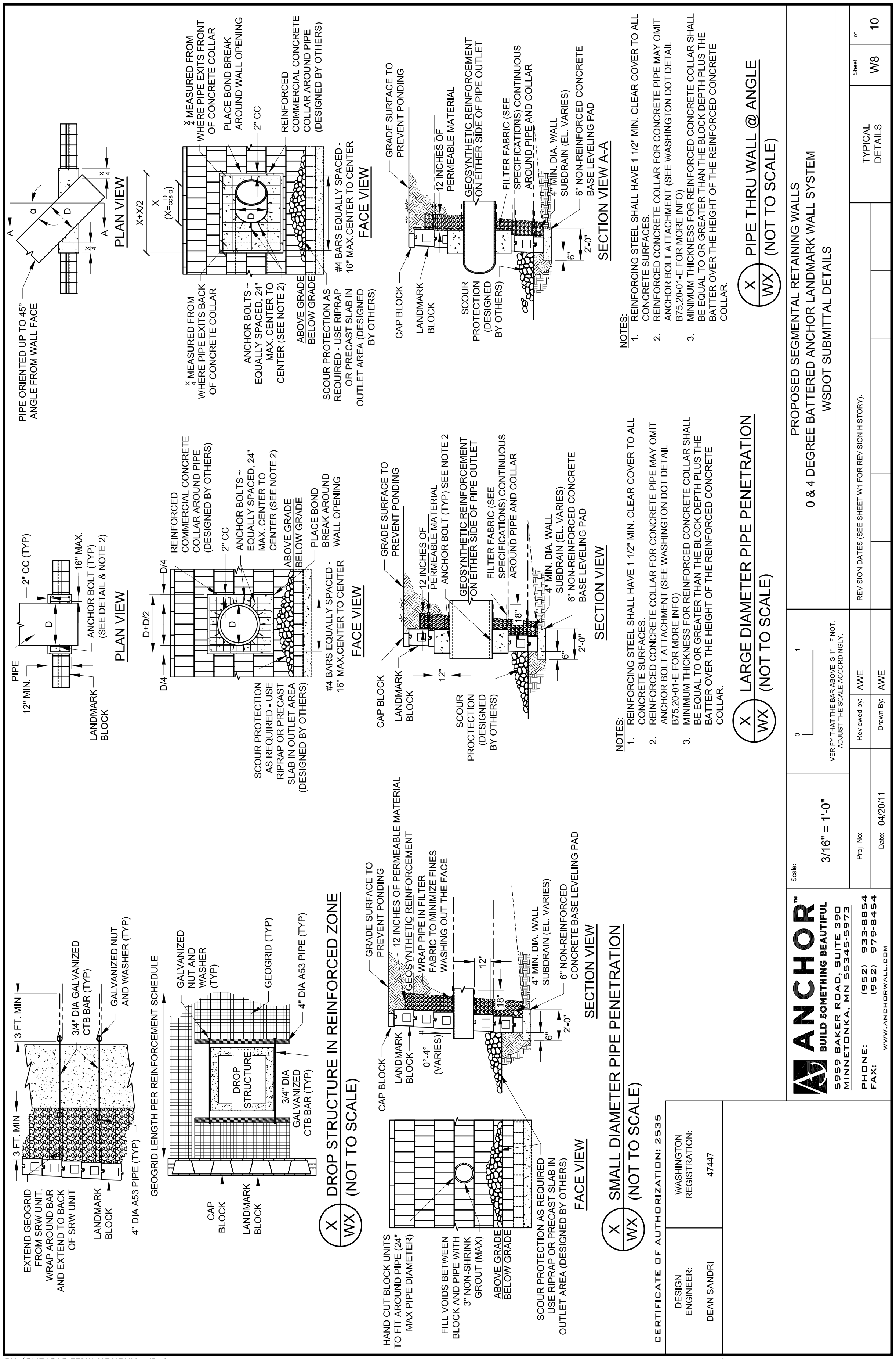

DROP STRUCTURE IN REINFORCED ZONE
(NOT TO SCALE)
EXTEND GEOGRID FROM SRW UNIT, WRAP AROUND BAR AND EXTEND TO BACK OF SRW UNIT
3 FT. MIN
3/4" DIA GALVANIZED CTB BAR (TYP)
GALVANIZED NUT AND WASHER (TYP)
LANDMARK BLOCK
4" DIA A53 PIPE (TYP)
GEOGRID LENGTH PER REINFORCEMENT SCHEDULE
GALVANIZED NUT AND WASHER (TYP)
GEOGRID (TYP)
DROP STRUCTURE
3/4" DIA GALVANIZED CTB BAR (TYP)
CAP BLOCK
SMALL DIAMETER PIPE PENETRATION
(NOT TO SCALE)
FACE VIEW
SECTION VIEW
HAND CUT BLOCK UNITS TO FIT AROUND PIPE (24" MAX PIPE DIAMETER)
FILL VOIDS BETWEEN BLOCK AND PIPE WITH 3" NON-SHRINK GROUT (MAX)
ABOVE GRADE
BELOW GRADE
SCOUR PROTECTION AS REQUIRED USE RIPRAP OR PRECAST SLAB IN OUTLET AREA (DESIGNED BY OTHERS)
GRADE SURFACE TO PREVENT PONDING
12 INCHES OF PERMEABLE MATERIAL
GEOSYNTHETIC REINFORCEMENT
WRAP PIPE IN FILTER FABRIC TO MINIMIZE FINES WASHING OUT THE FACE
4" MIN. DIA. WALL SUBDRAIN (EL. VARIES)
6" NON-REINFORCED CONCRETE BASE LEVELING PAD
0°-4° (VARIES)
LARGE DIAMETER PIPE PENETRATION
(NOT TO SCALE)
PLAN VIEW
PIPE
12" MIN.
2" CC (TYP)
16" MAX.
ANCHOR BOLT (TYP) (SEE DETAIL & NOTE 2)
REINFORCED COMMERCIAL CONCRETE COLLAR AROUND PIPE (DESIGNED BY OTHERS)
ANCHOR BOLTS ~ EQUALLY SPACED, 24" MAX. CENTER TO CENTER (SEE NOTE 2)
PLACE BOND BREAK AROUND WALL OPENING
#4 BARS EQUALLY SPACED - 16" MAX.CENTER TO CENTER
SCOUR PROTECTION AS REQUIRED - USE RIPRAP OR PRECAST SLAB IN OUTLET AREA (DESIGNED BY OTHERS)
ANCHOR BOLT (TYP) SEE NOTE 2
GEOSYNTHETIC REINFORCEMENT ON EITHER SIDE OF PIPE OUTLET
FILTER FABRIC (SEE SPECIFICATIONS) CONTINUOUS AROUND PIPE AND COLLAR
NOTES:
1. REINFORCING STEEL SHALL HAVE 1 1/2" MIN. CLEAR COVER TO ALL CONCRETE SURFACES.
2. REINFORCED CONCRETE COLLAR FOR CONCRETE PIPE MAY OMIT ANCHOR BOLT ATTACHMENT (SEE WASHINGTON DOT DETAIL B75.20-01-E FOR MORE INFO)
3. MINIMUM THICKNESS FOR REINFORCED CONCRETE COLLAR SHALL BE EQUAL TO OR GREATER THAN THE BLOCK DEPTH PLUS THE BATTER OVER THE HEIGHT OF THE REINFORCED CONCRETE COLLAR.
PIPE THRU WALL @ ANGLE
(NOT TO SCALE)
PIPE ORIENTED UP TO 45° ANGLE FROM WALL FACE
SECTION VIEW A-A
X/4 MEASURED FROM WHERE PIPE EXITS BACK OF CONCRETE COLLAR
X/4 MEASURED FROM WHERE PIPE EXITS FRONT OF CONCRETE COLLAR
CERTIFICATE OF AUTHORIZATION: 2535
DESIGN ENGINEER: DEAN SANDRI
WASHINGTON REGISTRATION: 47447
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PHONE: (952) 933-8854
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Scale: 3/16" = 1'-0"
VERIFY THAT THE BAR ABOVE IS 1". IF NOT, ADJUST THE SCALE ACCORDINGLY.
Reviewed by: AWE
Drawn By: AWE
Date: 04/20/11
REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):
PROPOSED SEGMENTAL RETAINING WALLS
0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM
WSDOT SUBMITTAL DETAILS
TYPICAL DETAILS
Sheet W8 of 10
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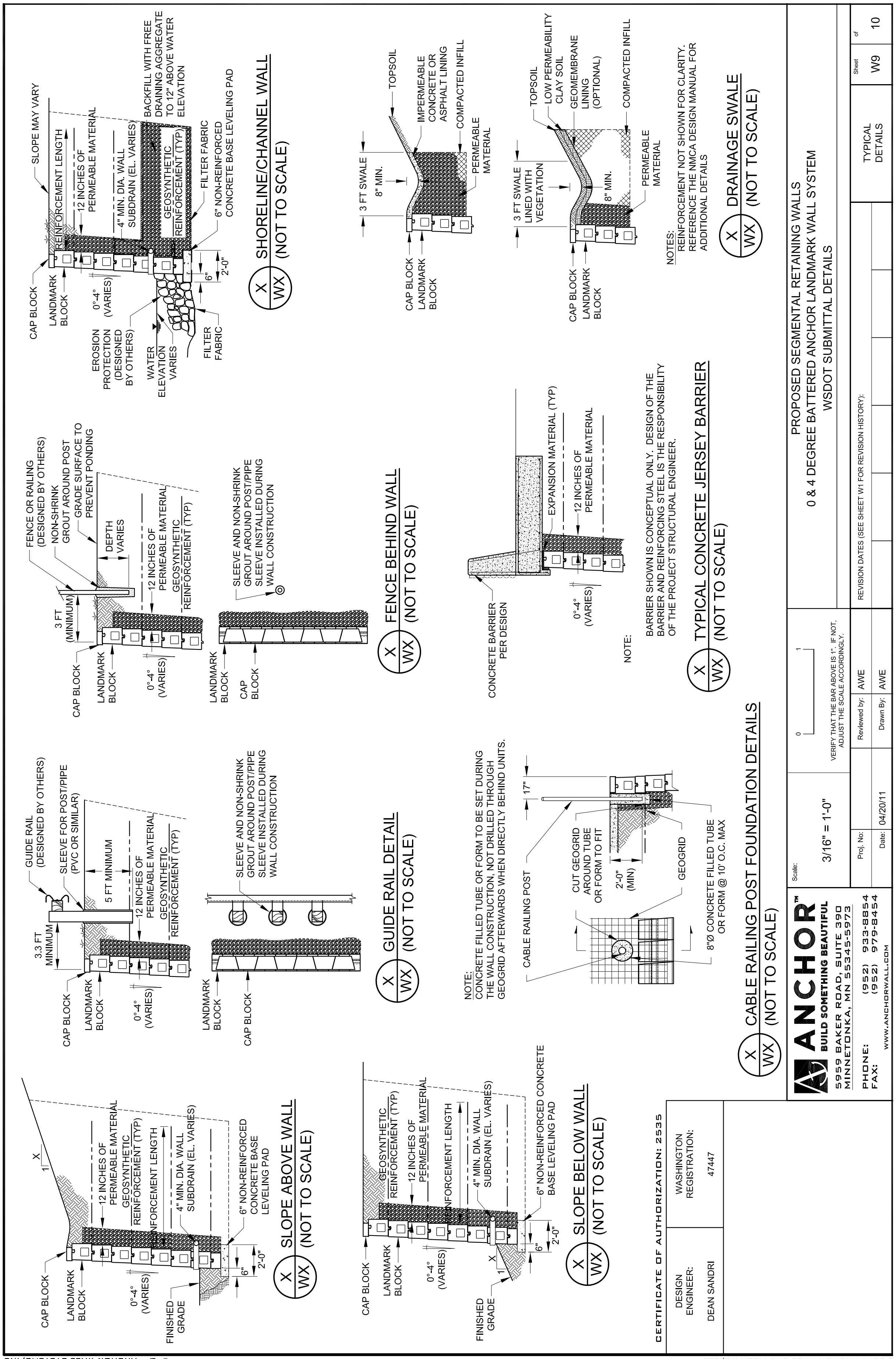
SLOPE ABOVE WALL (NOT TO SCALE)
CAP BLOCK
LANDMARK BLOCK
0°-4° (VARIES)
FINISHED GRADE
12 INCHES OF PERMEABLE MATERIAL
GEOSYNTHETIC REINFORCEMENT (TYP)
REINFORCEMENT LENGTH
4" MIN. DIA. WALL SUBDRAIN (EL. VARIES)
6" NON-REINFORCED CONCRETE BASE LEVELING PAD
SLOPE BELOW WALL (NOT TO SCALE)
GUIDE RAIL DETAIL (NOT TO SCALE)
GUIDE RAIL (DESIGNED BY OTHERS)
SLEEVE FOR POST/PIPE (PVC OR SIMILAR)
3.3 FT MINIMUM
5 FT MINIMUM
SLEEVE AND NON-SHRINK GROUT AROUND POST/PIPE SLEEVE INSTALLED DURING WALL CONSTRUCTION
CABLE RAILING POST FOUNDATION DETAILS (NOT TO SCALE)
NOTE: CONCRETE FILLED TUBE OR FORM TO BE SET DURING THE WALL CONSTRUCTION, NOT DRILLED THROUGH GEOGRID AFTERWARDS WHEN DIRECTLY BEHIND UNITS.
CABLE RAILING POST
CUT GEOGRID AROUND TUBE OR FORM TO FIT
2'-0" (MIN)
GEOGRID
8"Ø CONCRETE FILLED TUBE OR FORM @ 10' O.C. MAX
FENCE BEHIND WALL (NOT TO SCALE)
FENCE OR RAILING (DESIGNED BY OTHERS)
NON-SHRINK GROUT AROUND POST
GRADE SURFACE TO PREVENT PONDING
3 FT (MINIMUM)
DEPTH VARIES
TYPICAL CONCRETE JERSEY BARRIER (NOT TO SCALE)
CONCRETE BARRIER PER DESIGN
EXPANSION MATERIAL (TYP)
NOTE: BARRIER SHOWN IS CONCEPTUAL ONLY. DESIGN OF THE BARRIER AND REINFORCING STEEL IS THE RESPONSIBILITY OF THE PROJECT STRUCTURAL ENGINEER.
SHORELINE/CHANNEL WALL (NOT TO SCALE)
SLOPE MAY VARY
BACKFILL WITH FREE DRAINING AGGREGATE TO 12" ABOVE WATER ELEVATION
FILTER FABRIC
EROSION PROTECTION (DESIGNED BY OTHERS)
WATER ELEVATION VARIES
DRAINAGE SWALE (NOT TO SCALE)
3 FT SWALE
8" MIN.
TOPSOIL
IMPERMEABLE CONCRETE OR ASPHALT LINING
COMPACTED INFILL
PERMEABLE MATERIAL
3 FT SWALE LINED WITH VEGETATION
LOW PERMEABILITY CLAY SOIL
GEOMEMBRANE LINING (OPTIONAL)
NOTES: REINFORCEMENT NOT SHOWN FOR CLARITY. REFERENCE THE NMCA DESIGN MANUAL FOR ADDITIONAL DETAILS
CERTIFICATE OF AUTHORIZATION: 2535
DESIGN ENGINEER: DEAN SANDRI
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Scale: 3/16" = 1'-0"
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Proj. No:
Date: 04/20/11
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Drawn By: AWE
REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):
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0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM
WSDOT SUBMITTAL DETAILS
TYPICAL DETAILS
Sheet W9 of 10
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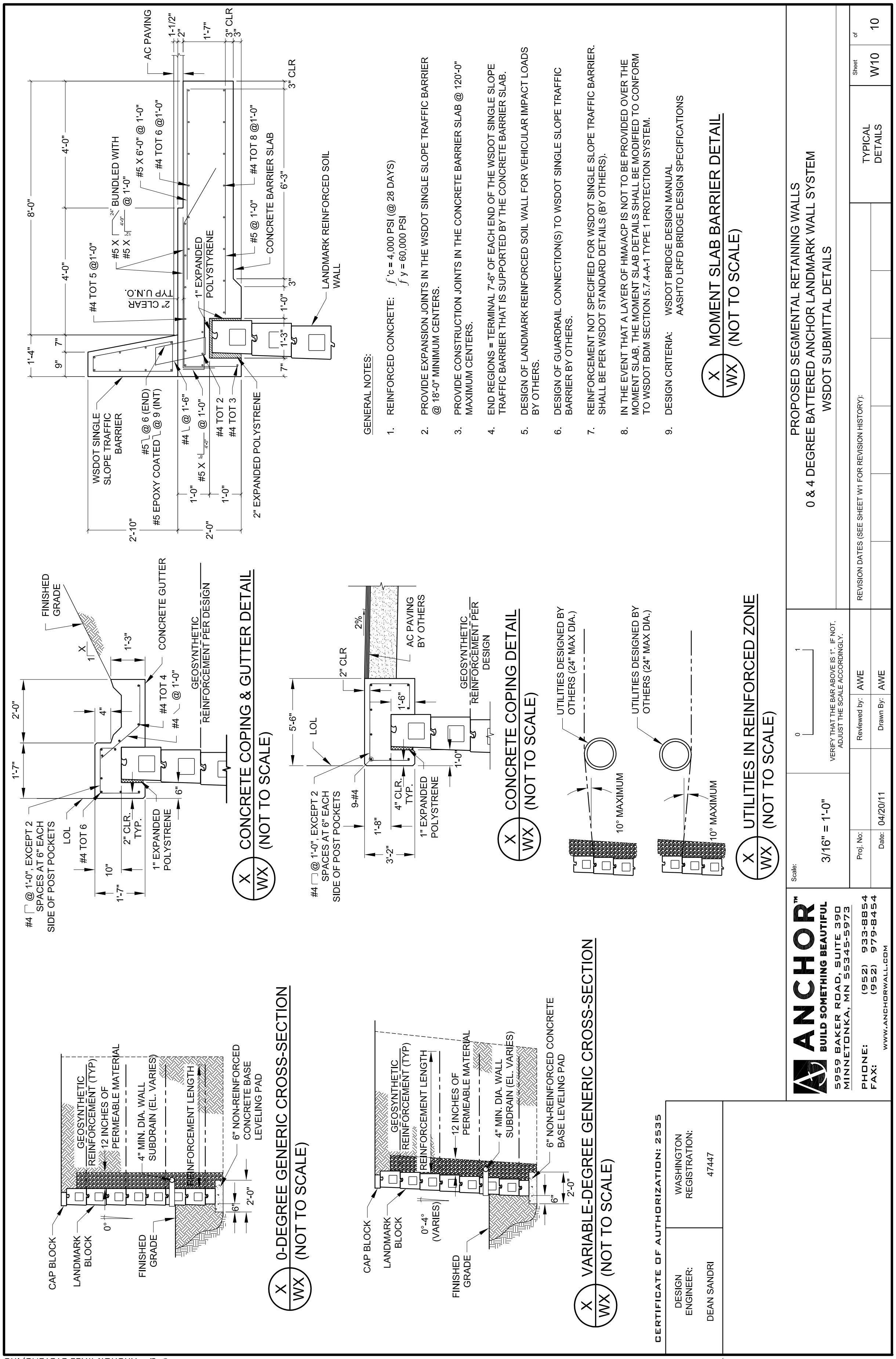
0-DEGREE GENERIC CROSS-SECTION
(NOT TO SCALE)
CAP BLOCK
LANDMARK BLOCK
FINISHED GRADE
GEOSYNTHETIC REINFORCEMENT (TYP)
12 INCHES OF PERMEABLE MATERIAL
4" MIN. DIA. WALL SUBDRAIN (EL. VARIES)
REINFORCEMENT LENGTH
6" NON-REINFORCED CONCRETE BASE LEVELING PAD
VARIABLE-DEGREE GENERIC CROSS-SECTION
(NOT TO SCALE)
0°-4° (VARIES)
CONCRETE COPING & GUTTER DETAIL
(NOT TO SCALE)
#4 @ 1'-0", EXCEPT 2 SPACES AT 6" EACH SIDE OF POST POCKETS
LOL
#4 TOT 6
2" CLR. TYP.
1" EXPANDED POLYSTRENE
CONCRETE GUTTER
FINISHED GRADE
#4 TOT 4
#4 @ 1'-0"
GEOSYNTHETIC REINFORCEMENT PER DESIGN
CONCRETE COPING DETAIL
(NOT TO SCALE)
9-#4
4" CLR. TYP.
AC PAVING BY OTHERS
2%
UTILITIES IN REINFORCED ZONE
(NOT TO SCALE)
UTILITIES DESIGNED BY OTHERS (24" MAX DIA.)
10° MAXIMUM
MOMENT SLAB BARRIER DETAIL
(NOT TO SCALE)
WSDOT SINGLE SLOPE TRAFFIC BARRIER
#5 EPOXY COATED @ 9 (INT)
#5 @ 6 (END)
2" EXPANDED POLYSTRENE
#4 TOT 2
#4 TOT 3
#4 TOT 5 @1'-0"
2" CLEAR TYP U.N.O.
1" EXPANDED POLYSTYRENE
BUNDLED WITH @ 1'-0"
#5 X 6'-0" @ 1'-0"
#4 TOT 6 @1'-0"
#4 TOT 8 @1'-0"
#5 @ 1'-0"
CONCRETE BARRIER SLAB
LANDMARK REINFORCED SOIL WALL
AC PAVING
GENERAL NOTES:
1. REINFORCED CONCRETE: f'c = 4,000 PSI (@ 28 DAYS) fy = 60,000 PSI
2. PROVIDE EXPANSION JOINTS IN THE WSDOT SINGLE SLOPE TRAFFIC BARRIER @ 18'-0" MINIMUM CENTERS.
3. PROVIDE CONSTRUCTION JOINTS IN THE CONCRETE BARRIER SLAB @ 120'-0" MAXIMUM CENTERS.
4. END REGIONS = TERMINAL 7'-6" OF EACH END OF THE WSDOT SINGLE SLOPE TRAFFIC BARRIER THAT IS SUPPORTED BY THE CONCRETE BARRIER SLAB.
5. DESIGN OF LANDMARK REINFORCED SOIL WALL FOR VEHICULAR IMPACT LOADS BY OTHERS.
6. DESIGN OF GUARDRAIL CONNECTION(S) TO WSDOT SINGLE SLOPE TRAFFIC BARRIER BY OTHERS.
7. REINFORCEMENT NOT SPECIFIED FOR WSDOT SINGLE SLOPE TRAFFIC BARRIER. SHALL BE PER WSDOT STANDARD DETAILS (BY OTHERS).
8. IN THE EVENT THAT A LAYER OF HMA/ACP IS NOT TO BE PROVIDED OVER THE MOMENT SLAB, THE MOMENT SLAB DETAILS SHALL BE MODIFIED TO CONFORM TO WSDOT BDM SECTION 5.7.4-A-1 TYPE 1 PROTECTION SYSTEM.
9. DESIGN CRITERIA: WSDOT BRIDGE DESIGN MANUAL AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS
CERTIFICATE OF AUTHORIZATION: 2535
DESIGN ENGINEER: DEAN SANDRI
WASHINGTON REGISTRATION: 47447
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REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):
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0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM
WSDOT SUBMITTAL DETAILS
TYPICAL DETAILS
Sheet W10 of 10
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Appendix 15-P Preapproved Wall Appendix: Specific Requirements and Details for Allan Block Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply:

Facing Blocks –Blocks acceptable for use with this wall system include, AB Classic, and AB Vertical. These blocks are for a facing batter of 1°, 3°, and 6° degrees. Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2 ft.

Soil Reinforcement – Only geosynthetic reinforcement listed in the WSDOT QPL and which has been evaluated for connection strength with the Allan Block wall system shall be used. For walls with a face batter of 1 degrees or more (i.e., facing blocks, AB Classic, and AB Vertical), this includes the following specific products that are approved for use with this wall system:

Miragrid 3XT	Stratagrid SG200
Miragrid 5XT	Stratagrid SG350

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications.

Reinforcement/Facing Block Connection Requirements – Connection testing was done for the range of blocks and geosynthetic reinforcements preapproved for this wall system. The connection between Allan Block facing units and the geosynthetic reinforcement is essentially a frictional connection. That being the case, the connection resistance is strongly dependent on the normal force between blocks and in the gravel in-fill inside the blocks and less dependent on the roll or lot specific tensile strength, T_{lot}, as well as the long-term effect of creep on the connection strength. However, neither T_{lot} for each test (only T_{MARV} values for the tested geogrids were provided), nor connection creep tests, were provided. Since no connection creep tests were provided, as required in the AASHTO LRFD Bridge Design manual, RF_{CR} must be used to obtain T_{ac}. Therefore, the long-term connection strength (i.e., T_{ac}) equation provided in the AASHTO LRFD Bridge Design Manual will need to be simplified to the equation shown below:

$$T_{ac} = \frac{T_{ultconn}}{RF_{CR} x RF_D} \qquad (15\text{-P-}1)$$

where,

$T_{ultconn}$ is the ultimate connection strength from the product specific connection strength tests, the results of which are provided in Table 15-S-1,

RF_{CR} = creep reduction factor for the geosynthetic, and

RF_D = the durability reduction factor for the geosynthetic.

RF_{CR} and RF_D shall be as provided in the WSDOT QPL, Appendix D.

Table 15-P-1 Approved connection strength design values for Allan Block walls

Applicable Facing Blocks	Geogrid Product	Normal Load, N (lbs/ft)	$T_{ultconn}$ (lbs/ft) Facing Batter = 1° or 3°	$T_{ultconn}$ (lbs/ft) Facing Batter = 6°
AB Classic and AB Vertical	Miragrid 3XT	N ≤ 2474 N > 2474	1239 + N*Tan 26° 2,450	1193 + N*Tan 29° 2,560
	Miragrid 5XT	N ≤ 3713 N > 3713	1320 + N*Tan 27° 3,210	1287 + N*Tan 29° 3,350
	Stratagrid SG200	N ≤ 2474 N > 2474	890 + N*Tan 34° 2,560	1383 + N*Tan 18° 2,190
	Stratagrid SG350	N ≤ 3713 N > 3713	1079 + N*Tan 19° 2,360	1257 + N*Tan 12° 2,050

N = normal load at reinforcement layer at facing, in lbs/ft of width parallel to face.

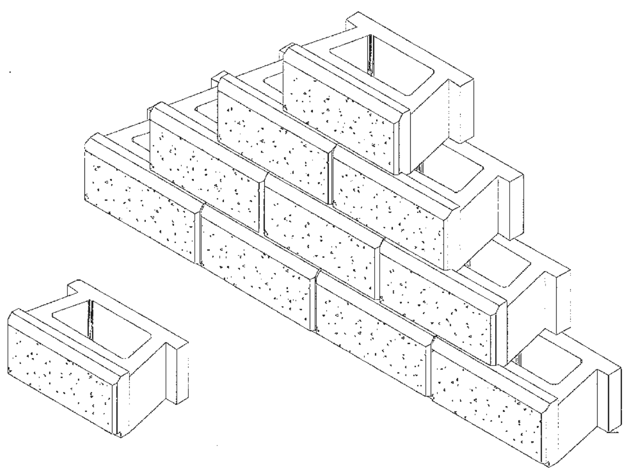
The connection strengths provided in the table assume that crushed rock is used to fill the interior of the blocks. Allan Block also provides the option to grout the interior of the blocks, creating a full mechanical connection. This connection approach is not preapproved, as connection strength data for this situation was not provided, and furthermore, the elevated pH that could be caused by the grout could accelerate chemical degradation. This has not been evaluated.

Approved details for the Allan Block wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet.
- In plan sheet 7 of 12, the guard rail detail, the guard rail post shall either be installed through precut holes in the geogrid layers that must be penetrated, or the geogrid layers shall be cut in a manner that prevents ripping or tearing of the geogrid.
- In plan sheet 5 of 12, regarding the geogrid at wall corner detail, cords in the wall facing alignment to form an angle point or a radius shall be no shorter than the width of the roll to insure good contact between the connectors and the geogrid cross-bar throughout the width of the geogrid. Alternatively, the geogrid roll could be cut longitudinally in half to allow a tighter radius, if necessary.
- It is noted in ASTM C1372 that a dimensional tolerance for the height of the block of ⅛ inch is allowed, but that WSDOT GDM Section 15-5.3.8 recommends a tighter dimensional tolerance of 1⁄16 inch. Based on WSDOT experience, for walls greater than 25 ft in height, some cracking of facing blocks due to differential vertical stresses tends to occur in the bottom portion of the wall. Therefore, blocks placed at depths below the wall top of 25 ft or more should be cast to a vertical dimensional tolerance of 1⁄16 inch to reduce the risk of significant cracking of facing blocks.

CONSTRUCTION DRAWINGS PREPARED FOR WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

ALLAN BLOCK DETAIL AND CONSTRUCTION NOTES

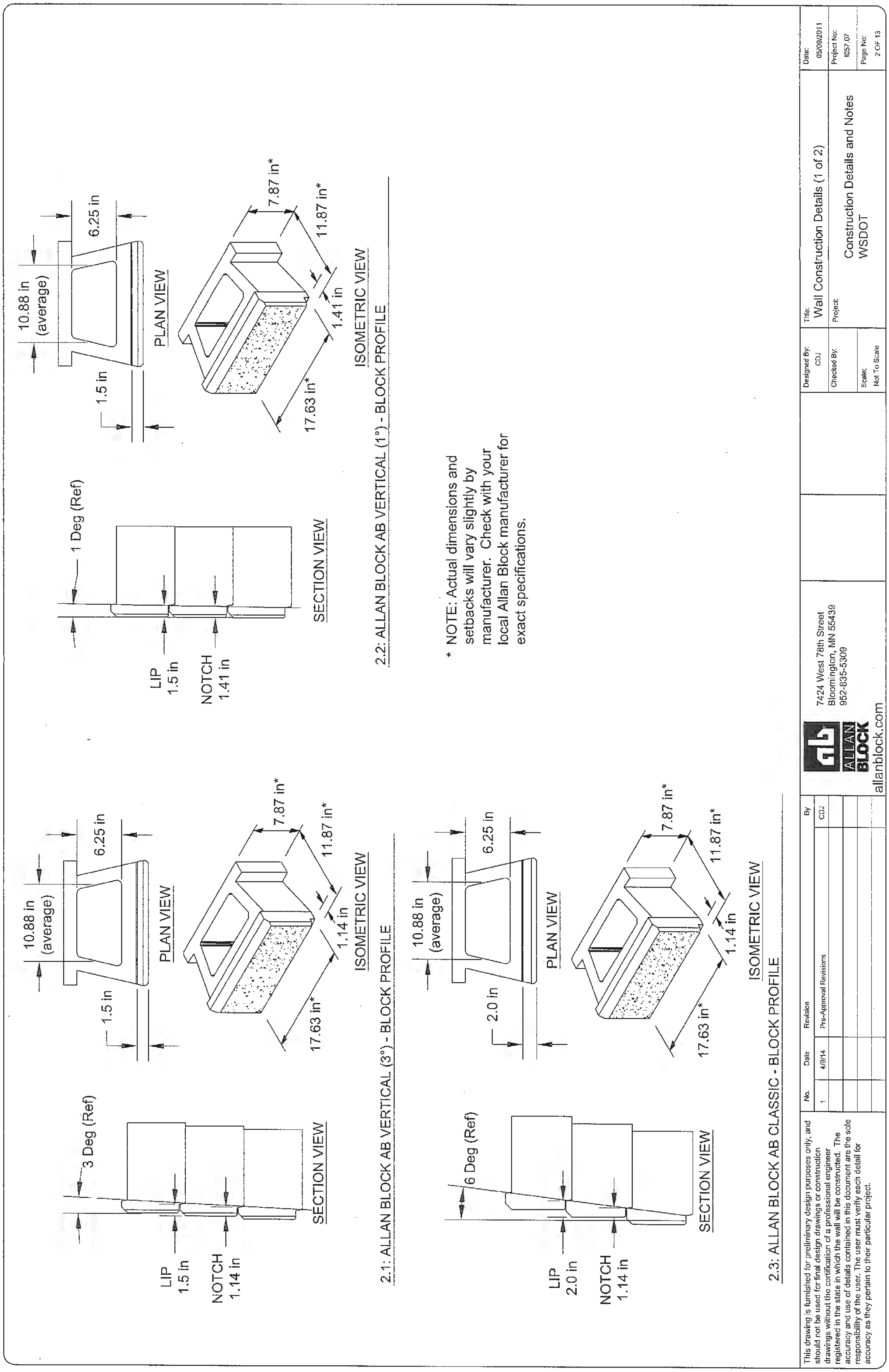


SHEET INDEX

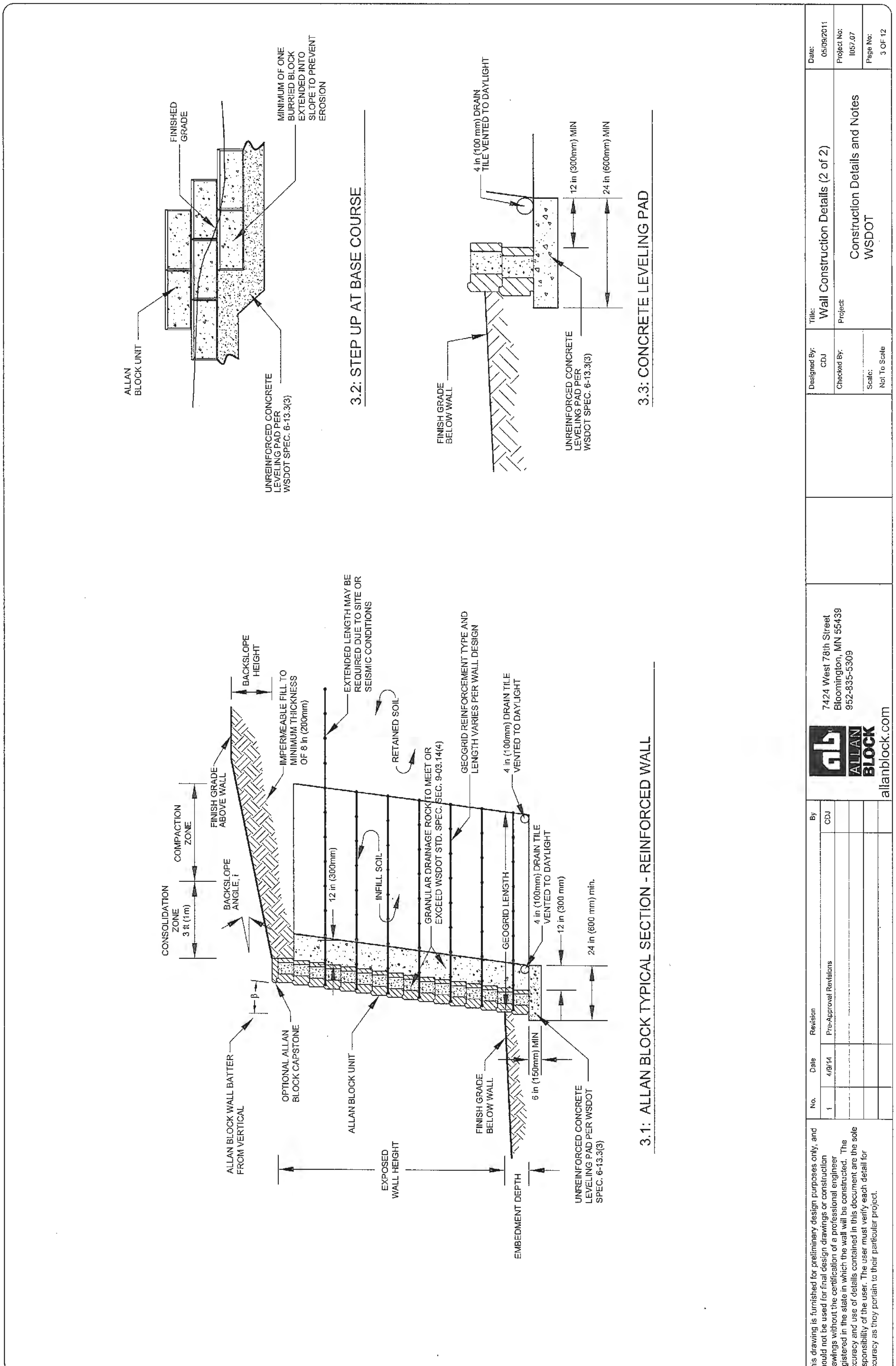
No.	Date	Revision	By
1	4/9/14	Pre-Approval Revisions	CDJ

ALLAN BLOCK
7424 West 78th Street
Bloomington, MN 55439
952-835-5309
allanblock.com

Designed By: CDJ
Checked By:
Scale: Not To Scale
Title: Title Sheet
Project: Construction Details and Notes WSDOT
Date: 05/09/2011
Project No: 1057.07
Page No: 1 OF 12



2.1: ALLAN BLOCK AB VERTICAL (3°) - BLOCK PROFILE
SECTION VIEW
3 Deg (Ref)
LIP 1.5 in
NOTCH 1.14 in
PLAN VIEW
10.88 in (average)
1.5 in
6.25 in
ISOMETRIC VIEW
17.63 in*
7.87 in*
11.87 in*
1.14 in
2.2: ALLAN BLOCK AB VERTICAL (1°) - BLOCK PROFILE
SECTION VIEW
1 Deg (Ref)
LIP 1.5 in
NOTCH 1.41 in
PLAN VIEW
10.88 in (average)
1.5 in
6.25 in
ISOMETRIC VIEW
17.63 in*
7.87 in*
11.87 in*
1.41 in
2.3: ALLAN BLOCK AB CLASSIC - BLOCK PROFILE
SECTION VIEW
6 Deg (Ref)
LIP 2.0 in
NOTCH 1.14 in
PLAN VIEW
10.88 in (average)
2.0 in
6.25 in
ISOMETRIC VIEW
17.63 in*
7.87 in*
11.87 in*
1.14 in
* NOTE: Actual dimensions and setbacks will vary slightly by manufacturer. Check with your local Allan Block manufacturer for exact specifications.
This drawing is furnished for preliminary design purposes only, and should not be used for final design drawings or construction drawings without the certification of a professional engineer registered in the state in which the wall will be constructed. The accuracy and use of details contained in this document are the sole responsibility of the user. The user must verify each detail for accuracy as they pertain to their particular project.
No. Date Revision By
1 4/9/14 Pre-Approval Revisions CDJ
ALLAN BLOCK
allanblock.com
7424 West 78th Street
Bloomington, MN 55439
952-835-5309
Designed By: CDJ
Checked By:
Scale: Not To Scale
Title: Wall Construction Details (1 of 2)
Project: Construction Details and Notes WSDOT
Date: 05/09/2011
Project No: 1057.07
Page No: 2 OF 13


ALLAN BLOCK UNIT
FINISHED GRADE
MINIMUM OF ONE BURRIED BLOCK EXTENDED INTO SLOPE TO PREVENT EROSION
UNREINFORCED CONCRETE LEVELING PAD PER WSDOT SPEC. 6-13.3(3)
3.2: STEP UP AT BASE COURSE
4 in (100 mm) DRAIN TILE VENTED TO DAYLIGHT
12 in (300mm) MIN
24 in (600mm) MIN
FINISH GRADE BELOW WALL
UNREINFORCED CONCRETE LEVELING PAD PER WSDOT SPEC. 6-13.3(3)
3.3: CONCRETE LEVELING PAD
CONSOLIDATION ZONE 3 ft (1m)
COMPACTION ZONE
BACKSLOPE ANGLE, i
FINISH GRADE ABOVE WALL
BACKSLOPE HEIGHT
IMPERMEABLE FILL TO MINIMUM THICKNESS OF 8 in (200mm)
EXTENDED LENGTH MAY BE REQUIRED DUE TO SITE OR SEISMIC CONDITIONS
RETAINED SOIL
12 in (300mm)
INFILL SOIL
GRANULAR DRAINAGE ROCK TO MEET OR EXCEED WSDOT STD. SPEC. SEC. 9-03.14(4)
GEOGRID REINFORCEMENT TYPE AND LENGTH VARIES PER WALL DESIGN
GEOGRID LENGTH
4 in (100mm) DRAIN TILE VENTED TO DAYLIGHT
4 in (100mm) DRAIN TILE VENTED TO DAYLIGHT
12 in (300 mm)
24 in (600 mm) min.
ALLAN BLOCK WALL BATTER FROM VERTICAL
β
OPTIONAL ALLAN BLOCK CAPSTONE
ALLAN BLOCK UNIT
FINISH GRADE BELOW WALL
6 in (150mm) MIN
UNREINFORCED CONCRETE LEVELING PAD PER WSDOT SPEC. 6-13.3(3)
EXPOSED WALL HEIGHT
EMBEDMENT DEPTH
3.1: ALLAN BLOCK TYPICAL SECTION - REINFORCED WALL
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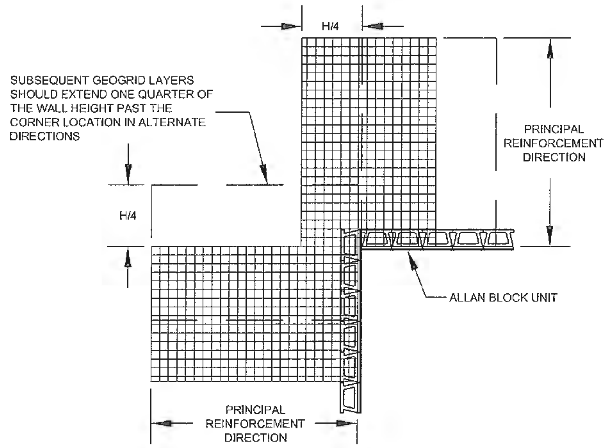
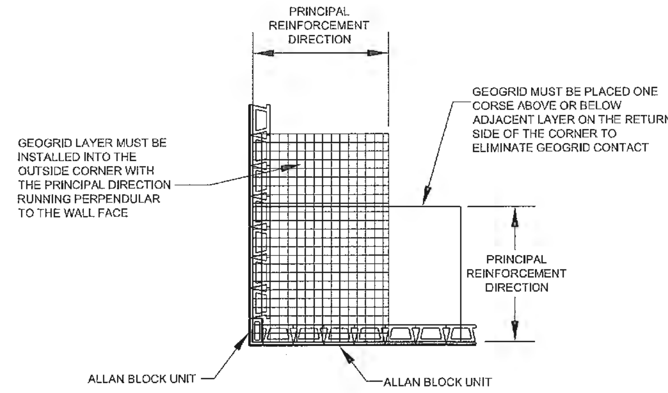
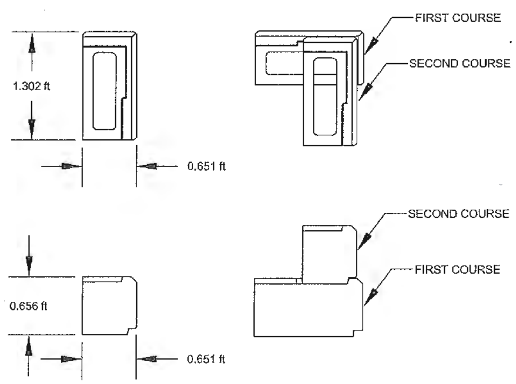
Page No: 3 OF 12

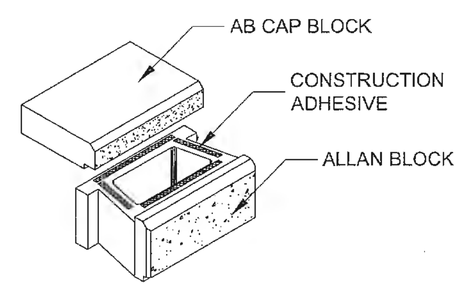


4.1: INSIDE CORNER GEOGRID OVERLAP



4.2: OUTSIDE CORNER GEOGRID OVERLAP





CONSTRUCTION NOTES:
1. ALTERNATE AB CORNER UNITS FOR PROPER RUNNING BOND.
2. FILL CORES WITH CLEAR STONE SIMILAR TO THE CORES OF THE WALL UNITS.

4.3: OUTSIDE CORNER DETAIL

4.4: ALLAN BLOCK CAP INSTALLATION

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1	4/9/14	Pre-Approval Revisions	CDJ

ALLAN BLOCK
allanblock.com
7424 West 78th Street
Bloomington, MN 55439
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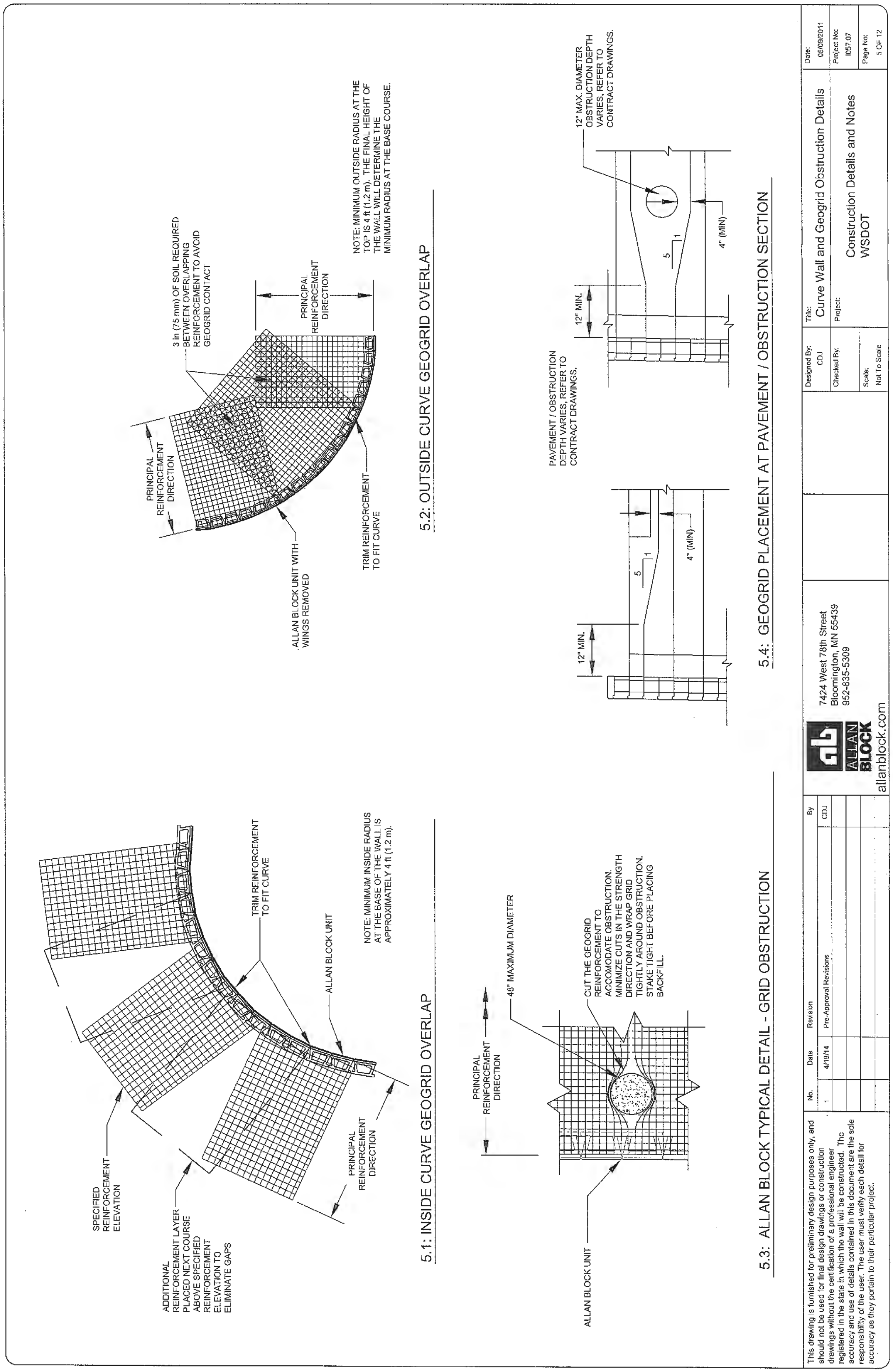
Designed By: CDJ
Checked By:
Scale: Not To Scale
Title: Corner and Cap Details
Project: Construction Details and Notes WSDOT
Date: 05/09/2011
Project No: 1057.07
Page No: 4 OF 12


5.1: INSIDE CURVE GEOGRID OVERLAP
SPECIFIED REINFORCEMENT ELEVATION
ADDITIONAL REINFORCEMENT LAYER PLACED NEXT COURSE ABOVE SPECIFIED REINFORCEMENT ELEVATION TO ELIMINATE GAPS
PRINCIPAL REINFORCEMENT DIRECTION
TRIM REINFORCEMENT TO FIT CURVE
ALLAN BLOCK UNIT
NOTE: MINIMUM INSIDE RADIUS AT THE BASE OF THE WALL IS APPROXIMATELY 4 ft (1.2 m).
5.2: OUTSIDE CURVE GEOGRID OVERLAP
PRINCIPAL REINFORCEMENT DIRECTION
3 in (75 mm) OF SOIL REQUIRED BETWEEN OVERLAPPING REINFORCEMENT TO AVOID GEOGRID CONTACT
ALLAN BLOCK UNIT WITH WINGS REMOVED
TRIM REINFORCEMENT TO FIT CURVE
NOTE: MINIMUM OUTSIDE RADIUS AT THE TOP IS 4 ft (1.2 m). THE FINAL HEIGHT OF THE WALL WILL DETERMINE THE MINIMUM RADIUS AT THE BASE COURSE.
5.3: ALLAN BLOCK TYPICAL DETAIL - GRID OBSTRUCTION
PRINCIPAL REINFORCEMENT DIRECTION
48" MAXIMUM DIAMETER
CUT THE GEOGRID REINFORCEMENT TO ACCOMODATE OBSTRUCTION. MINIMIZE CUTS IN THE STRENGTH DIRECTION AND WRAP GRID TIGHTLY AROUND OBSTRUCTION. STAKE TIGHT BEFORE PLACING BACKFILL.
ALLAN BLOCK UNIT
5.4: GEOGRID PLACEMENT AT PAVEMENT / OBSTRUCTION SECTION
12" MIN.
5
1
4" (MIN)
PAVEMENT / OBSTRUCTION DEPTH VARIES, REFER TO CONTRACT DRAWINGS.
12" MAX. DIAMETER OBSTRUCTION DEPTH VARIES, REFER TO CONTRACT DRAWINGS.
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1 4/19/14 Pre-Approval Revisions CDJ
ALLAN BLOCK
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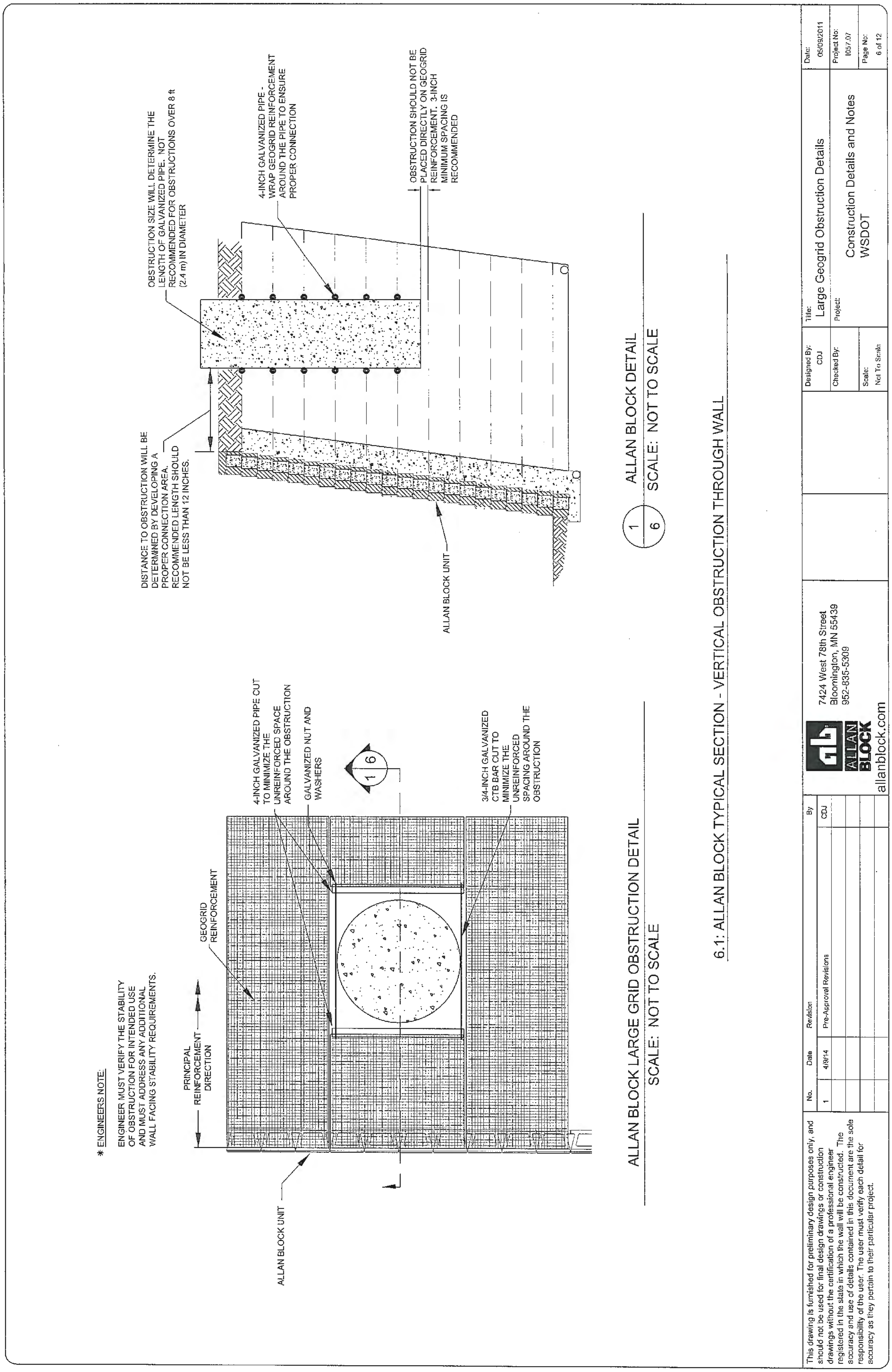
7424 West 78th Street
Bloomington, MN 55439
952-835-5309
Designed By: CDJ
Checked By:
Scale: Not To Scale
Title: Curve Wall and Geogrid Obstruction Details
Project: Construction Details and Notes WSDOT
Date: 05/09/2011
Project No: 1057.07
Page No: 5 OF 12



OBSTRUCTION SIZE WILL DETERMINE THE LENGTH OF GALVANIZED PIPE. NOT RECOMMENDED FOR OBSTRUCTIONS OVER 8 ft (2.4 m) IN DIAMETER
4-INCH GALVANIZED PIPE - WRAP GEOGRID REINFORCEMENT AROUND THE PIPE TO ENSURE PROPER CONNECTION
OBSTRUCTION SHOULD NOT BE PLACED DIRECTLY ON GEOGRID REINFORCEMENT. 3-INCH MINIMUM SPACING IS RECOMMENDED
DISTANCE TO OBSTRUCTION WILL BE DETERMINED BY DEVELOPING A PROPER CONNECTION AREA. RECOMMENDED LENGTH SHOULD NOT BE LESS THAN 12 INCHES.
ALLAN BLOCK UNIT
1
6
ALLAN BLOCK DETAIL
SCALE: NOT TO SCALE
* ENGINEERS NOTE:
ENGINEER MUST VERIFY THE STABILITY OF OBSTRUCTION FOR INTENDED USE AND MUST ADDRESS ANY ADDITIONAL WALL FACING STABILITY REQUIREMENTS.
PRINCIPAL REINFORCEMENT DIRECTION
GEOGRID REINFORCEMENT
4-INCH GALVANIZED PIPE CUT TO MINIMIZE THE UNREINFORCED SPACE AROUND THE OBSTRUCTION
GALVANIZED NUT AND WASHERS
3/4-INCH GALVANIZED CTB BAR CUT TO MINIMIZE THE UNREINFORCED SPACING AROUND THE OBSTRUCTION
ALLAN BLOCK UNIT
ALLAN BLOCK LARGE GRID OBSTRUCTION DETAIL
SCALE: NOT TO SCALE
6.1: ALLAN BLOCK TYPICAL SECTION - VERTICAL OBSTRUCTION THROUGH WALL
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ALLAN BLOCK
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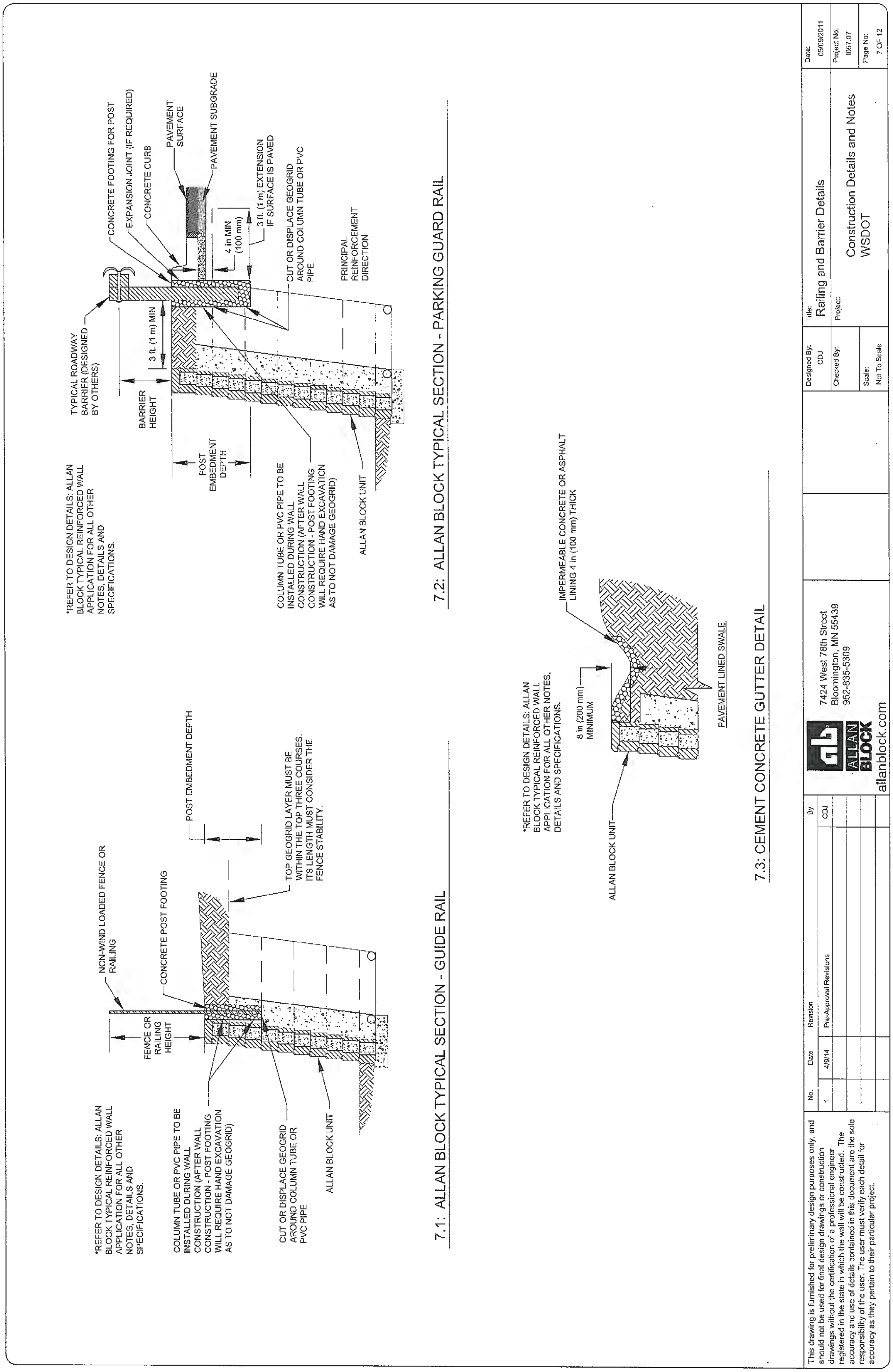
7424 West 78th Street
Bloomington, MN 55439
952-835-5309
Designed By: CDJ
Checked By:
Scale: Not To Scale
Title: Large Geogrid Obstruction Details
Project: Construction Details and Notes WSDOT
Date: 05/09/2011
Project No: 1057.07
Page No: 6 of 12


*REFER TO DESIGN DETAILS: ALLAN BLOCK TYPICAL REINFORCED WALL APPLICATION FOR ALL OTHER NOTES, DETAILS AND SPECIFICATIONS.
NON-WIND LOADED FENCE OR RAILING
CONCRETE POST FOOTING
FENCE OR RAILING HEIGHT
POST EMBEDMENT DEPTH
TOP GEOGRID LAYER MUST BE WITHIN THE TOP THREE COURSES. ITS LENGTH MUST CONSIDER THE FENCE STABILITY.
COLUMN TUBE OR PVC PIPE TO BE INSTALLED DURING WALL CONSTRUCTION (AFTER WALL CONSTRUCTION - POST FOOTING WILL REQUIRE HAND EXCAVATION AS TO NOT DAMAGE GEOGRID)
CUT OR DISPLACE GEOGRID AROUND COLUMN TUBE OR PVC PIPE
ALLAN BLOCK UNIT
7.1: ALLAN BLOCK TYPICAL SECTION - GUIDE RAIL
*REFER TO DESIGN DETAILS: ALLAN BLOCK TYPICAL REINFORCED WALL APPLICATION FOR ALL OTHER NOTES, DETAILS AND SPECIFICATIONS.
TYPICAL ROADWAY BARRIER (DESIGNED BY OTHERS)
BARRIER HEIGHT
3 ft. (1 m) MIN
POST EMBEDMENT DEPTH
CONCRETE FOOTING FOR POST
EXPANSION JOINT (IF REQUIRED)
CONCRETE CURB
PAVEMENT SURFACE
PAVEMENT SUBGRADE
4 in MIN (100 mm)
3 ft. (1 m) EXTENSION IF SURFACE IS PAVED
CUT OR DISPLACE GEOGRID AROUND COLUMN TUBE OR PVC PIPE
PRINCIPAL REINFORCEMENT DIRECTION
COLUMN TUBE OR PVC PIPE TO BE INSTALLED DURING WALL CONSTRUCTION (AFTER WALL CONSTRUCTION - POST FOOTING WILL REQUIRE HAND EXCAVATION AS TO NOT DAMAGE GEOGRID)
ALLAN BLOCK UNIT
7.2: ALLAN BLOCK TYPICAL SECTION - PARKING GUARD RAIL
*REFER TO DESIGN DETAILS: ALLAN BLOCK TYPICAL REINFORCED WALL APPLICATION FOR ALL OTHER NOTES, DETAILS AND SPECIFICATIONS.
ALLAN BLOCK UNIT
8 in (200 mm) MINIMUM
IMPERMEABLE CONCRETE OR ASPHALT LINING 4 in (100 mm) THICK
PAVEMENT LINED SWALE
7.3: CEMENT CONCRETE GUTTER DETAIL
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No. Date Revision By
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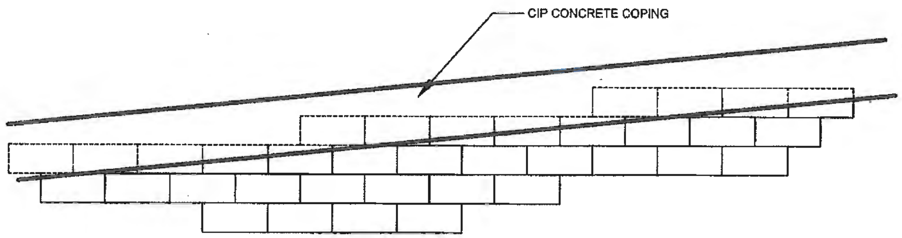
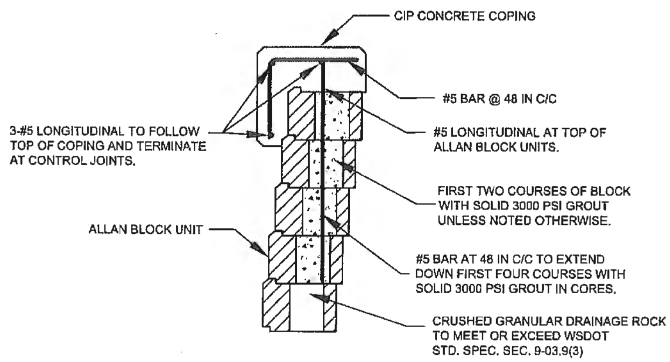
ALLAN BLOCK
7424 West 78th Street
Bloomington, MN 55439
952-835-5309
allanblock.com
Designed By: CDJ
Checked By:
Scale: Not To Scale
Title: Railing and Barrier Details
Project: Construction Details and Notes WSDOT
Date: 05/09/2011
Project No: 1057.07
Page No: 7 OF 12



ELEVATION VIEW

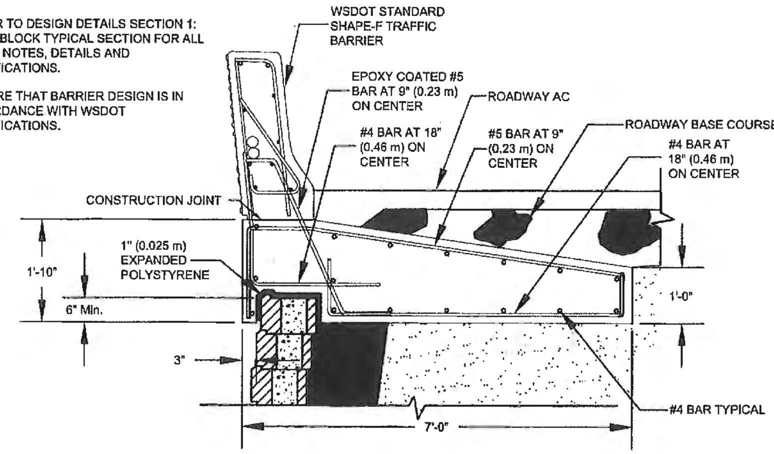


8.1: ALLAN BLOCK TYPICAL SECTION - CIP CONCRETE COPING

WSDOT Conditional Pre-approval:
The details showing grout in the interior cells of the blocks are not pre-approved when the grout is in direct contact with the geogrid reinforcement.

*REFER TO DESIGN DETAILS SECTION 1: ALLAN BLOCK TYPICAL SECTION FOR ALL OTHER NOTES, DETAILS AND SPECIFICATIONS.

*ENSURE THAT BARRIER DESIGN IS IN ACCORDANCE WITH WSDOT SPECIFICATIONS.



CONCRETE CONSTRUCTION NOTES:
1. POSITIVE BOND BREAKER AND EXPANSION JOINT MATERIAL MUST BE PROVIDED BETWEEN CAST IN PLACE SLAB AND ALLAN BLOCK UNITS
2. CONCRETE MINIMUM COMPRESSIVE STRENGTH f'c = 4000 psi (28 MPa) REBAR MINIMUM TENSILE STRENGTH fy = 65 ksi (45 MPa)
3. CAULK EXPANSION JOINT 4" (0.10 m) ABOVE FINISHED GRADE
4. EXPANSION JOUNTS ARE AT 30' (9 m) ON CENTER MAXIMUM AND 20' (9 m) ON CENTER MINIMUM
5. CONTROL JOINTS ARE AT 10' (3 m) ON CENTER

8.2: ALLAN BLOCK TYPICAL SECTION - TRAFFIC BARRIER

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No.	Date	Revision	By
1	4/9/14	Pre-Approval Revisions	CDJ
2	9/15/14	References to WSDOT Std. Spec. for Crushed Rock	CDJ

ALLAN BLOCK
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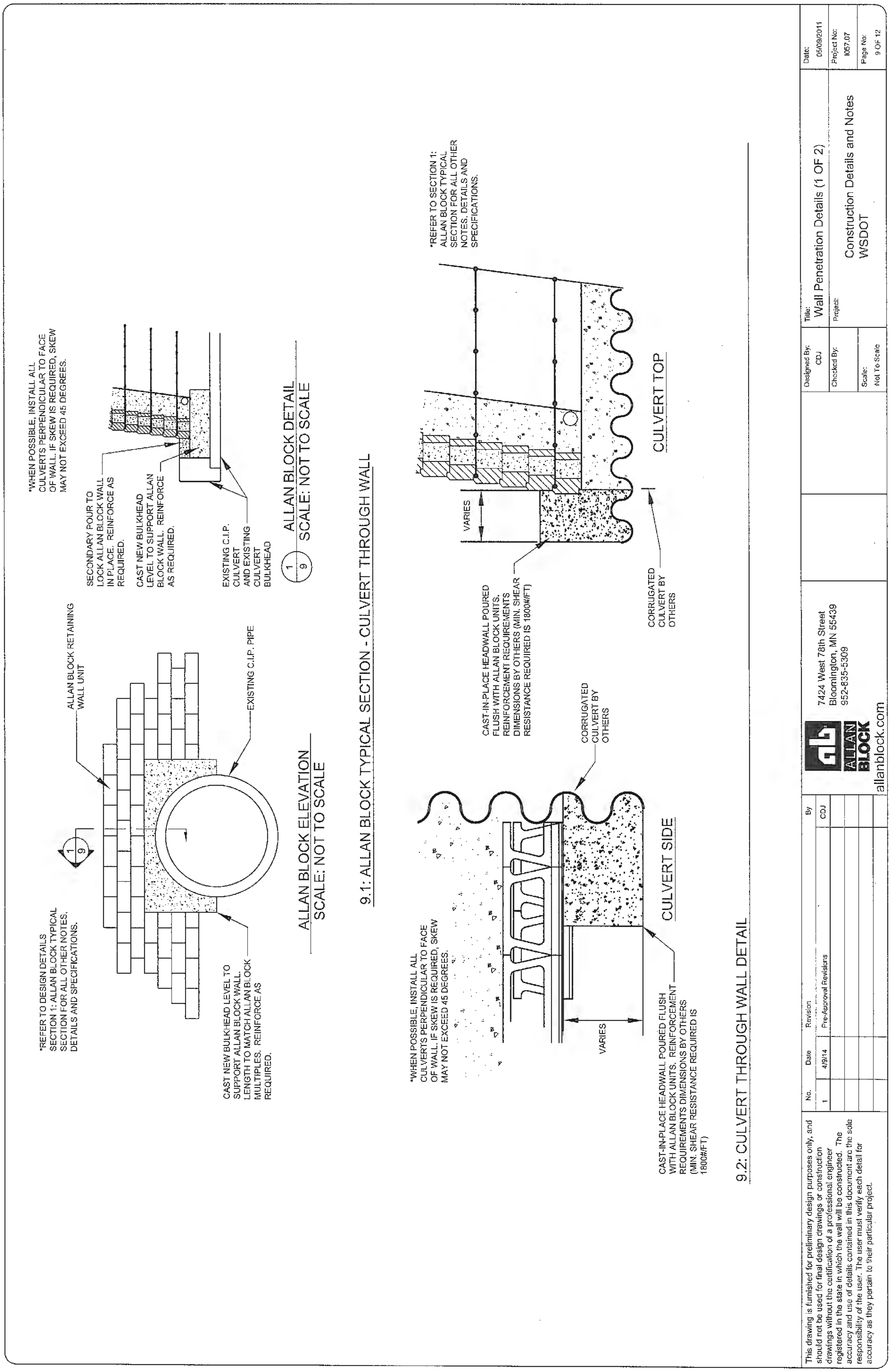
Designed By: CDJ	Title: Traffic Barrier and Coping Details	Date: 05/09/2011
Checked By:	Project: Construction Details and Notes WSDOT	Project No: 1057.07
Scale: Not To Scale		Page No: 8 OF 12


ALLAN BLOCK ELEVATION
SCALE: NOT TO SCALE
ALLAN BLOCK DETAIL
SCALE: NOT TO SCALE
9.1: ALLAN BLOCK TYPICAL SECTION - CULVERT THROUGH WALL
CULVERT SIDE
CULVERT TOP
9.2: CULVERT THROUGH WALL DETAIL
*REFER TO DESIGN DETAILS SECTION 1: ALLAN BLOCK TYPICAL SECTION FOR ALL OTHER NOTES, DETAILS AND SPECIFICATIONS.
ALLAN BLOCK RETAINING WALL UNIT
EXISTING C.I.P. PIPE
CAST NEW BULKHEAD LEVEL TO SUPPORT ALLAN BLOCK WALL. LENGTH TO MATCH ALLAN BLOCK MULTIPLES. REINFORCE AS REQUIRED.
*WHEN POSSIBLE, INSTALL ALL CULVERTS PERPENDICULAR TO FACE OF WALL. IF SKEW IS REQUIRED, SKEW MAY NOT EXCEED 45 DEGREES.
SECONDARY POUR TO LOCK ALLAN BLOCK WALL IN PLACE. REINFORCE AS REQUIRED.
CAST NEW BULKHEAD LEVEL TO SUPPORT ALLAN BLOCK WALL. REINFORCE AS REQUIRED.
EXISTING C.I.P. CULVERT AND EXISTING CULVERT BULKHEAD
CAST-IN-PLACE HEADWALL POURED FLUSH WITH ALLAN BLOCK UNITS. REINFORCEMENT REQUIREMENTS DIMENSIONS BY OTHERS (MIN. SHEAR RESISTANCE REQUIRED IS 1800#/FT)
CORRUGATED CULVERT BY OTHERS
VARIES
*REFER TO SECTION 1: ALLAN BLOCK TYPICAL SECTION FOR ALL OTHER NOTES, DETAILS AND SPECIFICATIONS.
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ALLAN BLOCK
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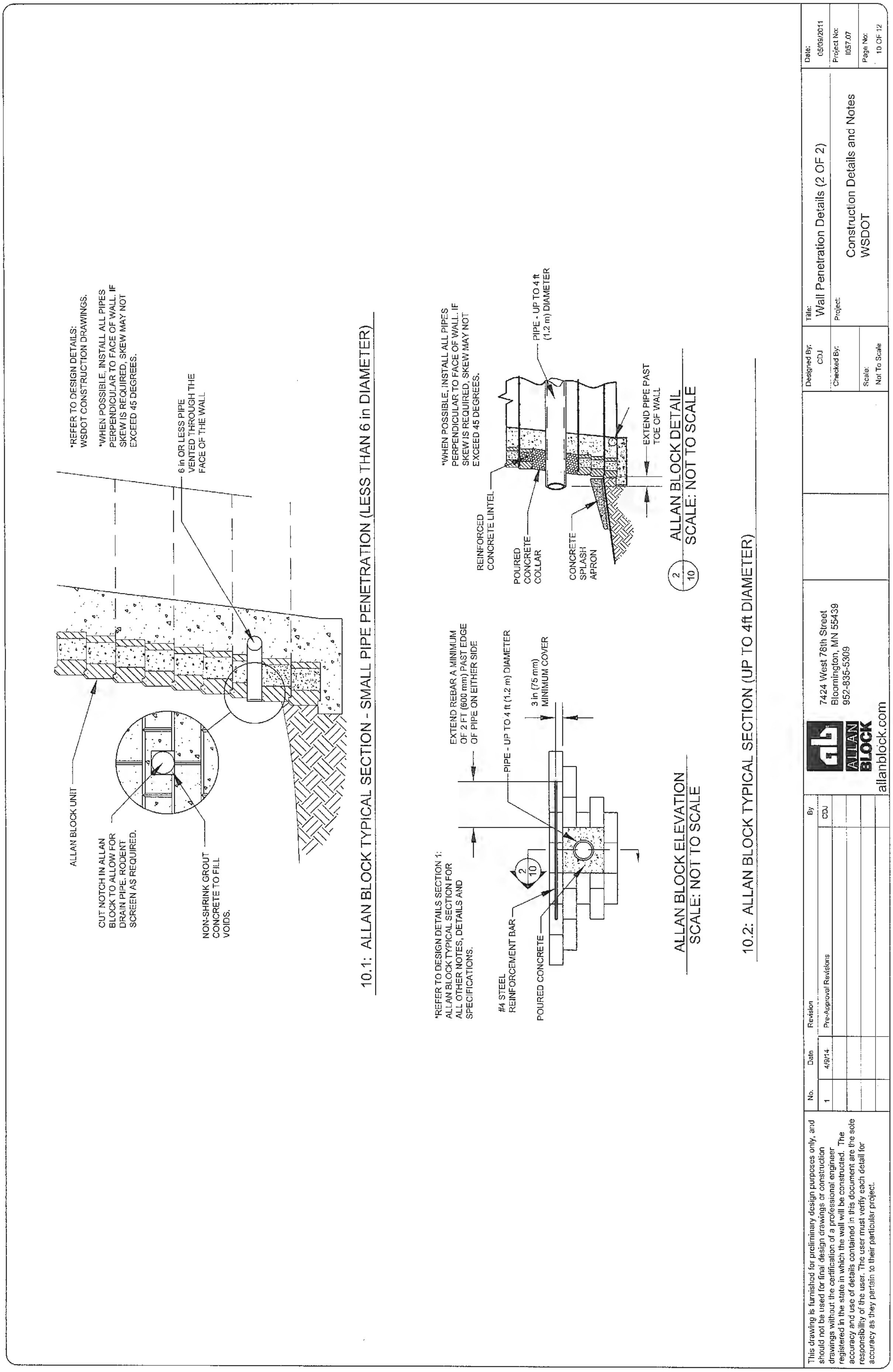
7424 West 78th Street
Bloomington, MN 55439
952-835-5309
Designed By: CDJ
Checked By:
Scale: Not To Scale
Title: Wall Penetration Details (1 OF 2)
Project: Construction Details and Notes WSDOT
Date: 05/09/2011
Project No: I057.07
Page No: 9 OF 12


*REFER TO DESIGN DETAILS: WSDOT CONSTRUCTION DRAWINGS.
*WHEN POSSIBLE, INSTALL ALL PIPES PERPENDICULAR TO FACE OF WALL. IF SKEW IS REQUIRED, SKEW MAY NOT EXCEED 45 DEGREES.
6 in OR LESS PIPE VENTED THROUGH THE FACE OF THE WALL
ALLAN BLOCK UNIT
CUT NOTCH IN ALLAN BLOCK TO ALLOW FOR DRAIN PIPE. RODENT SCREEN AS REQUIRED.
NON-SHRINK GROUT CONCRETE TO FILL VOIDS.
10.1: ALLAN BLOCK TYPICAL SECTION - SMALL PIPE PENETRATION (LESS THAN 6 in DIAMETER)
*REFER TO DESIGN DETAILS SECTION 1: ALLAN BLOCK TYPICAL SECTION FOR ALL OTHER NOTES, DETAILS AND SPECIFICATIONS.
#4 STEEL REINFORCEMENT BAR
POURED CONCRETE
EXTEND REBAR A MINIMUM OF 2 FT (600 mm) PAST EDGE OF PIPE ON EITHER SIDE
PIPE - UP TO 4 ft (1.2 m) DIAMETER
3 in (75 mm) MINIMUM COVER
2
10
ALLAN BLOCK ELEVATION
SCALE: NOT TO SCALE
*WHEN POSSIBLE, INSTALL ALL PIPES PERPENDICULAR TO FACE OF WALL. IF SKEW IS REQUIRED, SKEW MAY NOT EXCEED 45 DEGREES.
REINFORCED CONCRETE LINTEL
POURED CONCRETE COLLAR
CONCRETE SPLASH APRON
PIPE - UP TO 4 ft (1.2 m) DIAMETER
EXTEND PIPE PAST TOE OF WALL
ALLAN BLOCK DETAIL
SCALE: NOT TO SCALE
10.2: ALLAN BLOCK TYPICAL SECTION (UP TO 4ft DIAMETER)
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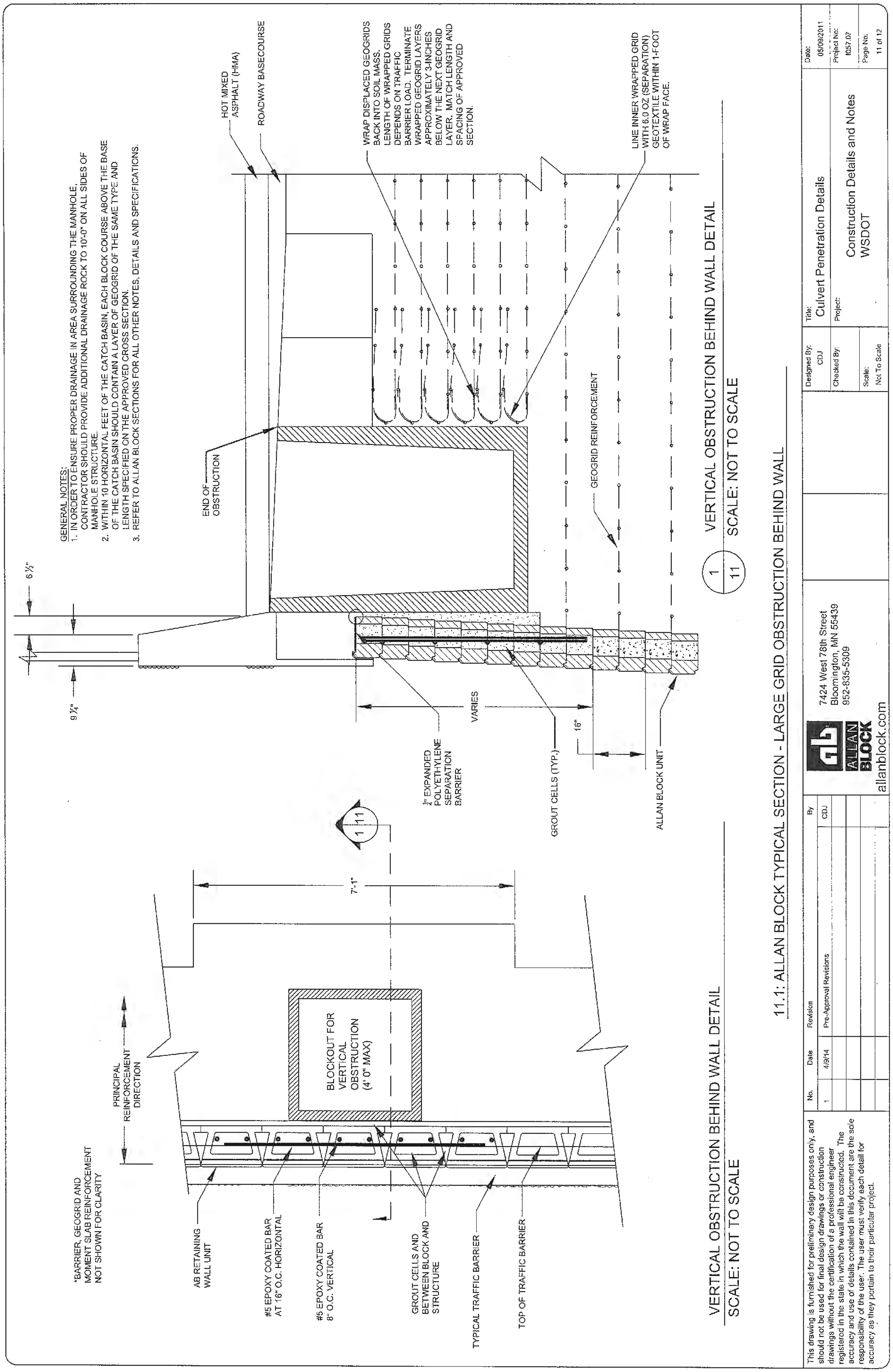
GENERAL NOTES:
1. IN ORDER TO ENSURE PROPER DRAINAGE IN AREA SURROUNDING THE MANHOLE, CONTRACTOR SHOULD PROVIDE ADDITIONAL DRAINAGE ROCK TO 10'-0" ON ALL SIDES OF MANHOLE STRUCTURE.
2. WITHIN 10 HORIZONTAL FEET OF THE CATCH BASIN, EACH BLOCK COURSE ABOVE THE BASE OF THE CATCH BASIN SHOULD CONTAIN A LAYER OF GEOGRID OF THE SAME TYPE AND LENGTH SPECIFIED ON THE APPROVED CROSS SECTION.
3. REFER TO ALLAN BLOCK SECTIONS FOR ALL OTHER NOTES, DETAILS AND SPECIFICATIONS.
HOT MIXED ASPHALT (HMA)
ROADWAY BASECOURSE
WRAP DISPLACED GEOGRIDS BACK INTO SOIL MASS. LENGTH OF WRAPPED GRIDS DEPENDS ON TRAFFIC BARRIER LOAD. TERMINATE WRAPPED GEOGRID LAYERS APPROXIMATELY 3-INCHES BELOW THE NEXT GEOGRID LAYER. MATCH LENGTH AND SPACING OF APPROVED SECTION.
LINE INNER WRAPPED GRID WITH 6.0 OZ (SEPARATION) GEOTEXTILE WITHIN 1-FOOT OF WRAP FACE.
END OF OBSTRUCTION
GEOGRID REINFORCEMENT
6 1/2"
9 1/4"
VARIES
1/2" EXPANDED POLYETHYLENE SEPARATION BARRIER
GROUT CELLS (TYP.)
16"
ALLAN BLOCK UNIT
1 11
VERTICAL OBSTRUCTION BEHIND WALL DETAIL
SCALE: NOT TO SCALE
7'-1"
BLOCKOUT FOR VERTICAL OBSTRUCTION (4' 0" MAX)
PRINCIPAL REINFORCEMENT DIRECTION
*BARRIER, GEOGRID AND MOMENT SLAB REINFORCEMENT NOT SHOWN FOR CLARITY
AB RETAINING WALL UNIT
#5 EPOXY COATED BAR AT 16" O.C. HORIZONTAL
#5 EPOXY COATED BAR 8" O.C. VERTICAL
GROUT CELLS AND BETWEEN BLOCK AND STRUCTURE
TYPICAL TRAFFIC BARRIER
TOP OF TRAFFIC BARRIER
VERTICAL OBSTRUCTION BEHIND WALL DETAIL
SCALE: NOT TO SCALE
11.1: ALLAN BLOCK TYPICAL SECTION - LARGE GRID OBSTRUCTION BEHIND WALL
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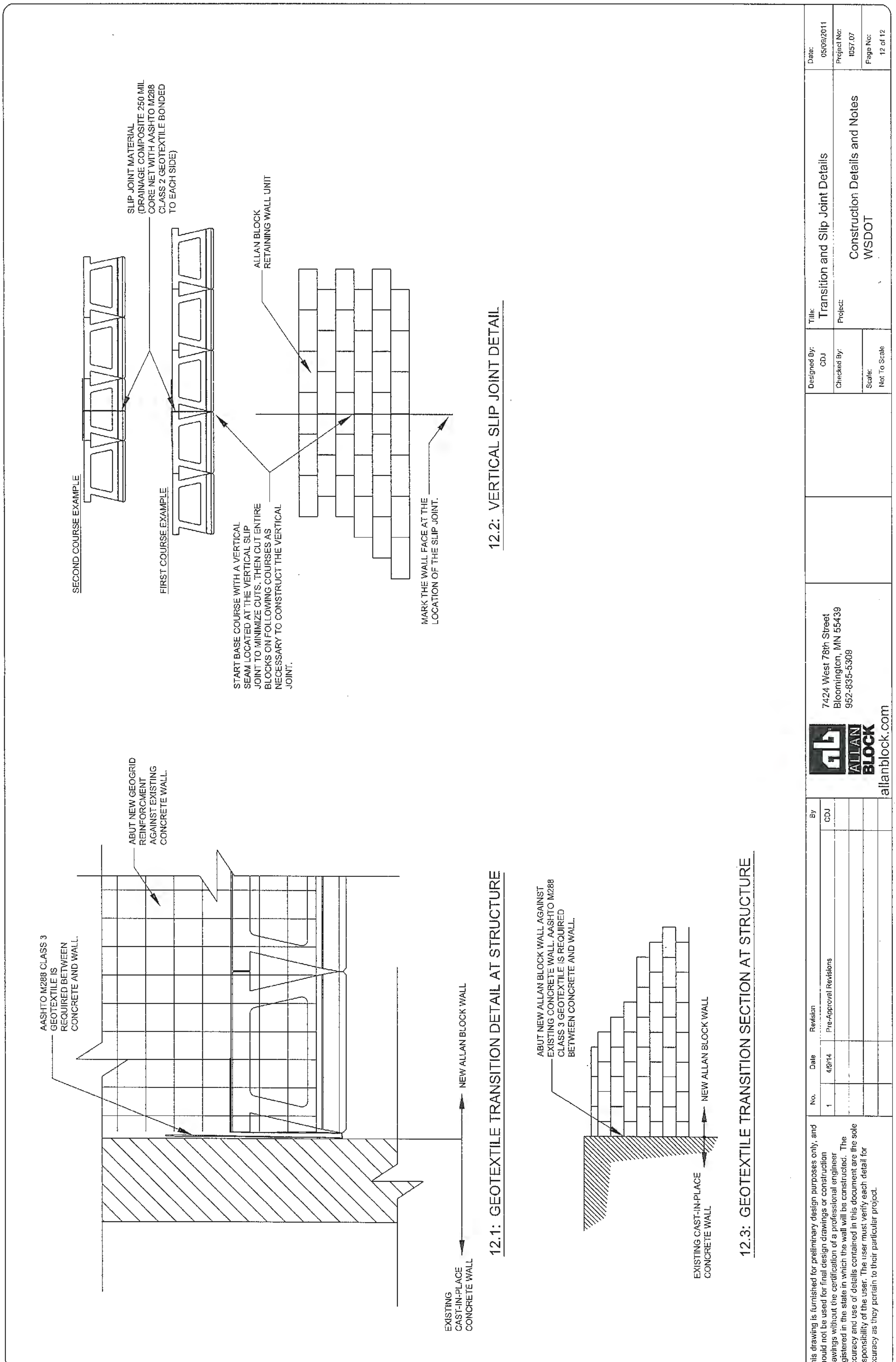
ALLAN BLOCK
allanblock.com
7424 West 78th Street
Bloomington, MN 55439
952-835-5309
Designed By: CDJ
Checked By:
Scale: Not To Scale
Title: Culvert Penetration Details
Project: Construction Details and Notes WSDOT
Date: 05/09/2011
Project No: 1057.07
Page No: 11 of 12


SLIP JOINT MATERIAL (DRAINAGE COMPOSITE 250 MIL CORE NET WITH AASHTO M288 CLASS 2 GEOTEXTILE BONDED TO EACH SIDE)
SECOND COURSE EXAMPLE
FIRST COURSE EXAMPLE
ALLAN BLOCK RETAINING WALL UNIT
START BASE COURSE WITH A VERTICAL SEAM LOCATED AT THE VERTICAL SLIP JOINT TO MINIMIZE CUTS. THEN CUT ENTIRE BLOCKS ON FOLLOWING COURSES AS NECESSARY TO CONSTRUCT THE VERTICAL JOINT.
MARK THE WALL FACE AT THE LOCATION OF THE SLIP JOINT.
12.2: VERTICAL SLIP JOINT DETAIL
AASHTO M288 CLASS 3 GEOTEXTILE IS REQUIRED BETWEEN CONCRETE AND WALL.
ABUT NEW GEOGRID REINFORCMENT AGAINST EXISTING CONCRETE WALL.
EXISTING CAST-IN-PLACE CONCRETE WALL
NEW ALLAN BLOCK WALL
12.1: GEOTEXTILE TRANSITION DETAIL AT STRUCTURE
ABUT NEW ALLAN BLOCK WALL AGAINST EXISTING CONCRETE WALL. AASHTO M288 CLASS 3 GEOTEXTILE IS REQUIRED BETWEEN CONCRETE AND WALL.
EXISTING CAST-IN-PLACE CONCRETE WALL
NEW ALLAN BLOCK WALL
12.3: GEOTEXTILE TRANSITION SECTION AT STRUCTURE
This drawing is furnished for preliminary design purposes only, and should not be used for final design drawings or construction drawings without the certification of a professional engineer registered in the state in which the wall will be constructed. The accuracy and use of details contained in this document are the sole responsibility of the user. The user must verify each detail for accuracy as they pertain to their particular project.
No. Date Revision By
1 4/9/14 Pre-Approval Revisions CDJ
ALLAN BLOCK
allanblock.com
7424 West 78th Street
Bloomington, MN 55439
952-835-5309
Designed By: CDJ
Checked By:
Scale: Not To Scale
Title: Transition and Slip Joint Details
Project: Construction Details and Notes WSDOT
Date: 05/09/2011
Project No: 1057.07
Page No: 12 of 12

Appendix 15-Q Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply:

Facing Blocks –Blocks acceptable for use with this wall system are the 28-inch Positive Connection blocks. The 41-inch blocks shown in the drawings are not considered part of the approved system.

Soil Reinforcement – Only geosynthetic reinforcement listed in the WSDOT QPL and which has been evaluated for connection strength with the Redi-Rock Positive Connection wall system shall be used. The following products are approved for use with this wall system:

Miragrid 5XT
Miragrid 8XT
Miragrid 10XT
Miragrid 20XT
Miragrid 24XT

All Miragrid products for the Redi-Rock Positive Connection system will be 12-inch wide rolls consisting of 11 longitudinal ribs. TenCate Geosynthetics will provide certification of the wide width tensile strength of the 12-inch wide rolls.

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications.

Reinforcement/Facing Block Connection Requirements – The connection between the facing units and the geosynthetic reinforcement is essentially independent of the normal force between the blocks (i.e., not a frictional connection), as the reinforcement strips wrap around the internal wall of the block as a continuous layer. The design facing/reinforcement connection strength shall be as specified in the following table:

Table 15-Q-1 Approved connection strength design values for Redi-Rock walls

Geogrid Product	$T_{ultconn}$ (lbs/ft)	T_{lot} (lbs/ft)
Miragrid 5XT	4,460	5,334
Miragrid 8XT	7,928	8,055
Miragrid 10XT	8,681	10,635
Miragrid 20XT	13,447	16,397
Miragrid 24XT	20,199	29,130

T_{ac}, the long-term connection strength, shall be calculated as follows:

$$T_{ac} = \frac{T_{MARV} \bullet CR_u}{RF_{CR} \bullet RF_D} \tag{15-Q-1}$$

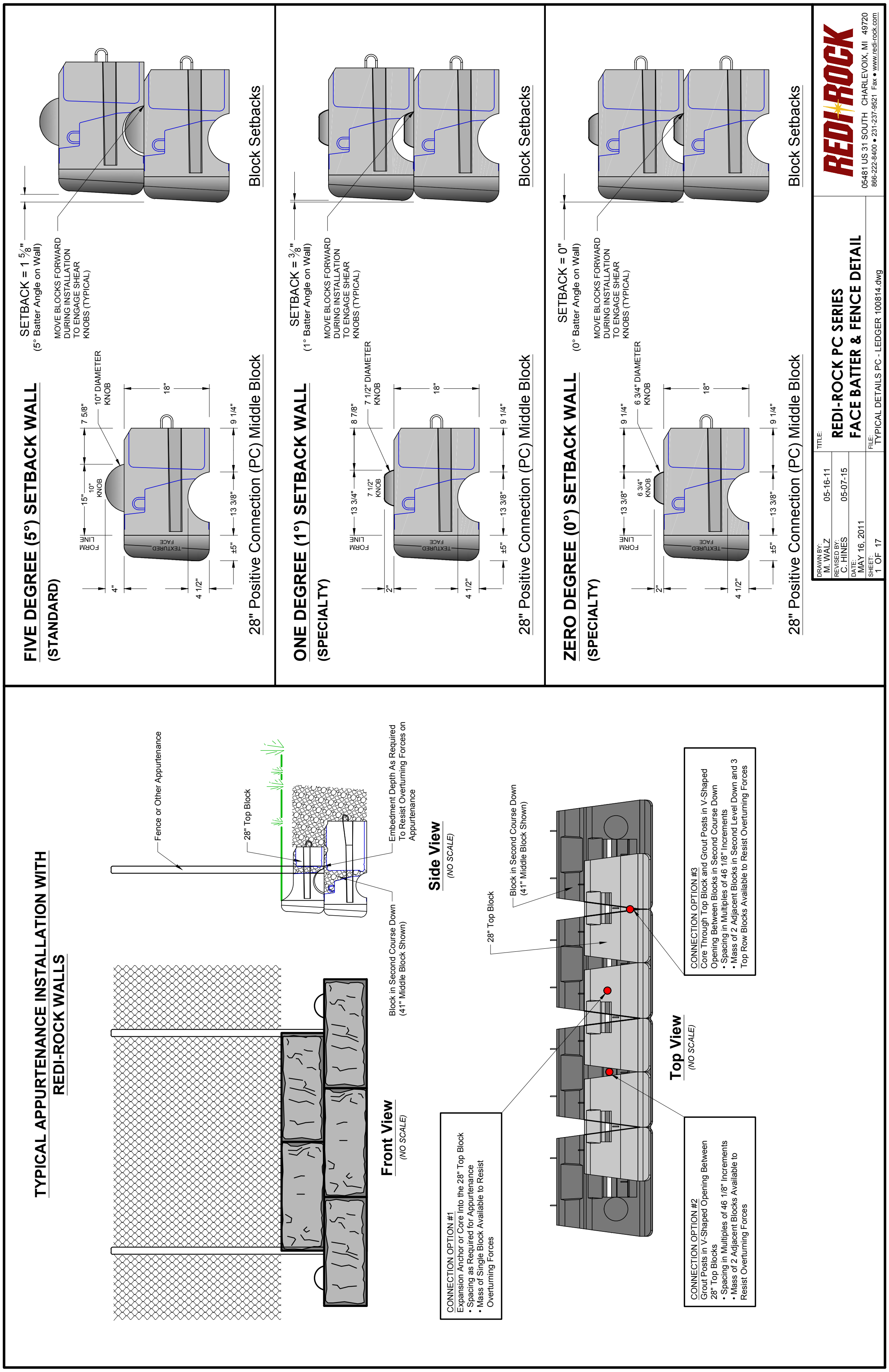
where,

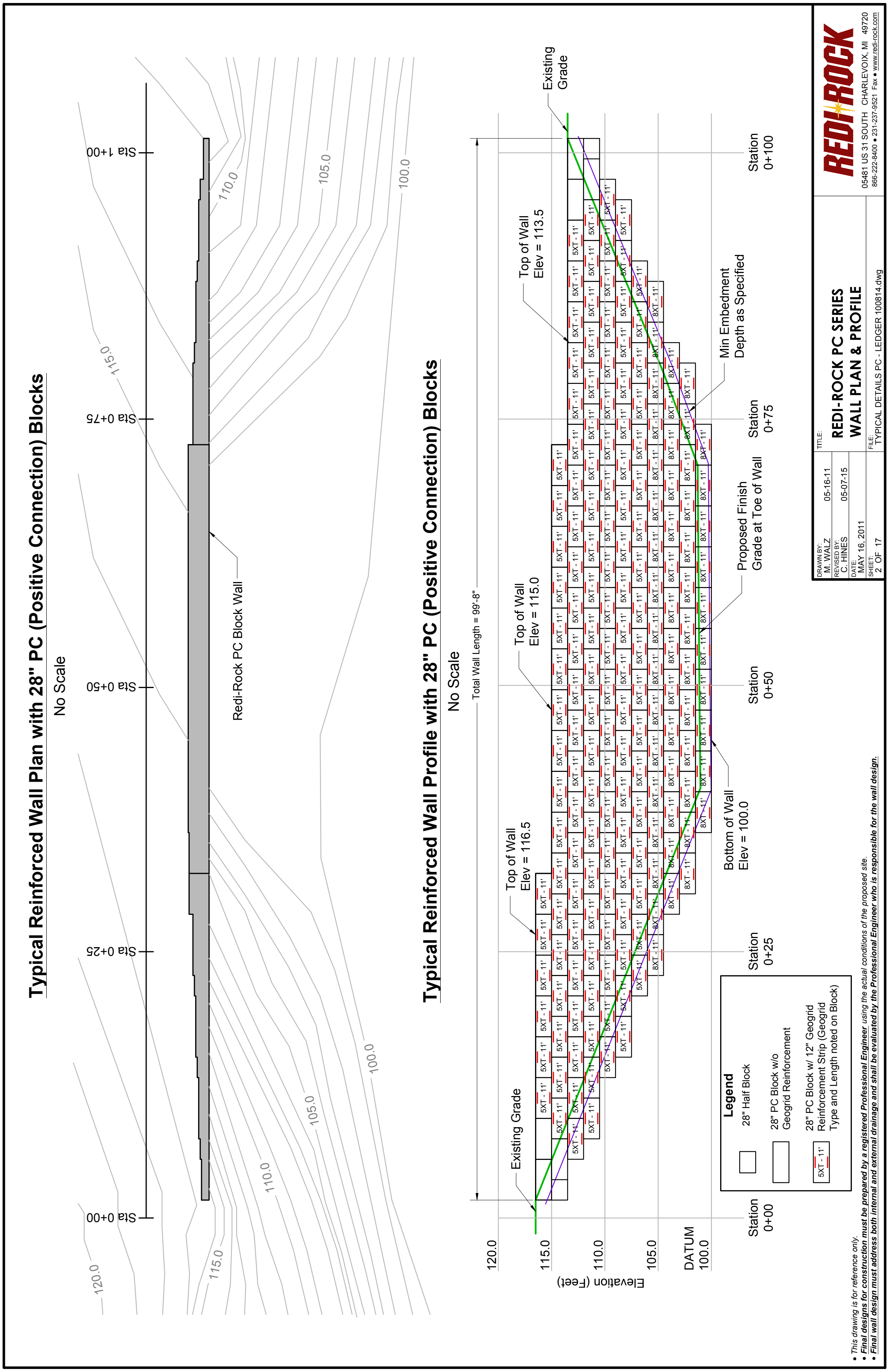
T_{MARV} = the minimum average roll value for the ultimate geosynthetic strength T_{ult},
CR_u = $T_{ultconn}/T_{lot}$, in which $T_{ultconn}$ is the ultimate connection strength and T_{lot} is the lot specific ultimate tensile strength, (i.e., the lot or roll of material specific to the connection testing),
RF_{CR} = creep reduction factor for the geosynthetic, and
RF_D = the durability reduction factor for the geosynthetic.

RF_{CR} and RF_D shall be as provided in the WSDOT QPL, Appendix D.

Approved details for the Redi-Rock Positive Connection wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- Retaining wall heights up to a maximum of 33 feet.
- Retaining walls having a wall face batter of one degree to five degrees.
- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet.
- The pipe penetration details for pipes oriented up to a 45 degree skew angle as measured from perpendicular to the wall face are preapproved for pipe diameters of 18 inches or less.
- The cast-in-place concrete to be constructed around pipes that are protruding through the wall face is considered non-preapproved. Detailed stamped drawings and stamped engineering calculations are to be submitted for approval on a project specific basis.
- Reinforcement pullout design shall be calculated based on the default values for geogrid reinforcement provided in the latest edition of the AASHTO LRFD Bridge Design Specifications.


TYPICAL APPURTENANCE INSTALLATION WITH REDI-ROCK WALLS
Front View
(NO SCALE)
Fence or Other Appurtenance
28" Top Block
Block in Second Course Down (41" Middle Block Shown)
Embedment Depth As Required To Resist Overturning Forces on Appurtenance
Side View
(NO SCALE)
CONNECTION OPTION #1
Expansion Anchor or Core Into the 28" Top Block
• Spacing as Required for Appurtenance
• Mass of Single Block Available to Resist Overturning Forces
28" Top Block
Block in Second Course Down (41" Middle Block Shown)
Top View
(NO SCALE)
CONNECTION OPTION #2
Grout Posts in V-Shaped Opening Between 28" Top Blocks
• Spacing in Multiples of 46 1/8" Increments
• Mass of 2 Adjacent Blocks Available to Resist Overturning Forces
CONNECTION OPTION #3
Core Through Top Block and Grout Posts in V-Shaped Opening Between Blocks in Second Course Down
• Spacing in Multiples of 46 1/8" Increments
• Mass of 2 Adjacent Blocks in Second Level Down and 3 Top Row Blocks Available to Resist Overturning Forces
FIVE DEGREE (5°) SETBACK WALL
(STANDARD)
SETBACK = 1 5/8"
(5° Batter Angle on Wall)
MOVE BLOCKS FORWARD DURING INSTALLATION TO ENGAGE SHEAR KNOBS (TYPICAL)
FORM LINE
15"
10" KNOB
7 5/8"
10" DIAMETER KNOB
4"
TEXTURED FACE
18"
4 1/2"
±5"
13 3/8"
9 1/4"
28" Positive Connection (PC) Middle Block
Block Setbacks
ONE DEGREE (1°) SETBACK WALL
(SPECIALTY)
SETBACK = 3/8"
(1° Batter Angle on Wall)
MOVE BLOCKS FORWARD DURING INSTALLATION TO ENGAGE SHEAR KNOBS (TYPICAL)
FORM LINE
13 3/4"
7 1/2" KNOB
8 7/8"
7 1/2" DIAMETER KNOB
2"
TEXTURED FACE
18"
4 1/2"
±5"
13 3/8"
9 1/4"
28" Positive Connection (PC) Middle Block
Block Setbacks
ZERO DEGREE (0°) SETBACK WALL
(SPECIALTY)
SETBACK = 0"
(0° Batter Angle on Wall)
MOVE BLOCKS FORWARD DURING INSTALLATION TO ENGAGE SHEAR KNOBS (TYPICAL)
FORM LINE
13 3/8"
6 3/4" KNOB
9 1/4"
6 3/4" DIAMETER KNOB
2"
TEXTURED FACE
18"
4 1/2"
±5"
13 3/8"
9 1/4"
28" Positive Connection (PC) Middle Block
Block Setbacks
DRAWN BY: M. WALZ 05-16-11
REVISED BY: C. HINES 05-07-15
DATE: MAY 16, 2011
SHEET: 1 OF 17
TITLE: REDI-ROCK PC SERIES FACE BATTER & FENCE DETAIL
FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg
REDI-ROCK
05481 US 31 SOUTH CHARLEVOIX, MI 49720
866-222-8400 • 231-237-9521 Fax • www.redi-rock.com


Typical Reinforced Wall Plan with 28" PC (Positive Connection) Blocks
No Scale
Sta 0+00
Sta 0+25
Sta 0+50
Sta 0+75
Sta 1+00
Redi-Rock PC Block Wall
120.0
115.0
110.0
105.0
100.0
Typical Reinforced Wall Profile with 28" PC (Positive Connection) Blocks
No Scale
Total Wall Length = 99'-8"
Existing Grade
Top of Wall Elev = 116.5
Top of Wall Elev = 115.0
Top of Wall Elev = 113.5
Bottom of Wall Elev = 100.0
Proposed Finish Grade at Toe of Wall
Min Embedment Depth as Specified
Elevation (Feet)
DATUM
Station 0+00
Station 0+25
Station 0+50
Station 0+75
Station 0+100
5XT - 11'
8XT - 11'
Legend
28" Half Block
28" PC Block w/o Geogrid Reinforcement
28" PC Block w/ 12" Geogrid Reinforcement Strip (Geogrid Type and Length noted on Block)
• This drawing is for reference only.
• Final designs for construction must be prepared by a registered Professional Engineer using the actual conditions of the proposed site.
• Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.
DRAWN BY: M. WALZ 05-16-11
REVISED BY: C. HINES 05-07-15
DATE: MAY 16, 2011
SHEET: 2 OF 17
TITLE: REDI-ROCK PC SERIES WALL PLAN & PROFILE
FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg
REDI-ROCK
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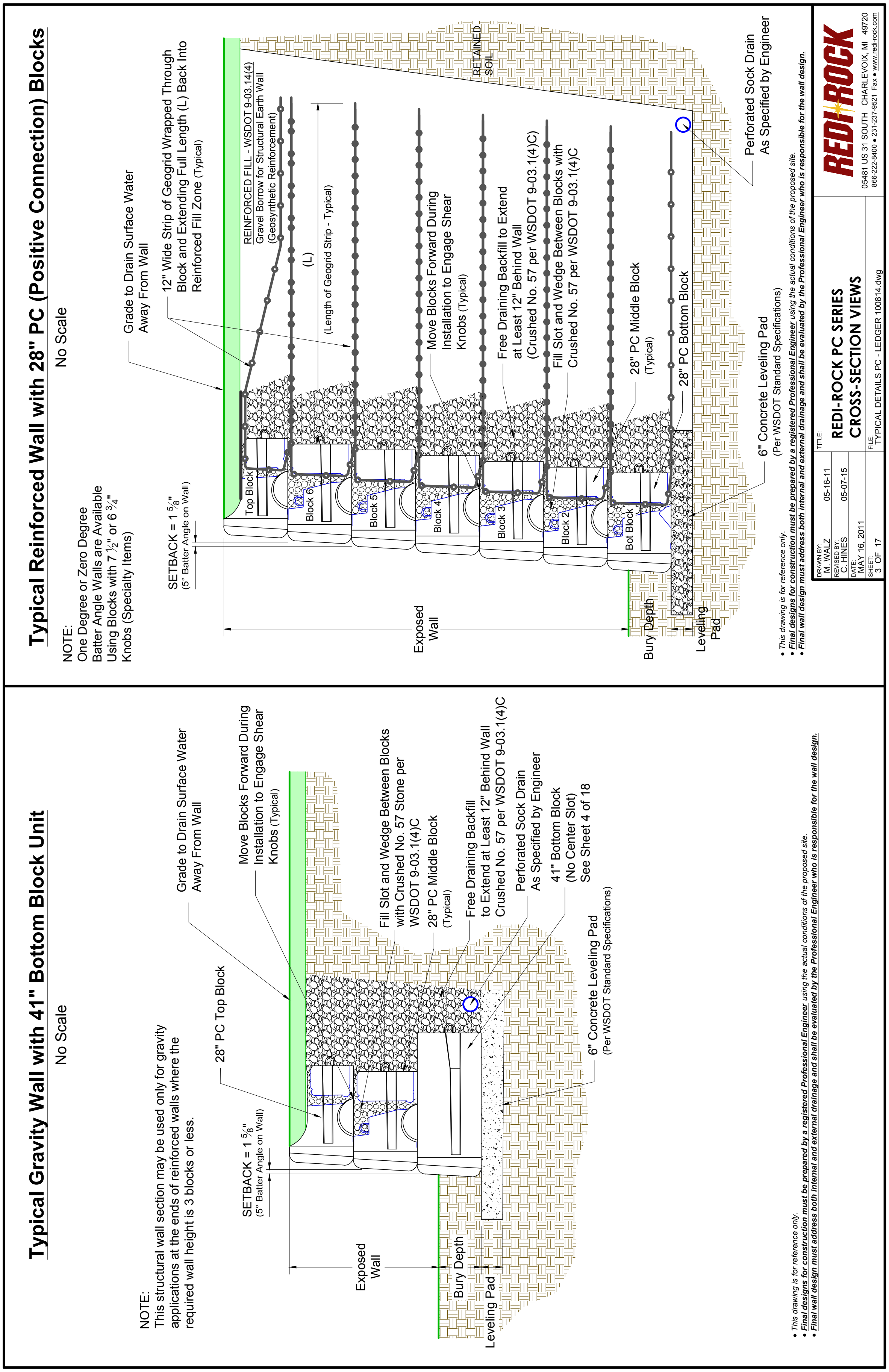
866-222-8400 • 231-237-9521 Fax • www.redi-rock.com


Typical Reinforced Wall with 28" PC (Positive Connection) Blocks
No Scale
NOTE:
One Degree or Zero Degree Batter Angle Walls are Available Using Blocks with 7 1/2" or 6 3/4" Knobs (Specialty Items)
SETBACK = 1 5/8"
(5° Batter Angle on Wall)
Grade to Drain Surface Water Away From Wall
12" Wide Strip of Geogrid Wrapped Through Block and Extending Full Length (L) Back Into Reinforced Fill Zone (Typical)
REINFORCED FILL - WSDOT 9-03.14(4)
Gravel Borrow for Structural Earth Wall
(Geosynthetic Reinforcement)
(L)
(Length of Geogrid Strip - Typical)
RETAINED SOIL
Move Blocks Forward During Installation to Engage Shear Knobs (Typical)
Free Draining Backfill to Extend at Least 12" Behind Wall (Crushed No. 57 per WSDOT 9-03.1(4)C)
Fill Slot and Wedge Between Blocks with Crushed No. 57 per WSDOT 9-03.1(4)C
28" PC Middle Block (Typical)
28" PC Bottom Block
Perforated Sock Drain As Specified by Engineer
6" Concrete Leveling Pad (Per WSDOT Standard Specifications)
Top Block
Block 6
Block 5
Block 4
Block 3
Block 2
Bot Block
Exposed Wall
Bury Depth
Leveling Pad
• This drawing is for reference only.
• Final designs for construction must be prepared by a registered Professional Engineer using the actual conditions of the proposed site.
• Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.
DRAWN BY: M. WALZ 05-16-11
REVISED BY: C. HINES 05-07-15
DATE: MAY 16, 2011
SHEET: 3 OF 17
TITLE: REDI-ROCK PC SERIES CROSS-SECTION VIEWS
FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg
REDI-ROCK
05481 US 31 SOUTH CHARLEVOIX, MI 49720
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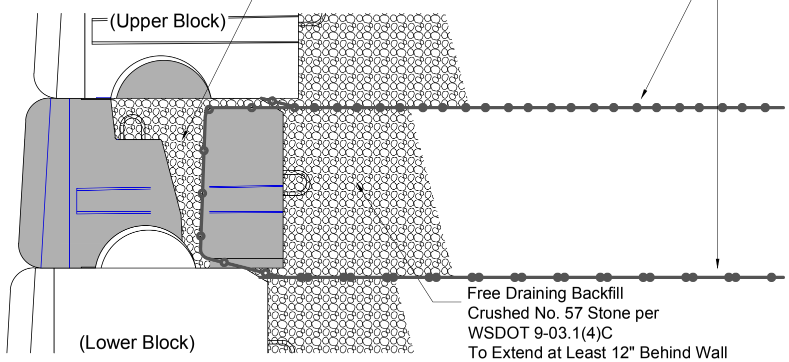
Typical Gravity Wall with 41" Bottom Block Unit
No Scale
NOTE:
This structural wall section may be used only for gravity applications at the ends of reinforced walls where the required wall height is 3 blocks or less.
SETBACK = 1 5/8"
(5° Batter Angle on Wall)
28" PC Top Block
Grade to Drain Surface Water Away From Wall
Move Blocks Forward During Installation to Engage Shear Knobs (Typical)
Fill Slot and Wedge Between Blocks with Crushed No. 57 Stone per WSDOT 9-03.1(4)C
28" PC Middle Block (Typical)
Free Draining Backfill to Extend at Least 12" Behind Wall Crushed No. 57 per WSDOT 9-03.1(4)C
Perforated Sock Drain As Specified by Engineer
41" Bottom Block (No Center Slot) See Sheet 4 of 18
6" Concrete Leveling Pad (Per WSDOT Standard Specifications)
Exposed Wall
Bury Depth
Leveling Pad
• This drawing is for reference only.
• Final designs for construction must be prepared by a registered Professional Engineer using the actual conditions of the proposed site.
• Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.

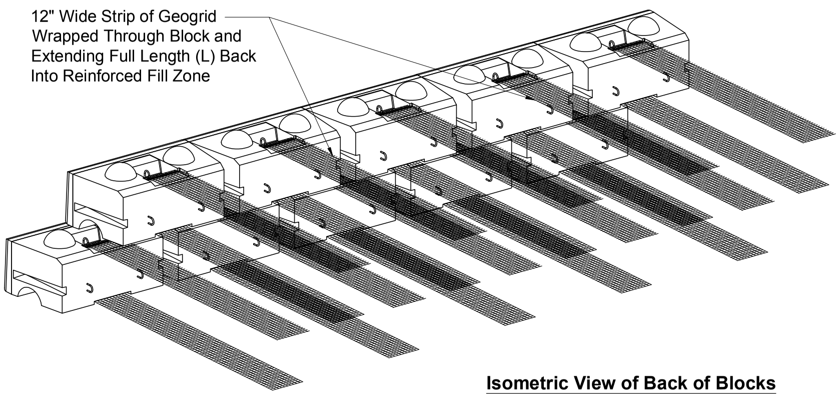
Positive Connection (PC) Details

See www.redi-rock.com for Geogrid Connection and Interface Shear Test Reports.



Section View Through Blocks

No Scale



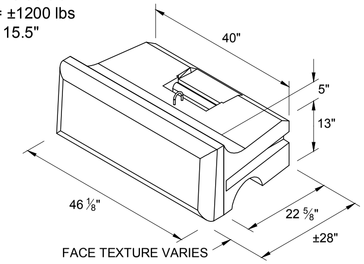
Isometric View of Back of Blocks

No Scale

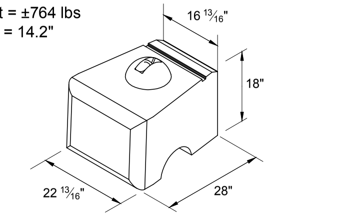
- *This drawing is for reference only.*
- ***Final designs for construction must be prepared by a registered Professional Engineer** using the actual conditions of the proposed site.*
- ***Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.***

REDI-ROCK PC (POSITIVE CONNECTION) BLOCKS

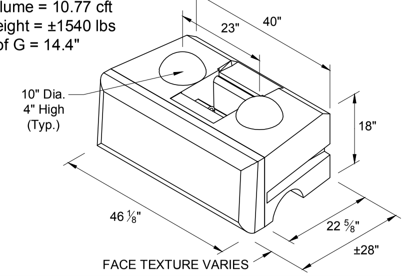
Top - 28" PC Block
Volume = 8.38 cft
Weight = ±1200 lbs
C of G = 15.5"



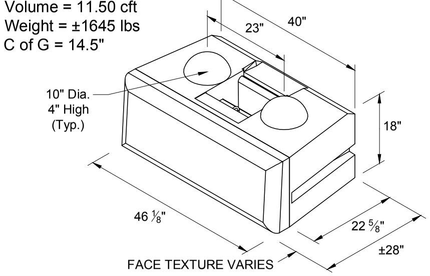
Half - 28" Middle Block
Volume = 5.34 cft
Weight = ±764 lbs
C of G = 14.2"



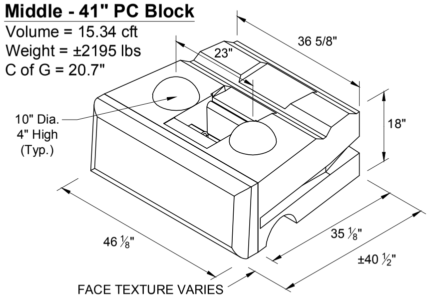
Middle - 28" PC Block
Volume = 10.77 cft
Weight = ±1540 lbs
C of G = 14.4"



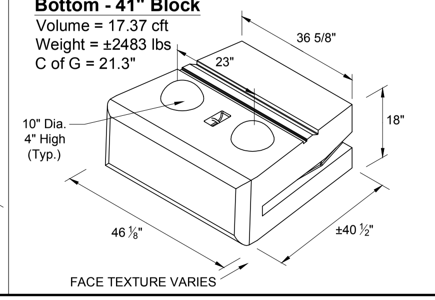
Bottom - 28" PC Block
Volume = 11.50 cft
Weight = ±1645 lbs
C of G = 14.5"



Middle - 41" PC Block
Volume = 15.34 cft
Weight = ±2195 lbs
C of G = 20.7"



Bottom - 41" Block
Volume = 17.37 cft
Weight = ±2483 lbs
C of G = 21.3"



NOTES:

Volume and Center of Gravity (C of G) calculations are based on the blocks as shown.
Center of Gravity is measured from the back of the block.
Half blocks may include a fork lift slot on one side.
Actual weights and volumes may vary.
Weight shown is based on 143 pcf concrete.

DRAWN BY: M. WALZ	05-16-11	TITLE: REDI-ROCK PC SERIES SYSTEM COMPONENT DETAILS
REVISED BY: C. HINES	05-07-15	
DATE: MAY 16, 2011		
SHEET NO.: 4 of 17		DRAWING FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg

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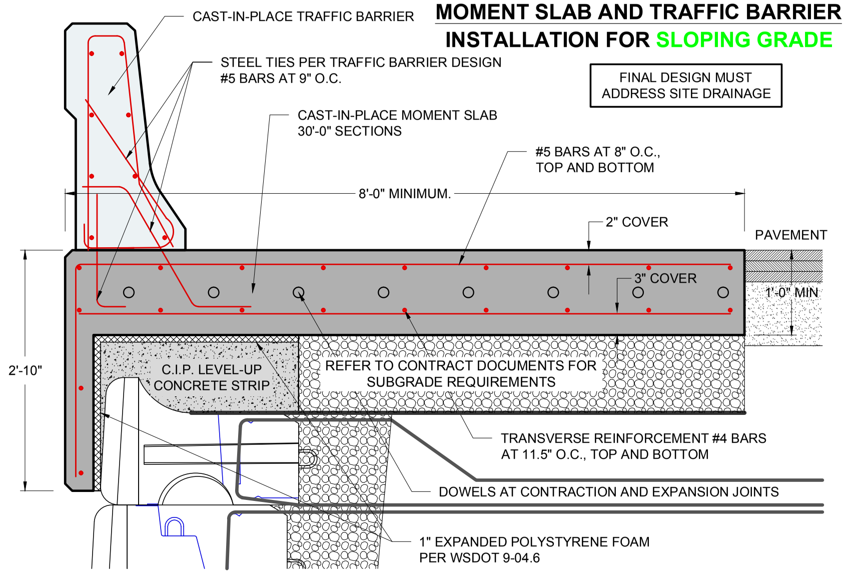
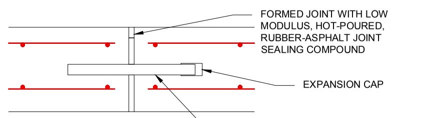
MOMENT SLAB AND TRAFFIC BARRIER INSTALLATION FOR SLOPING GRADE

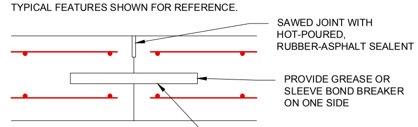


EXPANSION JOINTS SHALL BE PROVIDED IN MOMENT SLAB EVERY 90'-0".
EXPANSION JOINT SHALL BE DOT STANDARD DETAIL.
TYPICAL FEATURES SHOWN FOR REFERENCE.



EXPANSION JOINT

CONTRACTION JOINTS SHALL BE PROVIDED IN MOMENT SLAB EVERY 30'-0" BETWEEN EXPANSION JOINTS.
CONTRACTION JOINT SHALL BE DOT STANDARD DETAIL.
TYPICAL FEATURES SHOWN FOR REFERENCE.



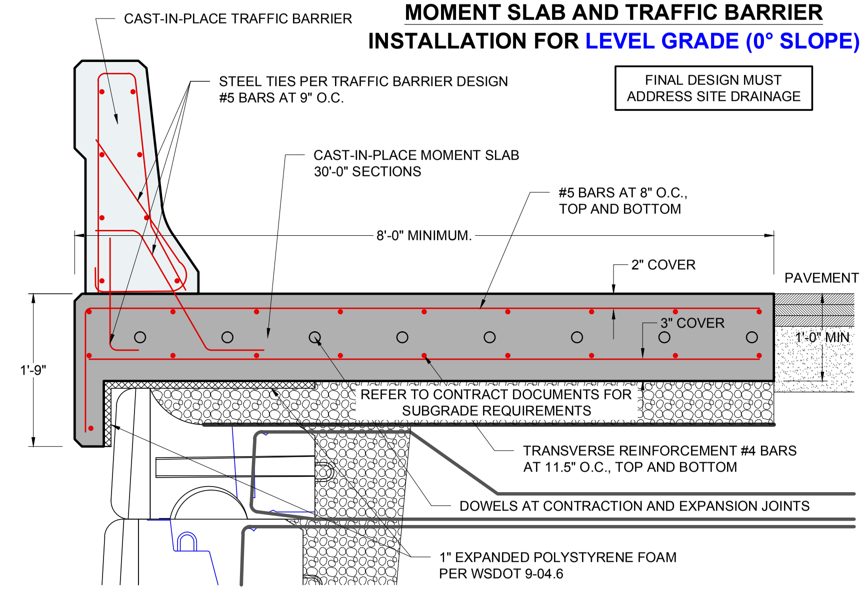
CONTRACTION JOINT

MATERIALS
CONCRETE FOR CAST-IN-PLACE MOMENT SLAB SHALL BE DOT STANDARD STRUCTURE MIX. MINIMUM 28 DAY COMPRESSIVE STRENGTH SHALL BE 4,000 PSI OR HIGHER AS SPECIFIED.
CAST-IN-PLACE LEVEL UP CONCRETE SHALL BE MANUFACTURED IN ACCORDANCE WITH ASTM C94 AND SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 4,000 PSI.
REINFORCING STEEL SHALL CONFORM TO ASTM A706 OR AASHTO M31 GRADE 60.
ALL REINFORCING STEEL SHALL BE EPOXY COATED IN ACCORDANCE WITH ASTM A775 OR ASTM A934.

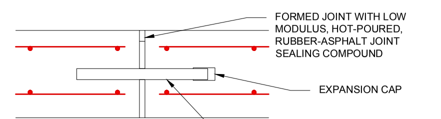
DESIGN
MOMENT SLAB SHOWN IS DIMENSIONED BASED ON AN EQUIVALENT STATIC LOAD OF 10,000 LBS PER NCHRP REPORT 663.
MOMENT SLAB REINFORCEMENT SHOWN IS BASED ON AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 5TH EDITION, 2010, TL-4 LOADING DETAILED IN TABLE A13.2.1. AND REQUIREMENTS SET FORTH IN WSDOT MOMENT SLAB DESIGN MEMORANDUM DATED 12/9/2011 AS PREPARED BY THE BRIDGE AND STRUCTURES OFFICE.

THE SELECTION AND USE OF THIS DETAIL, WHILE DESIGNED IN ACCORDANCE WITH GENERALLY ACCEPTED ENGINEERING PRINCIPLES AND PRACTICES, IS THE SOLE RESPONSIBILITY OF THE REGISTERED PROFESSIONAL ENGINEER IN CHARGE OF THE PROJECT.

MOMENT SLAB AND TRAFFIC BARRIER INSTALLATION FOR LEVEL GRADE (0° SLOPE)

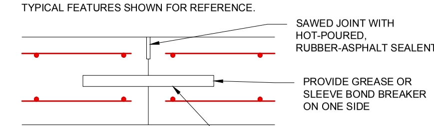


EXPANSION JOINTS SHALL BE PROVIDED IN MOMENT SLAB EVERY 90'-0".
EXPANSION JOINT SHALL BE DOT STANDARD DETAIL.
TYPICAL FEATURES SHOWN FOR REFERENCE.



EXPANSION JOINT

CONTRACTION JOINTS SHALL BE PROVIDED IN MOMENT SLAB EVERY 30'-0" BETWEEN EXPANSION JOINTS.
CONTRACTION JOINT SHALL BE DOT STANDARD DETAIL.
TYPICAL FEATURES SHOWN FOR REFERENCE.



CONTRACTION JOINT

MATERIALS
CONCRETE FOR CAST-IN-PLACE MOMENT SLAB SHALL BE DOT STANDARD STRUCTURE MIX. MINIMUM 28 DAY COMPRESSIVE STRENGTH SHALL BE 4,000 PSI OR HIGHER AS SPECIFIED.
REINFORCING STEEL SHALL CONFORM TO ASTM A706 OR AASHTO M31 GRADE 60.
ALL REINFORCING STEEL SHALL BE EPOXY COATED IN ACCORDANCE WITH ASTM A775 OR ASTM A934.

DESIGN
MOMENT SLAB SHOWN IS DIMENSIONED BASED ON AN EQUIVALENT STATIC LOAD OF 10,000 LBS PER NCHRP REPORT 663.
MOMENT SLAB REINFORCEMENT SHOWN IS BASED ON AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 5TH EDITION, 2010, TL-4 LOADING DETAILED IN TABLE A13.2.1. AND REQUIREMENTS SET FORTH IN WSDOT MOMENT SLAB DESIGN MEMORANDUM DATED 12/9/2011 AS PREPARED BY THE BRIDGE AND STRUCTURES OFFICE.

THE SELECTION AND USE OF THIS DETAIL, WHILE DESIGNED IN ACCORDANCE WITH GENERALLY ACCEPTED ENGINEERING PRINCIPLES AND PRACTICES, IS THE SOLE RESPONSIBILITY OF THE REGISTERED PROFESSIONAL ENGINEER IN CHARGE OF THE PROJECT.

DRAWN BY: C. HINES 08-28-12	TITLE: REDI-ROCK PC SERIES TRAFFIC BARRIER & MOMENT SLAB
REVISED BY: C. HINES 05-07-15	
DATE: AUGUST 28, 2012	
SHEET: 5 OF 17	FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg

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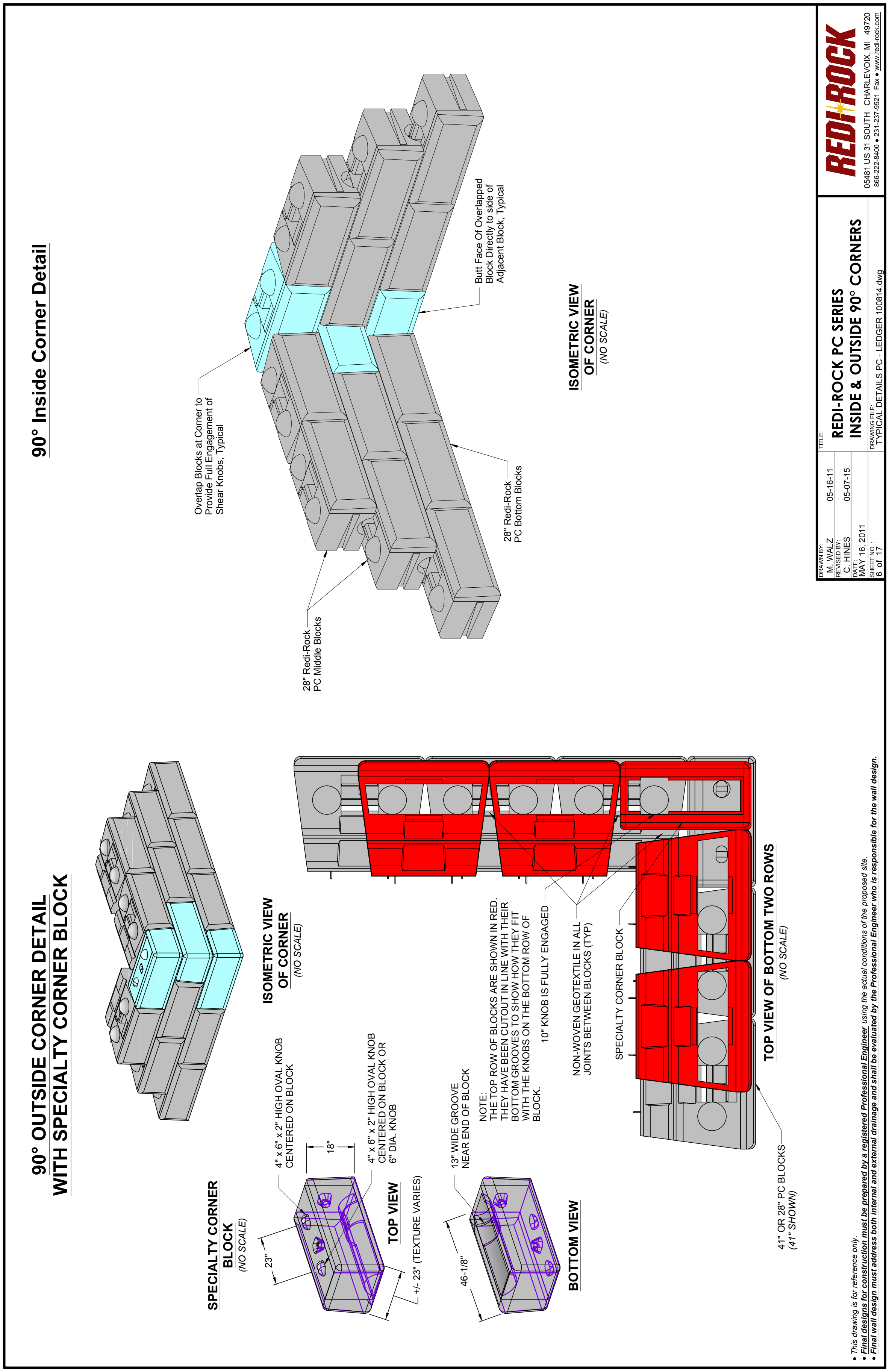
90° OUTSIDE CORNER DETAIL
WITH SPECIALTY CORNER BLOCK
SPECIALTY CORNER BLOCK
(NO SCALE)
23"
+/- 23" (TEXTURE VARIES)
4" x 6" x 2" HIGH OVAL KNOB CENTERED ON BLOCK
18"
4" x 6" x 2" HIGH OVAL KNOB CENTERED ON BLOCK OR 6" DIA. KNOB
TOP VIEW
46-1/8"
13" WIDE GROOVE NEAR END OF BLOCK
BOTTOM VIEW
ISOMETRIC VIEW OF CORNER
(NO SCALE)
NOTE: THE TOP ROW OF BLOCKS ARE SHOWN IN RED. THEY HAVE BEEN CUTOUT IN LINE WITH THEIR BOTTOM GROOVES TO SHOW HOW THEY FIT WITH THE KNOBS ON THE BOTTOM ROW OF BLOCK.
10" KNOB IS FULLY ENGAGED
NON-WOVEN GEOTEXTILE IN ALL JOINTS BETWEEN BLOCKS (TYP)
SPECIALTY CORNER BLOCK
41" OR 28" PC BLOCKS (41" SHOWN)
TOP VIEW OF BOTTOM TWO ROWS
(NO SCALE)
90° Inside Corner Detail
Overlap Blocks at Corner to Provide Full Engagement of Shear Knobs, Typical
28" Redi-Rock PC Middle Blocks
28" Redi-Rock PC Bottom Blocks
Butt Face Of Overlapped Block Directly to side of Adjacent Block, Typical
ISOMETRIC VIEW OF CORNER
(NO SCALE)
• This drawing is for reference only.
• Final designs for construction must be prepared by a registered Professional Engineer using the actual conditions of the proposed site.
• Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.
DRAWN BY: M. WALZ 05-16-11
REVISED BY: C. HINES 05-07-15
DATE: MAY 16, 2011
SHEET NO.: 6 of 17
TITLE: REDI-ROCK PC SERIES INSIDE & OUTSIDE 90° CORNERS
DRAWING FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg
REDI-ROCK
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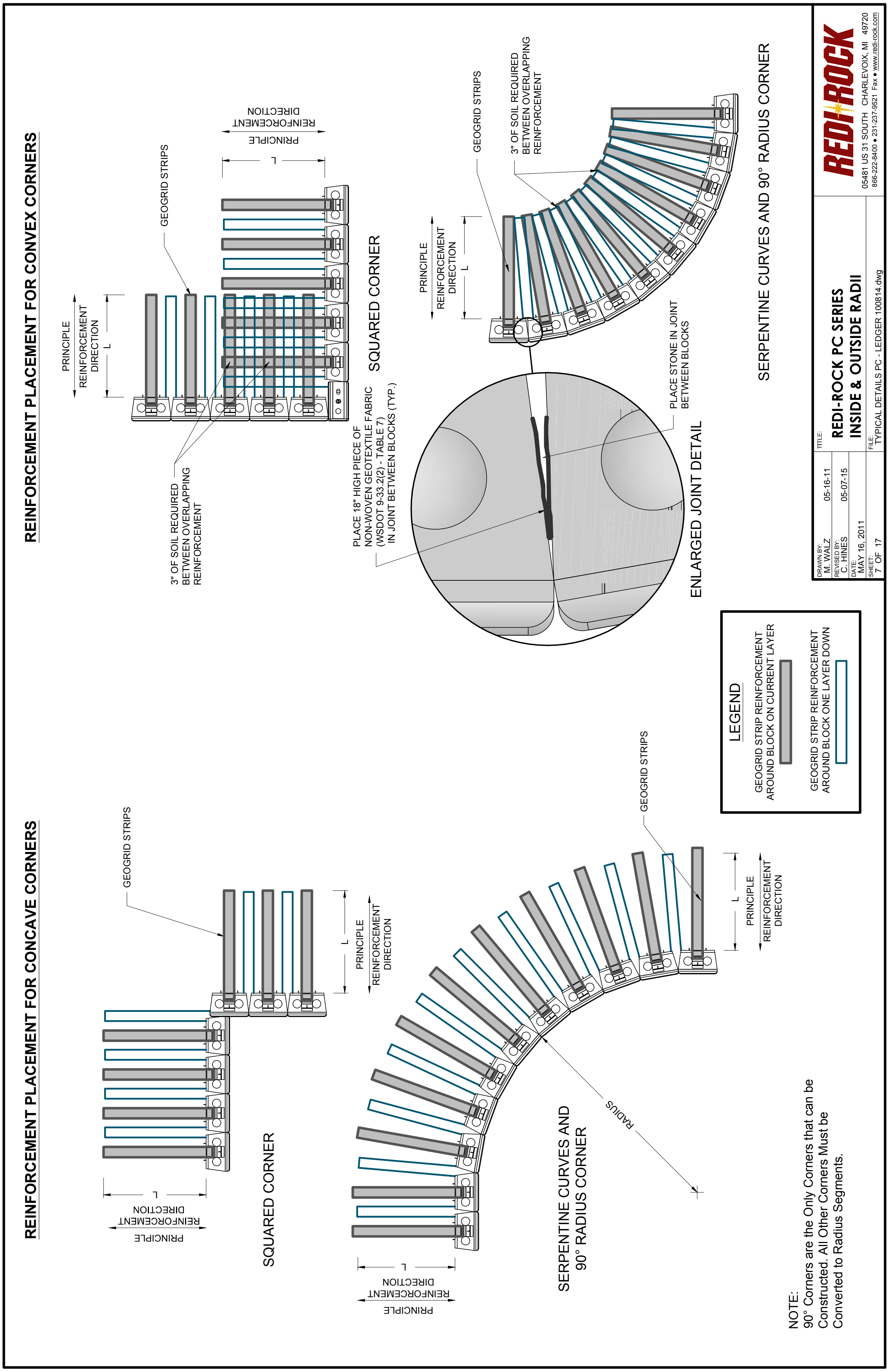
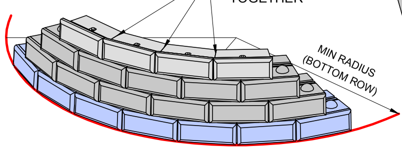
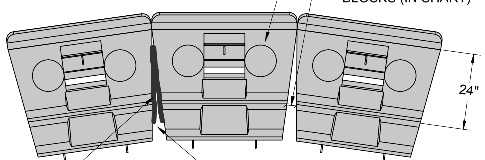

REINFORCEMENT PLACEMENT FOR CONCAVE CORNERS
GEOGRID STRIPS
PRINCIPLE REINFORCEMENT DIRECTION
L
SQUARED CORNER
SERPENTINE CURVES AND 90° RADIUS CORNER
RADIUS
NOTE:
90° Corners are the Only Corners that can be Constructed. All Other Corners Must be Converted to Radius Segments.
REINFORCEMENT PLACEMENT FOR CONVEX CORNERS
3" OF SOIL REQUIRED BETWEEN OVERLAPPING REINFORCEMENT
PLACE 18" HIGH PIECE OF NON-WOVEN GEOTEXTILE FABRIC (WSDOT 9-33.2(2) - TABLE 7) IN JOINT BETWEEN BLOCKS (TYP.)
ENLARGED JOINT DETAIL
PLACE STONE IN JOINT BETWEEN BLOCKS
LEGEND
GEOGRID STRIP REINFORCEMENT AROUND BLOCK ON CURRENT LAYER
GEOGRID STRIP REINFORCEMENT AROUND BLOCK ONE LAYER DOWN
DRAWN BY: M. WALZ 05-16-11
REVISED BY: C. HINES 05-07-15
DATE: MAY 16, 2011
SHEET: 7 OF 17
TITLE: REDI-ROCK PC SERIES INSIDE & OUTSIDE RADII
FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg
REDI-ROCK
05481 US 31 SOUTH CHARLEVOIX, MI 49720
866-222-8400 • 231-237-9521 Fax • www.redi-rock.com

CONVEX CURVES

NUMBER OF COURSES	HEIGHT OF BLOCKS	BOTTOM ROW MIN. RADIUS (FROM FACE OF BLOCK)	DISTANCE BETWEEN BLOCKS PER SKETCH
1	1'-6"	14'-6"	0.13"
2	3'-0"	14'-8"	0.21"
3	4'-6"	14'-10"	0.28"
4	6'-0"	15'-0"	0.36"
5	7'-6"	15'-2"	0.43"
6	9'-0"	15'-4"	0.50"
7	10'-6"	15'-6"	0.57"
8	12'-0"	15'-8"	0.63"
9	13'-6"	15'-10"	0.70"
10	15'-0"	16'-0"	0.76"
11	16'-6"	16'-2"	0.83"
12	18'-0"	16'-4"	0.88"
13	19'-6"	16'-6"	0.95"
14	21'-0"	16'-8"	1.01"



ISOMETRIC VIEW

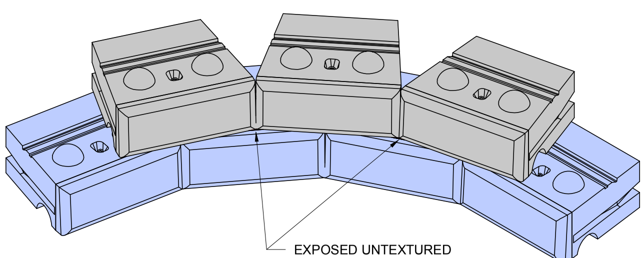


TOP VIEW

CONCAVE CURVES

NOTES:

- PLACE BLOCKS TIGHT TOGETHER
- NO MINIMUM RADIUS - BASED ONLY ON BLOCK GEOMETRY
- SMALLER RADII WILL RESULT IN MORE EXPOSED UNTEXTURED BLOCK SURFACE

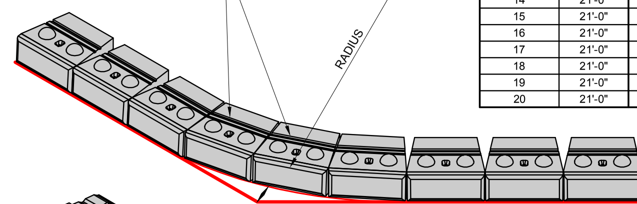


45° OUTSIDE CORNER RADIAL SOLUTION

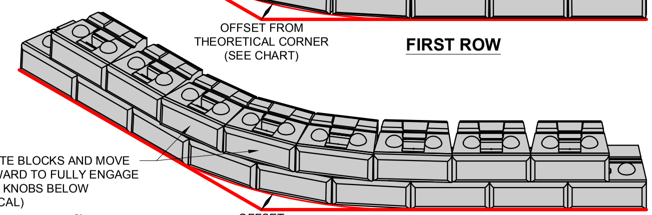
(REDI-ROCK PC BLOCKS)

MINIMUM RADIUS AND OFFSET FOR BOTTOM ROW

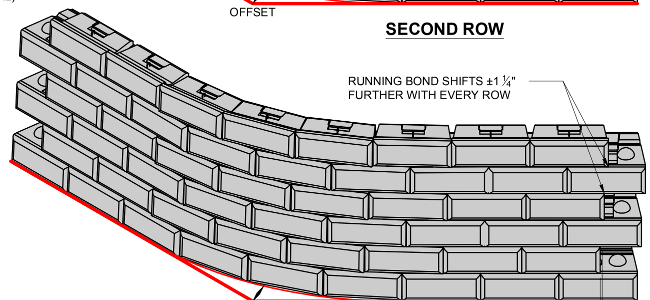
NUMBER OF COURSES	HEIGHT OF BLOCKS	RADIUS FROM FACE OF BLOCK	OFFSET
1	1'-6"	14'-6"	± 14 3/8"
2	3'-0"	14'-8"	± 14 1/2"
3	4'-6"	14'-10"	± 14 5/8"
4	6'-0"	15'-0"	± 14 7/8"
5	7'-6"	15'-2"	± 15"
6	9'-0"	15'-4"	± 15 1/8"
7	10'-6"	15'-6"	± 15 3/8"
8	12'-0"	15'-8"	± 15 1/2"
9	13'-6"	15'-10"	± 15 5/8"
10	15'-0"	16'-0"	± 15 7/8"
11	16'-6"	16'-2"	± 16"
12	18'-0"	16'-4"	± 16 1/8"
13	19'-6"	16'-6"	± 16 3/8"
14	21'-0"	16'-8"	± 16 1/2"
15	21'-0"	16'-8"	± 16 1/2"
16	21'-0"	16'-8"	± 16 1/2"
17	21'-0"	16'-8"	± 16 1/2"
18	21'-0"	16'-8"	± 16 1/2"
19	21'-0"	16'-8"	± 16 1/2"
20	21'-0"	16'-8"	± 16 1/2"



FIRST ROW

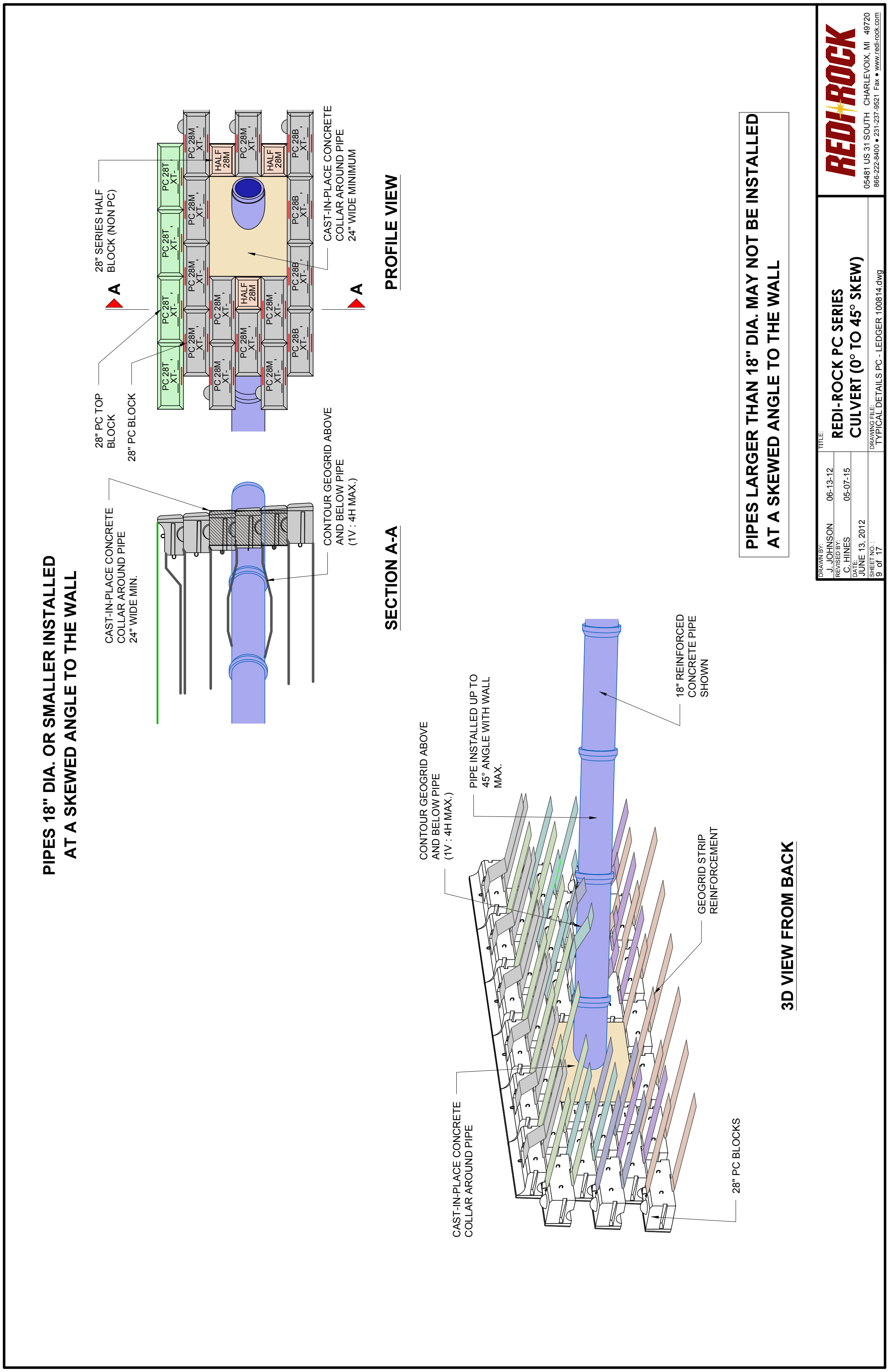


SECOND ROW



COMPLETED CORNER

DRAWN BY: M. WALZ 05-16-11 REVISED BY: C. HINES 05-07-15 DATE: MAY 16, 2011 SHEET: 8 OF 17	TITLE: REDI-ROCK PC SERIES OUTSIDE CORNER RADIAL SOLUTION FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg	REDI-ROCK 05481 US 31 SOUTH CHARLEVOIX, MI 49720 866-222-8400 • 231-237-9521 Fax • www.redi-rock.com


PIPES 18" DIA. OR SMALLER INSTALLED AT A SKEWED ANGLE TO THE WALL
28" SERIES HALF BLOCK (NON PC)
28" PC TOP BLOCK
28" PC BLOCK
CAST-IN-PLACE CONCRETE COLLAR AROUND PIPE 24" WIDE MINIMUM
PROFILE VIEW
CAST-IN-PLACE CONCRETE COLLAR AROUND PIPE 24" WIDE MIN.
CONTOUR GEOGRID ABOVE AND BELOW PIPE (1V : 4H MAX.)
SECTION A-A
PIPE INSTALLED UP TO 45° ANGLE WITH WALL MAX.
18" REINFORCED CONCRETE PIPE SHOWN
GEOGRID STRIP REINFORCEMENT
CAST-IN-PLACE CONCRETE COLLAR AROUND PIPE
28" PC BLOCKS
3D VIEW FROM BACK
PIPES LARGER THAN 18" DIA. MAY NOT BE INSTALLED AT A SKEWED ANGLE TO THE WALL
DRAWN BY: J. JOHNSON 06-13-12
REVISED BY: C. HINES 05-07-15
DATE: JUNE 13, 2012
SHEET NO.: 9 of 17
TITLE: REDI-ROCK PC SERIES CULVERT (0° TO 45° SKEW)
DRAWING FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg
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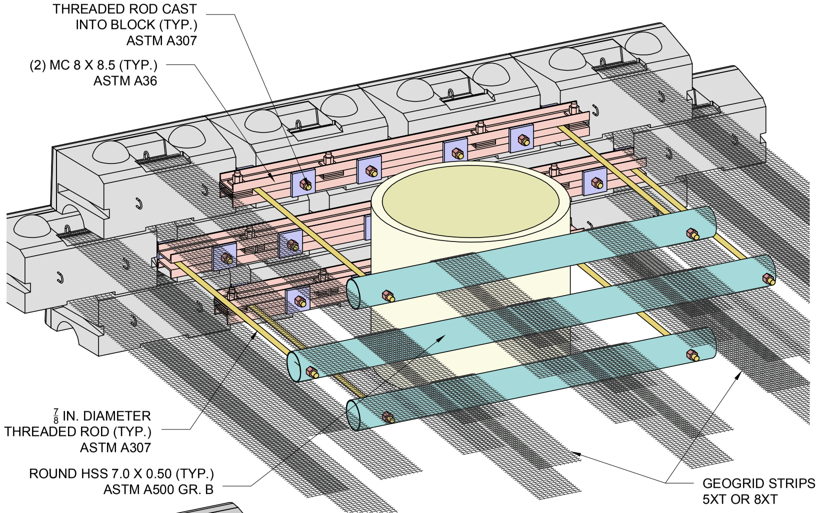
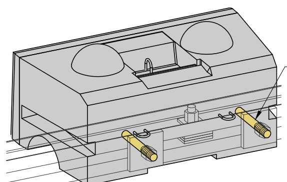
REINFORCEMENT PLACEMENT AROUND VERTICAL OBSTRUCTIONS

With the Redi-Rock PC Series

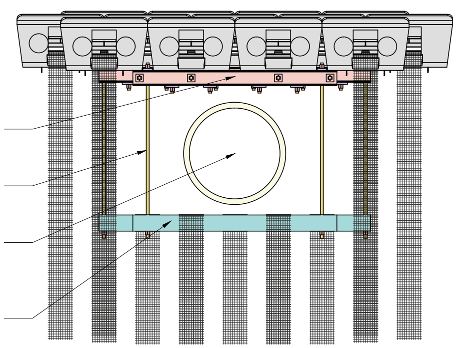
LARGE OBSTRUCTION - CONCEPTUAL DETAIL



BACK VIEW

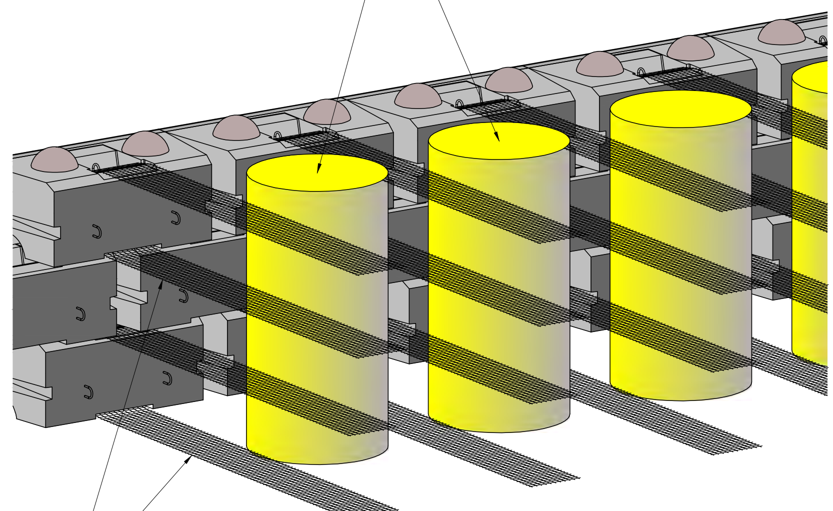


BLOCK DETAIL



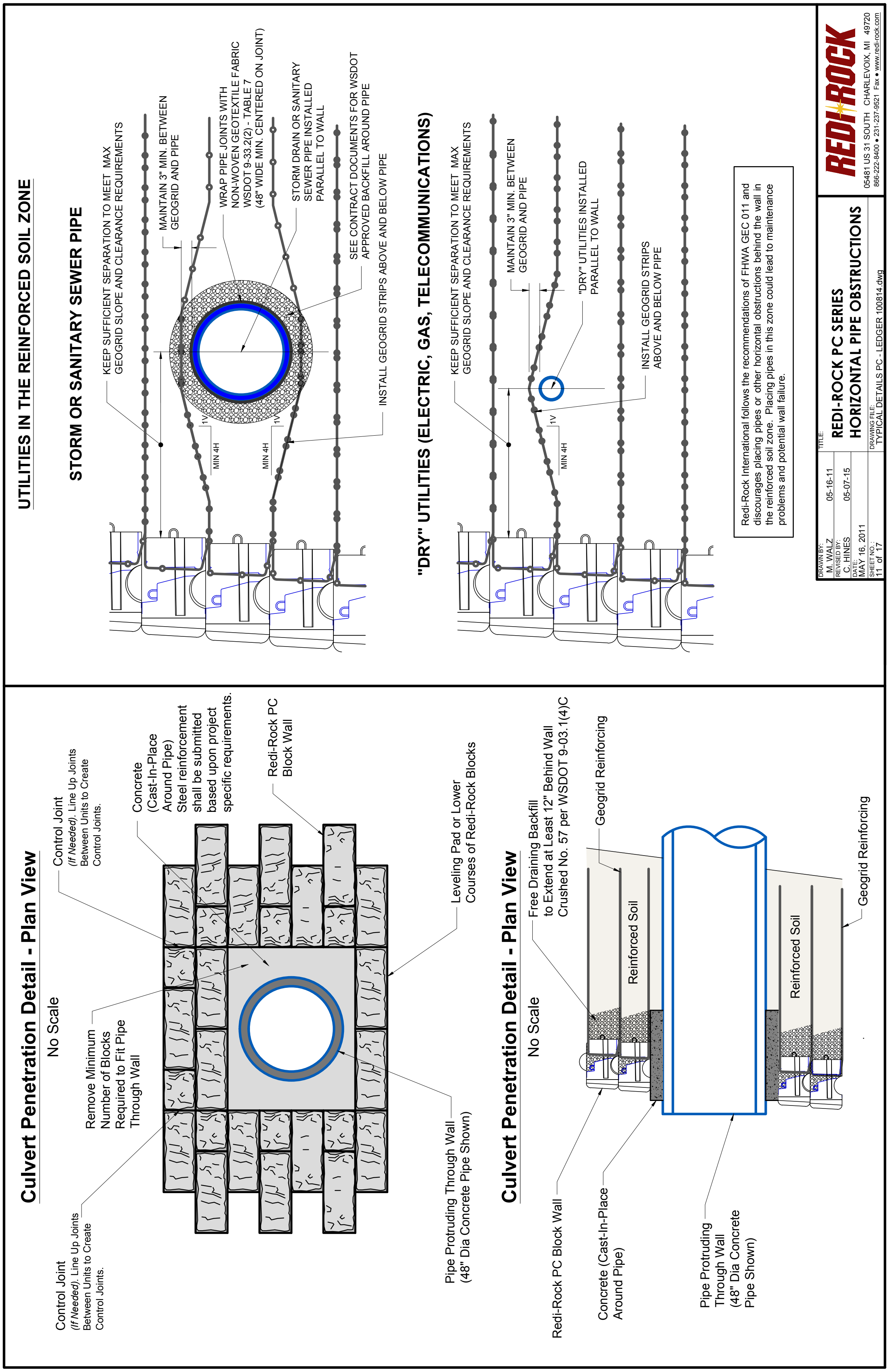
TOP VIEW

NOTE:
(1) ALL STRUCTURAL STEEL ELEMENTS TO BE HOT-DIPPED GALVANIZED IN ACCORDANCE WITH ASTM A123 COATING THICKNESS GRADE 85.
(2) THE ABOVE DETAIL IS VALID ONLY FOR OBSTRUCTIONS 84" AND SMALLER AND/OR PC WALL SECTIONS THAT DO NOT REQUIRE GEOGRID REINFORCEMENT WITH TENSILE STRENGTH HIGHER THAN 8XT.



ISOMETRIC VIEW OF BACK OF BLOCKS
No Scale

DRAWN BY: M. WALZ	05-16-11	TITLE: REDI-ROCK PC SERIES VERTICAL OBSTRUCTIONS	REDI-ROCK
REVISED BY: C. HINES	05-07-15		05481 US 31 SOUTH CHARLEVOIX, MI 49720
DATE: MAY 16, 2011			866-222-8400 • 231-237-9521 Fax • www.redi-rock.com
SHEET NO.: 10 of 17		DRAWING FILE: TYPICAL DETAILS PC - LEDGER100814.dwg	


UTILITIES IN THE REINFORCED SOIL ZONE
STORM OR SANITARY SEWER PIPE
KEEP SUFFICIENT SEPARATION TO MEET MAX GEOGRID SLOPE AND CLEARANCE REQUIREMENTS
MAINTAIN 3" MIN. BETWEEN GEOGRID AND PIPE
WRAP PIPE JOINTS WITH NON-WOVEN GEOTEXTILE FABRIC WSDOT 9-33.2(2) - TABLE 7 (48" WIDE MIN. CENTERED ON JOINT)
STORM DRAIN OR SANITARY SEWER PIPE INSTALLED PARALLEL TO WALL
SEE CONTRACT DOCUMENTS FOR WSDOT APPROVED BACKFILL AROUND PIPE
INSTALL GEOGRID STRIPS ABOVE AND BELOW PIPE
1V
MIN 4H
1V
MIN 4H
"DRY" UTILITIES (ELECTRIC, GAS, TELECOMMUNICATIONS)
KEEP SUFFICIENT SEPARATION TO MEET MAX GEOGRID SLOPE AND CLEARANCE REQUIREMENTS
MAINTAIN 3" MIN. BETWEEN GEOGRID AND PIPE
"DRY" UTILITIES INSTALLED PARALLEL TO WALL
INSTALL GEOGRID STRIPS ABOVE AND BELOW PIPE
1V
MIN 4H
Redi-Rock International follows the recommendations of FHWA GEC 011 and discourages placing pipes or other horizontal obstructions behind the wall in the reinforced soil zone. Placing pipes in this zone could lead to maintenance problems and potential wall failure.
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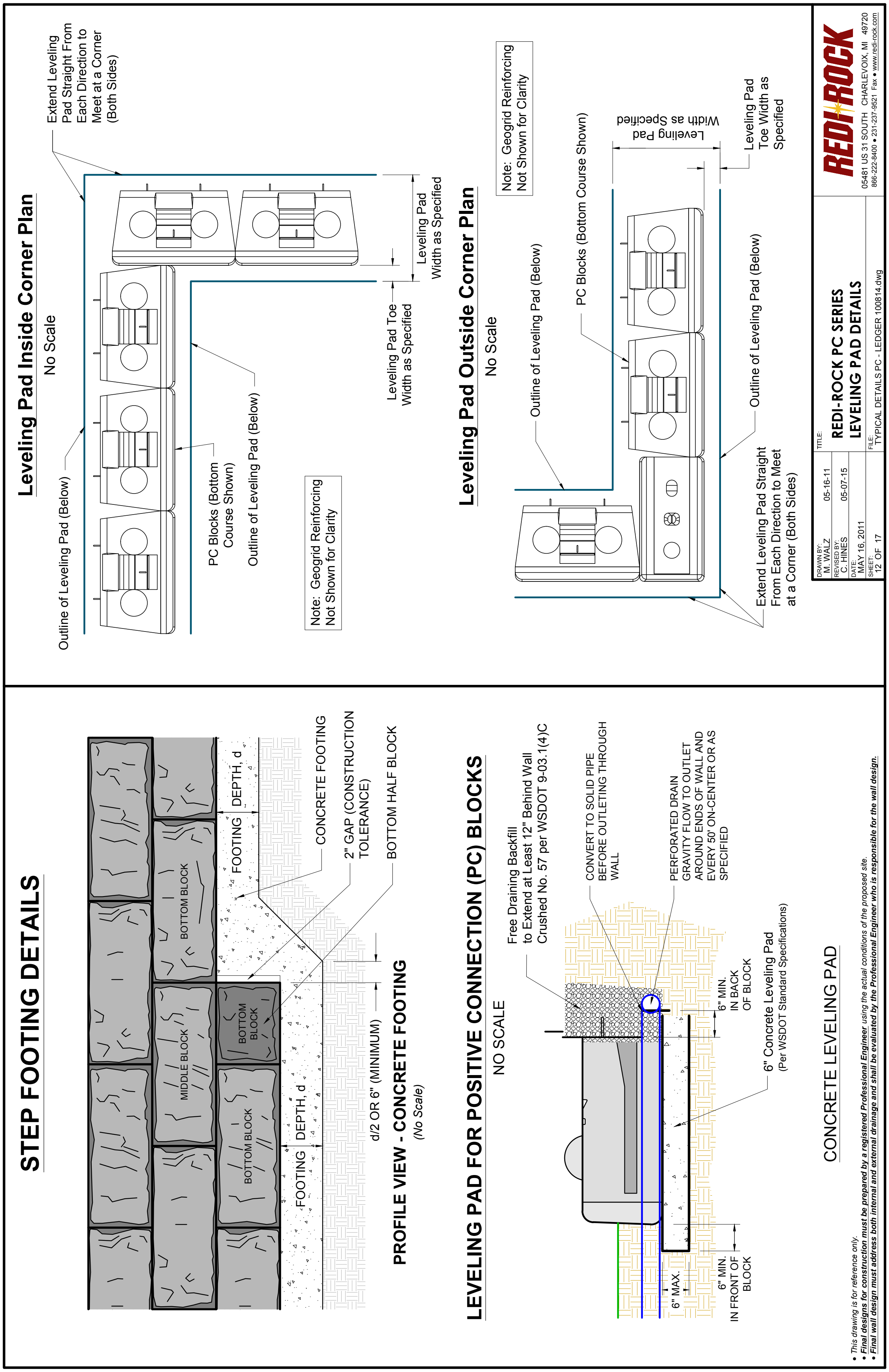
866-222-8400 • 231-237-9521 Fax • www.redi-rock.com
DRAWN BY: M. WALZ 05-16-11
REVISED BY: C. HINES 05-07-15
DATE: MAY 16, 2011
SHEET NO.: 11 of 17
TITLE: REDI-ROCK PC SERIES HORIZONTAL PIPE OBSTRUCTIONS
DRAWING FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg
Culvert Penetration Detail - Plan View
No Scale
Control Joint (If Needed). Line Up Joints Between Units to Create Control Joints.
Concrete (Cast-In-Place Around Pipe) Steel reinforcement shall be submitted based upon project specific requirements.
Redi-Rock PC Block Wall
Remove Minimum Number of Blocks Required to Fit Pipe Through Wall
Control Joint (If Needed). Line Up Joints Between Units to Create Control Joints.
Leveling Pad or Lower Courses of Redi-Rock Blocks
Pipe Protruding Through Wall (48" Dia Concrete Pipe Shown)
Culvert Penetration Detail - Plan View
No Scale
Free Draining Backfill to Extend at Least 12" Behind Wall Crushed No. 57 per WSDOT 9-03.1(4)C
Geogrid Reinforcing
Reinforced Soil
Reinforced Soil
Geogrid Reinforcing
Redi-Rock PC Block Wall
Concrete (Cast-In-Place Around Pipe)
Pipe Protruding Through Wall (48" Dia Concrete Pipe Shown)



STEP FOOTING DETAILS
BOTTOM BLOCK
MIDDLE BLOCK
BOTTOM BLOCK
BOTTOM BLOCK
FOOTING DEPTH, d
FOOTING DEPTH, d
CONCRETE FOOTING
2" GAP (CONSTRUCTION TOLERANCE)
BOTTOM HALF BLOCK
d/2 OR 6" (MINIMUM)
PROFILE VIEW - CONCRETE FOOTING
(No Scale)
LEVELING PAD FOR POSITIVE CONNECTION (PC) BLOCKS
NO SCALE
Free Draining Backfill to Extend at Least 12" Behind Wall Crushed No. 57 per WSDOT 9-03.1(4)C
CONVERT TO SOLID PIPE BEFORE OUTLETING THROUGH WALL
PERFORATED DRAIN GRAVITY FLOW TO OUTLET AROUND ENDS OF WALL AND EVERY 50' ON-CENTER OR AS SPECIFIED
6" MIN. IN BACK OF BLOCK
6" Concrete Leveling Pad
(Per WSDOT Standard Specifications)
6" MAX.
6" MIN. IN FRONT OF BLOCK
CONCRETE LEVELING PAD
Leveling Pad Inside Corner Plan
No Scale
Outline of Leveling Pad (Below)
Extend Leveling Pad Straight From Each Direction to Meet at a Corner (Both Sides)
PC Blocks (Bottom Course Shown)
Outline of Leveling Pad (Below)
Note: Geogrid Reinforcing Not Shown for Clarity
Leveling Pad Toe Width as Specified
Leveling Pad Width as Specified
Leveling Pad Outside Corner Plan
No Scale
Note: Geogrid Reinforcing Not Shown for Clarity
Outline of Leveling Pad (Below)
PC Blocks (Bottom Course Shown)
Leveling Pad Width as Specified
Leveling Pad Toe Width as Specified
Outline of Leveling Pad (Below)
Extend Leveling Pad Straight From Each Direction to Meet at a Corner (Both Sides)
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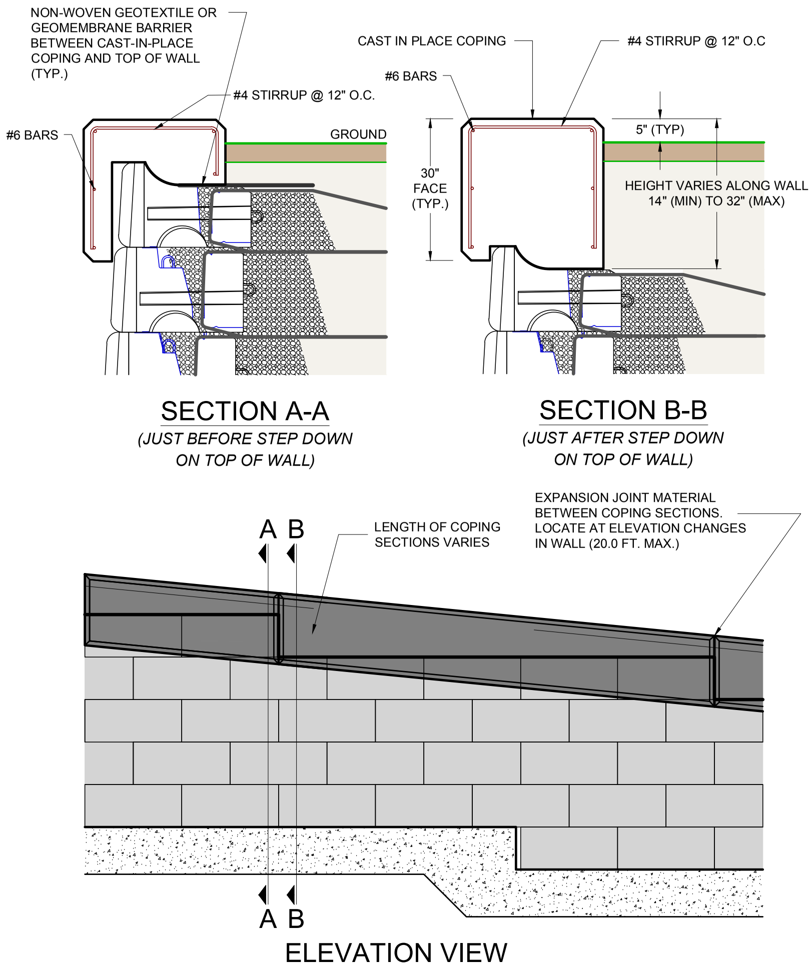
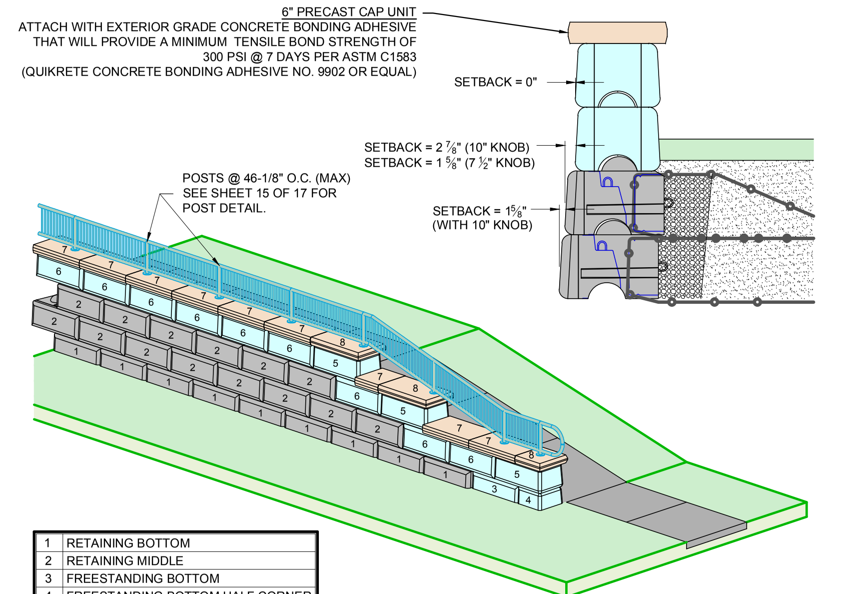
TITLE:
REDI-ROCK PC SERIES
LEVELING PAD DETAILS
FILE:
TYPICAL DETAILS PC - LEDGER 100814.dwg
DRAWN BY: M. WALZ 05-16-11
REVISED BY: C. HINES 05-07-15
DATE: MAY 16, 2011
SHEET: 12 OF 17
• This drawing is for reference only.
• Final designs for construction must be prepared by a registered Professional Engineer using the actual conditions of the proposed site.
• Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.

CAST-IN-PLACE COPING DETAIL

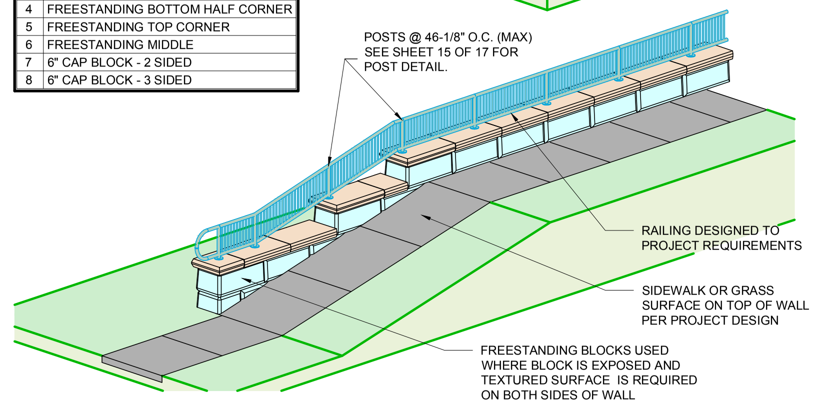
NO SCALE



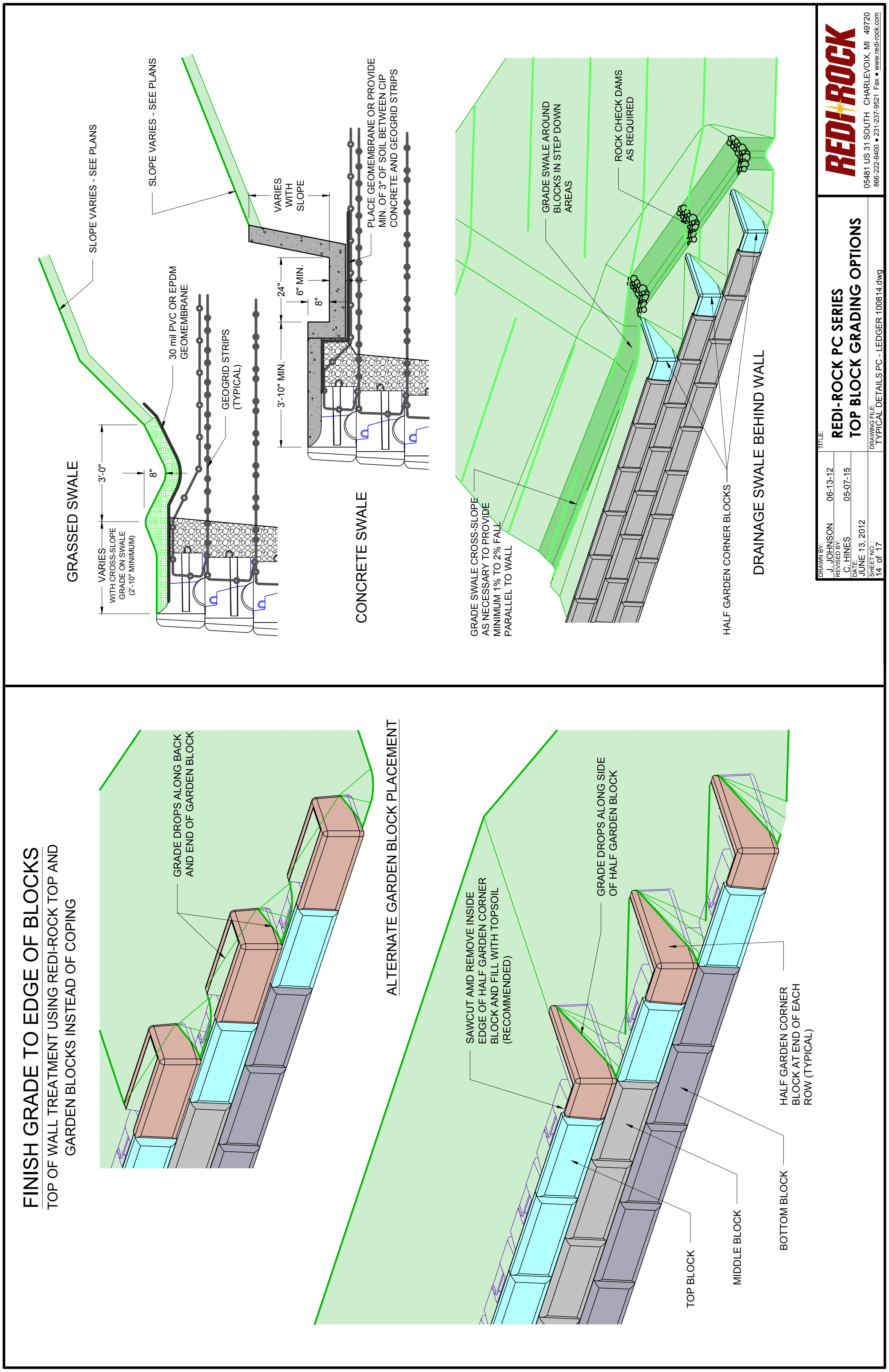
- *This drawing is for reference only.*
- ***Final designs for construction must be prepared by a registered Professional Engineer** using the actual conditions of the proposed site.*
- ***Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.***



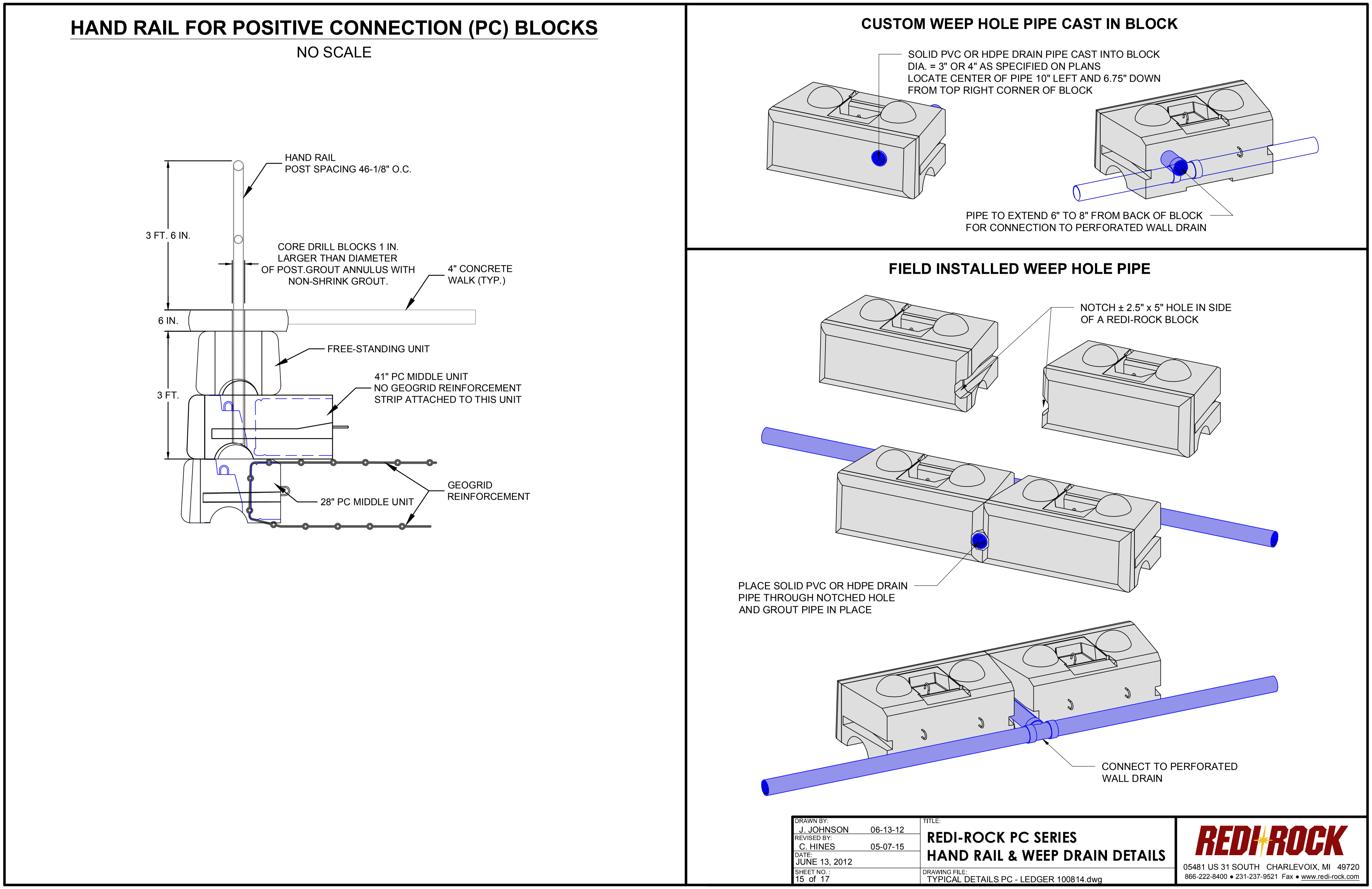
1	RETAINING BOTTOM
2	RETAINING MIDDLE
3	FREESTANDING BOTTOM
4	FREESTANDING BOTTOM HALF CORNER
5	FREESTANDING TOP CORNER
6	FREESTANDING MIDDLE
7	6" CAP BLOCK - 2 SIDED
8	6" CAP BLOCK - 3 SIDED



DRAWN BY: J. JOHNSON 06-13-12	TITLE: **REDI-ROCK PC SERIES COPING & CAP OPTIONS**	REDI-ROCK
REVISED BY: C. HINES 05-07-15		05481 US 31 SOUTH CHARLEVOIX, MI 49720
DATE: JUNE 13, 2012		866-222-8400 • 231-237-9521 Fax • www.redi-rock.com
SHEET: 13 OF 17	FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg	



GRASSED SWALE
SLOPE VARIES - SEE PLANS
30 mil PVC OR EPDM GEOMEMBRANE
GEOGRID STRIPS (TYPICAL)
VARIES WITH CROSS-SLOPE GRADE ON SWALE (2'-10" MINIMUM)
3'-0"
8"
CONCRETE SWALE
SLOPE VARIES - SEE PLANS
VARIES WITH SLOPE
PLACE GEOMEMBRANE OR PROVIDE MIN. OF 3" OF SOIL BETWEEN CIP CONCRETE AND GEOGRID STRIPS
24"
6" MIN.
8"
3'-10" MIN.
DRAINAGE SWALE BEHIND WALL
GRADE SWALE CROSS-SLOPE AS NECESSARY TO PROVIDE MINIMUM 1% TO 2% FALL PARALLEL TO WALL
GRADE SWALE AROUND BLOCKS IN STEP DOWN AREAS
ROCK CHECK DAMS AS REQUIRED
HALF GARDEN CORNER BLOCKS
FINISH GRADE TO EDGE OF BLOCKS
TOP OF WALL TREATMENT USING REDI-ROCK TOP AND GARDEN BLOCKS INSTEAD OF COPING
GRADE DROPS ALONG BACK AND END OF GARDEN BLOCK
ALTERNATE GARDEN BLOCK PLACEMENT
SAWCUT AMD REMOVE INSIDE EDGE OF HALF GARDEN CORNER BLOCK AND FILL WITH TOPSOIL (RECOMMENDED)
GRADE DROPS ALONG SIDE OF HALF GARDEN BLOCK
HALF GARDEN CORNER BLOCK AT END OF EACH ROW (TYPICAL)
TOP BLOCK
MIDDLE BLOCK
BOTTOM BLOCK
REDI-ROCK
05481 US 31 SOUTH CHARLEVOIX, MI 49720
866-222-8400 • 231-237-9521 Fax • www.redi-rock.com
TITLE:
REDI-ROCK PC SERIES
TOP BLOCK GRADING OPTIONS
DRAWING FILE:
TYPICAL DETAILS PC - LEDGER 100814.dwg
DRAWN BY:
J. JOHNSON 06-13-12
REVISED BY:
C. HINES 05-07-15
DATE:
JUNE 13, 2012
SHEET NO.:
14 of 17


HAND RAIL FOR POSITIVE CONNECTION (PC) BLOCKS
NO SCALE
HAND RAIL
POST SPACING 46-1/8" O.C.
3 FT. 6 IN.
CORE DRILL BLOCKS 1 IN. LARGER THAN DIAMETER OF POST.GROUT ANNULUS WITH NON-SHRINK GROUT.
4" CONCRETE WALK (TYP.)
6 IN.
FREE-STANDING UNIT
3 FT.
41" PC MIDDLE UNIT
NO GEOGRID REINFORCEMENT STRIP ATTACHED TO THIS UNIT
GEOGRID REINFORCEMENT
28" PC MIDDLE UNIT
CUSTOM WEEP HOLE PIPE CAST IN BLOCK
SOLID PVC OR HDPE DRAIN PIPE CAST INTO BLOCK
DIA. = 3" OR 4" AS SPECIFIED ON PLANS
LOCATE CENTER OF PIPE 10" LEFT AND 6.75" DOWN FROM TOP RIGHT CORNER OF BLOCK
PIPE TO EXTEND 6" TO 8" FROM BACK OF BLOCK FOR CONNECTION TO PERFORATED WALL DRAIN
FIELD INSTALLED WEEP HOLE PIPE
NOTCH ± 2.5" x 5" HOLE IN SIDE OF A REDI-ROCK BLOCK
PLACE SOLID PVC OR HDPE DRAIN PIPE THROUGH NOTCHED HOLE AND GROUT PIPE IN PLACE
CONNECT TO PERFORATED WALL DRAIN
DRAWN BY:
J. JOHNSON 06-13-12
REVISED BY:
C. HINES 05-07-15
DATE:
JUNE 13, 2012
SHEET NO.:
15 of 17
TITLE:
REDI-ROCK PC SERIES
HAND RAIL & WEEP DRAIN DETAILS
DRAWING FILE:
TYPICAL DETAILS PC - LEDGER 100814.dwg
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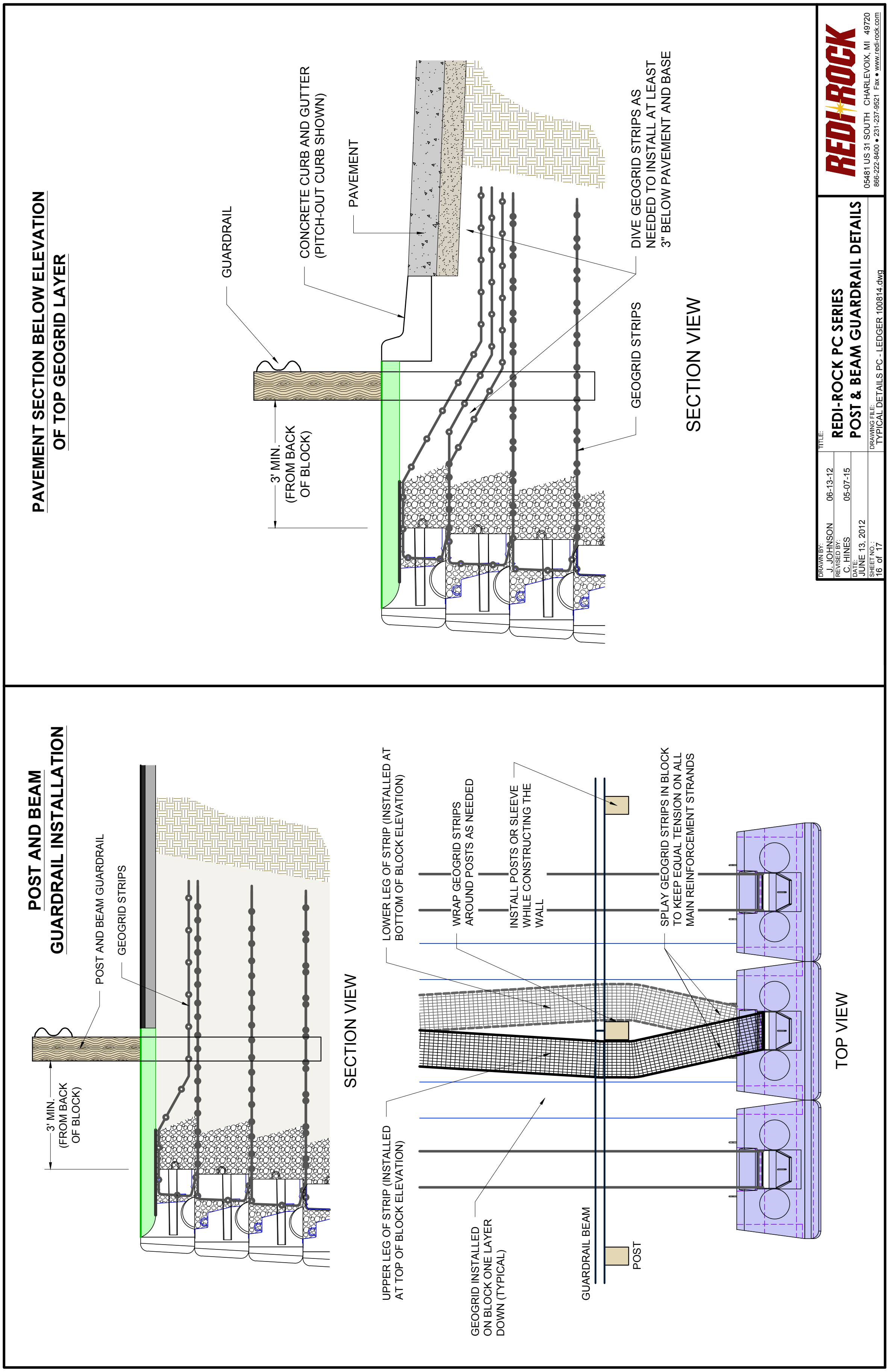
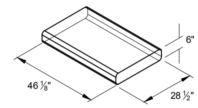
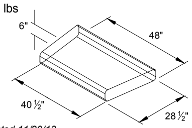

PAVEMENT SECTION BELOW ELEVATION OF TOP GEOGRID LAYER
GUARDRAIL
CONCRETE CURB AND GUTTER (PITCH-OUT CURB SHOWN)
PAVEMENT
3' MIN. (FROM BACK OF BLOCK)
DIVE GEOGRID STRIPS AS NEEDED TO INSTALL AT LEAST 3" BELOW PAVEMENT AND BASE
GEOGRID STRIPS
SECTION VIEW
POST AND BEAM GUARDRAIL INSTALLATION
POST AND BEAM GUARDRAIL
GEOGRID STRIPS
3' MIN. (FROM BACK OF BLOCK)
SECTION VIEW
LOWER LEG OF STRIP (INSTALLED AT BOTTOM OF BLOCK ELEVATION)
WRAP GEOGRID STRIPS AROUND POSTS AS NEEDED
INSTALL POSTS OR SLEEVE WHILE CONSTRUCTING THE WALL
SPLAY GEOGRID STRIPS IN BLOCK TO KEEP EQUAL TENSION ON ALL MAIN REINFORCEMENT STRANDS
UPPER LEG OF STRIP (INSTALLED AT TOP OF BLOCK ELEVATION)
GEOGRID INSTALLED ON BLOCK ONE LAYER DOWN (TYPICAL)
GUARDRAIL BEAM
POST
TOP VIEW
REDI-ROCK
05481 US 31 SOUTH CHARLEVOIX, MI 49720
866-222-8400 • 231-237-9521 Fax • www.redi-rock.com
DRAWN BY: J. JOHNSON 06-13-12
REVISED BY: C. HINES 05-07-15
DATE: JUNE 13, 2012
SHEET NO.: 16 of 17
TITLE: REDI-ROCK PC SERIES POST & BEAM GUARDRAIL DETAILS
DRAWING FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg

REDI-ROCK PC CAP & CORNER UNIT BLOCKS

Two-Sided Cap
Volume = 4.50 cft
Weight = ±644 lbs

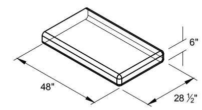


Two-Sided Curve Cap
Volume = 4.25 cft
Weight = ±608 lbs

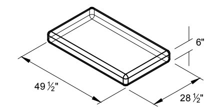


Dimensions Updated 11/20/13

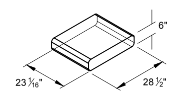
Three-Sided Cap
Volume = 4.68 cft
Weight = ±669 lbs



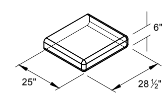
Four-Sided Cap
Volume = 4.81 cft
Weight = ±688 lbs



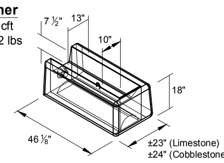
Two-Sided Half
Volume = 2.25 cft
Weight = ±322 lbs



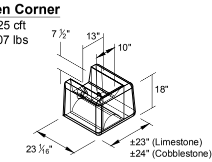
Three-Sided Half
Volume = 2.43 cft
Weight = ±347 lbs



Garden Corner
Volume = 8.26 cft
Weight = ±1182 lbs



Half Garden Corner
Volume = 4.25 cft
Weight = ±607 lbs



NOTES:

Volume is based on the blocks as shown.
Actual weights and volumes may vary.
Weight shown is based on 143 pcf concrete.

DRAWN BY: C. HINES 05-07-15	TITLE: REDI-ROCK PC SERIES CAP AND CORNER COMPONENT DETAILS	REDI-ROCK
REVISED BY:		05481 US 31 SOUTH CHARLEVOIX, MI 49720
DATE: OCTOBER 8, 2014		866-222-8400 • 231-237-9521 Fax • www.redi-rock.com
SHEET NO.: 17 of 17	DRAWING FILE: TYPICAL DETAILS PC - LEDGER 100814.dwg	

Appendix 15-R Preapproved Wall Appendix: Specific Requirements and Details for Lock and Load Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply:

Facing System – The wall shall be designed as a wrapped face wall system. The concrete counterfort that attaches to the facing panel shall penetrate through the geogrid reinforcement by only cutting transverse ribs as necessary to allow the counterfort to connect to the facing panel, as shown in the preapproved plans. The wall facing design shall demonstrate that the facing panel plus counterfort is stable for all limit states in accordance with the AASHTO LRFD Bridge Design Specifications, the *Bridge Design Manual* M 23-50, and the *Geotechnical Design Manual*.

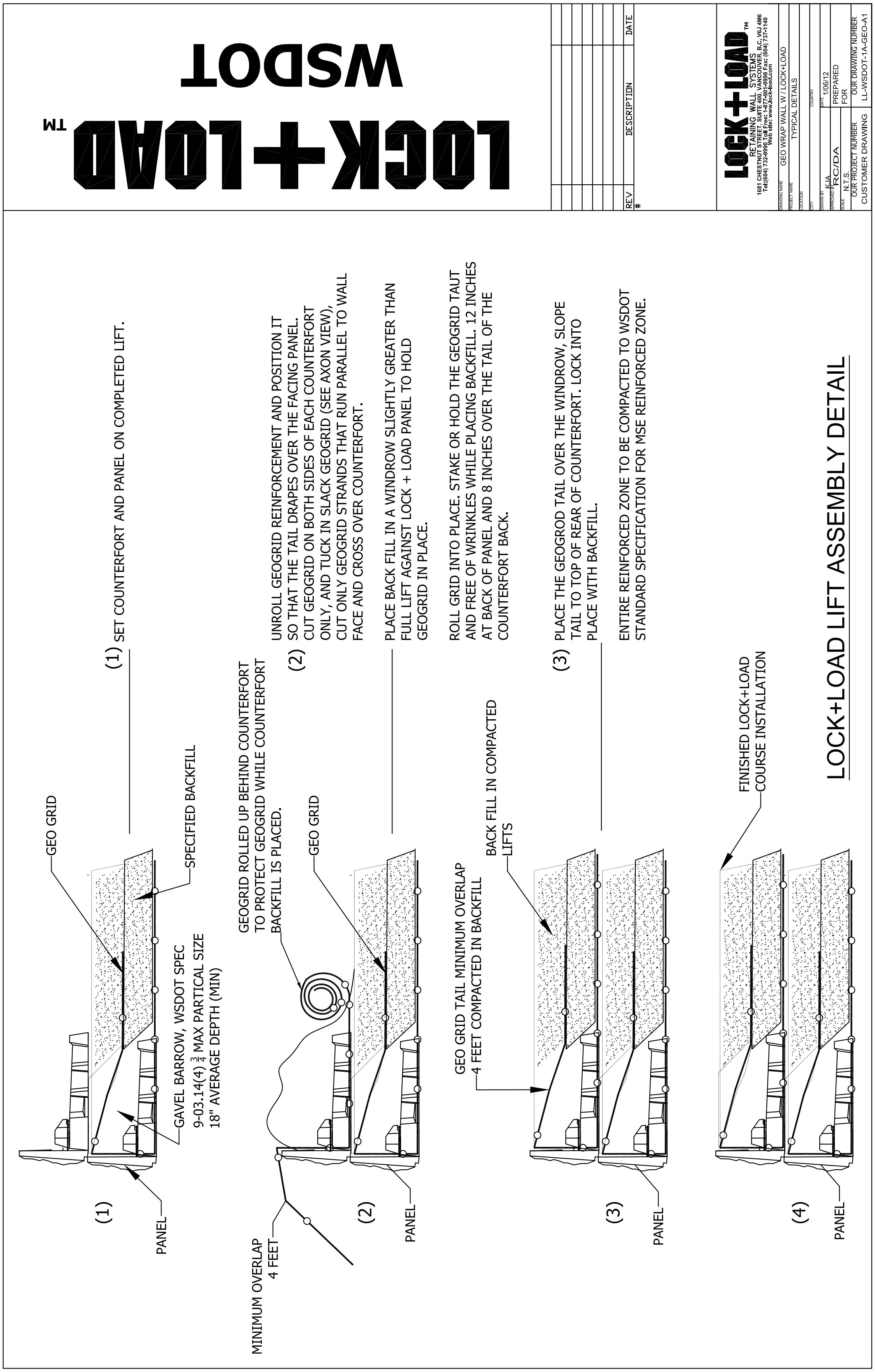
Soil Reinforcement – Only geosynthetic reinforcement listed in the QPL shall be used. The ultimate and long-term design strengths specified in Appendix D of the QPL shall be used.

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications.

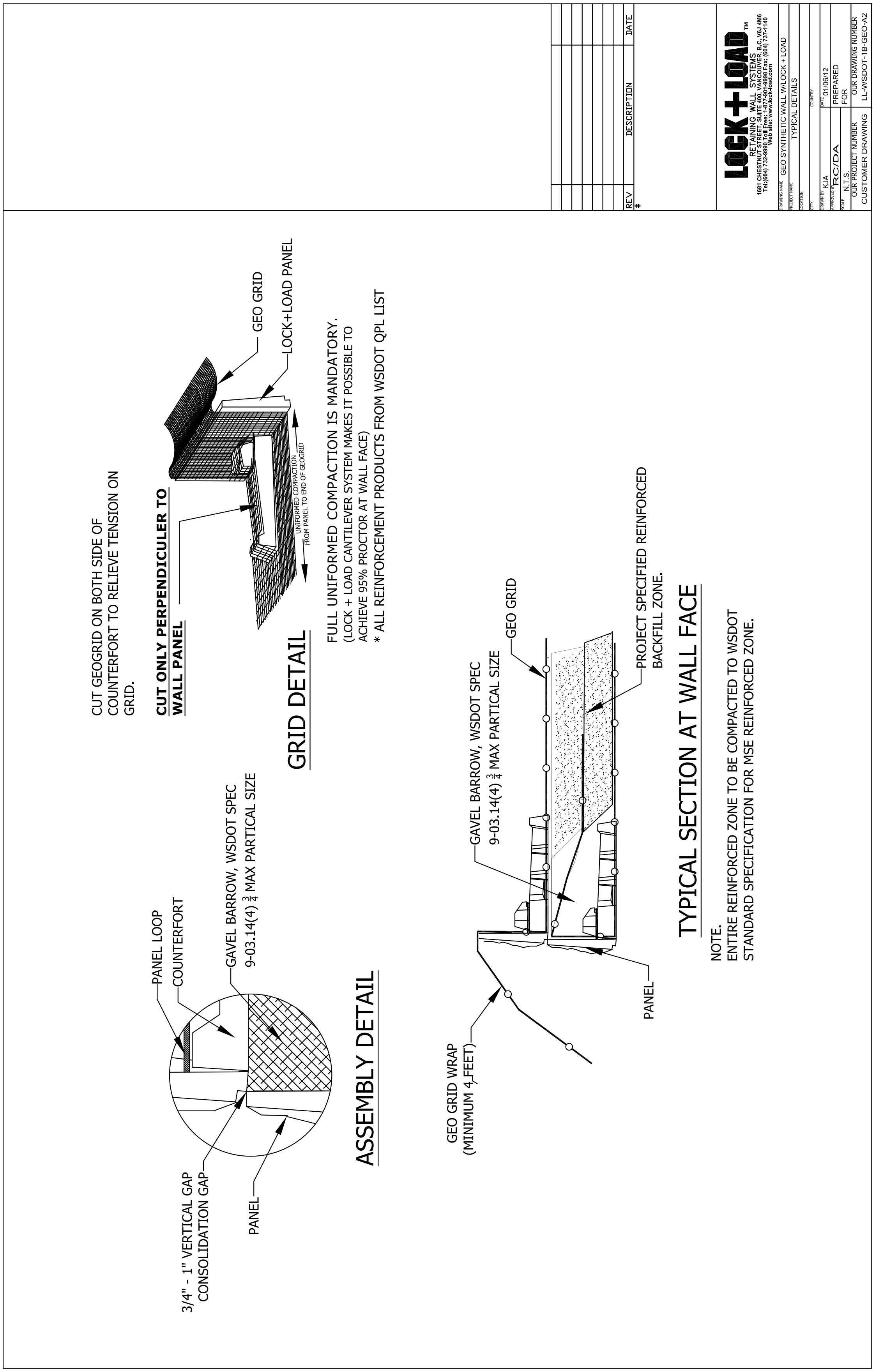
The Lock and Load Wall system shall only be used at locations where the wall will be above the water table.

Approved details for the Lock and Load wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

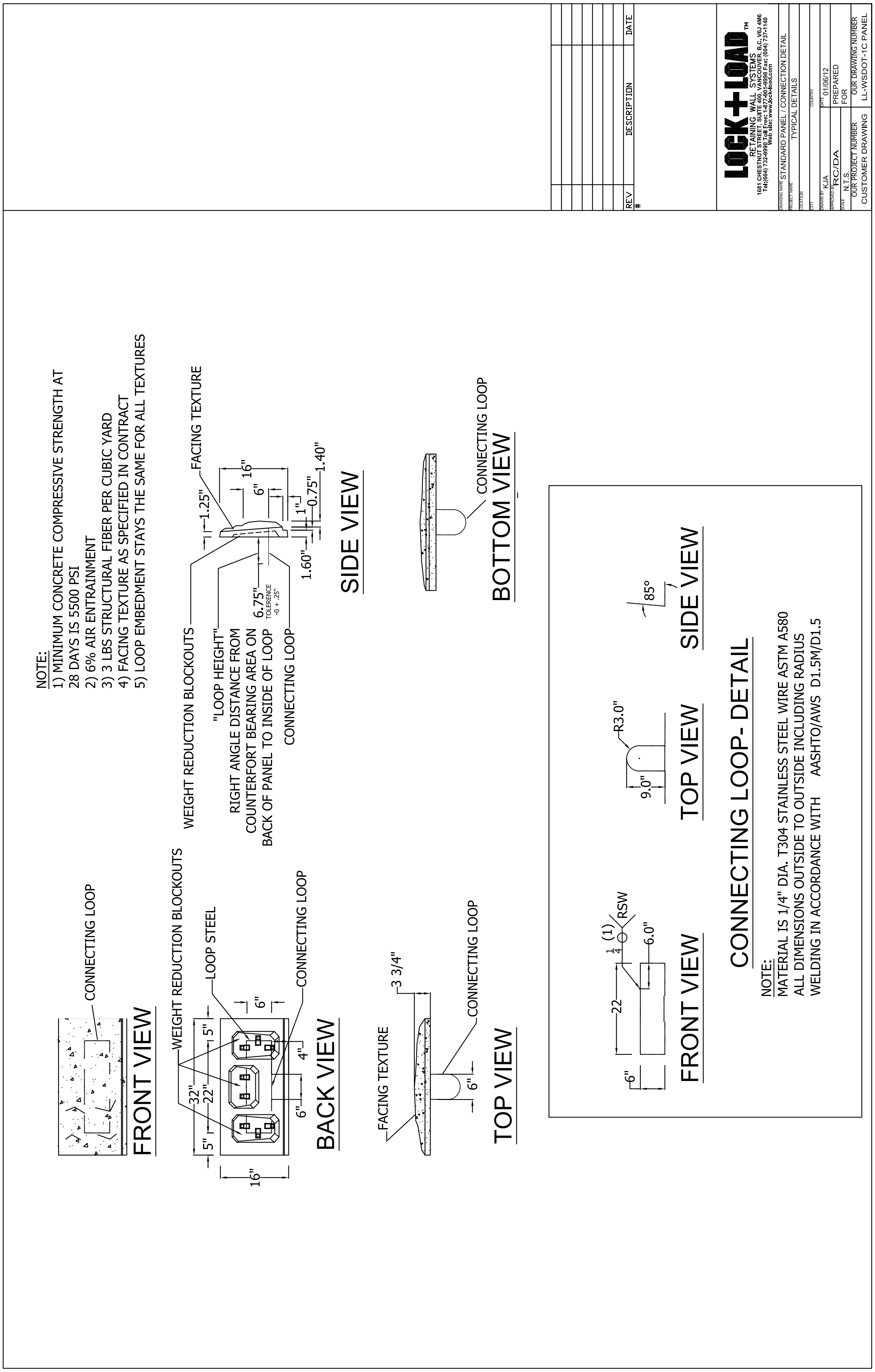
- WSDOT standard materials, including backfill used for the wall, shall be used where possible. With regard to the wall backfill, the entire reinforced zone for the wall shall be backfilled with WSDOT Gravel Borrow, not just the area shown in the plans (i.e., sheet 2). Where "filter fabric" is specified in the preapproved plans, it shall be a WSDOT Standard Specification Construction Geotextile for Underground Drainage material (Section 9-33).

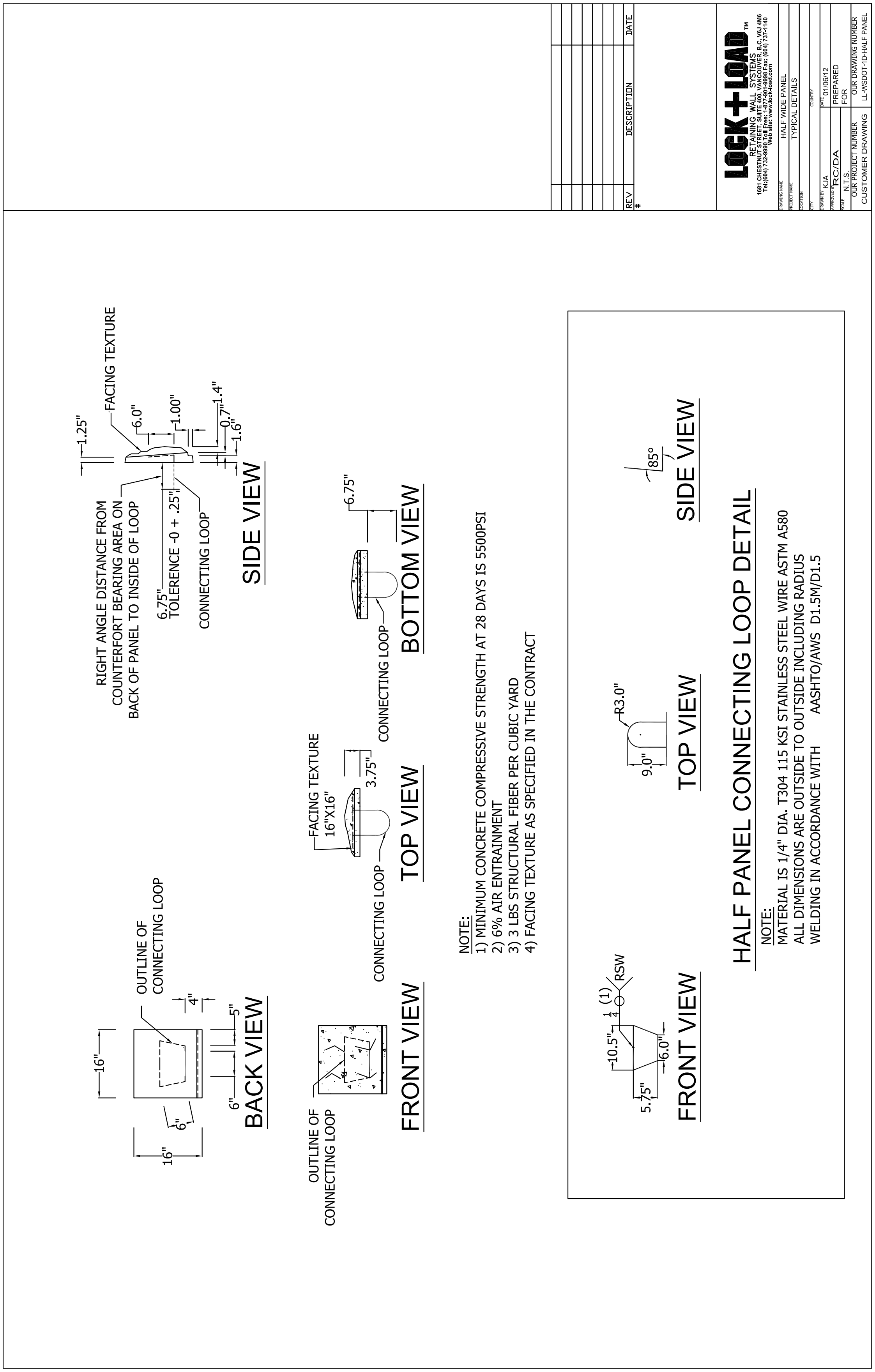


LOCK+LOAD™
WSDOT
REV #
DESCRIPTION
DATE
LOCK+LOAD™ RETAINING WALL SYSTEMS
GEO WRAP WALL W / LOCK+LOAD
TYPICAL DETAILS
K.I.A.
RC/DA
N.T.S.
1/06/12
PREPARED FOR
OUR PROJECT NUMBER
CUSTOMER DRAWING
OUR DRAWING NUMBER
LL-WSDOT-1A-GEO-A1
(1) SET COUNTERFORT AND PANEL ON COMPLETED LIFT.
(2) UNROLL GEOGRID REINFORCEMENT AND POSITION IT SO THAT THE TAIL DRAPES OVER THE FACING PANEL. CUT GEOGRID ON BOTH SIDES OF EACH COUNTERFORT ONLY, AND TUCK IN SLACK GEOGRID (SEE AXON VIEW), CUT ONLY GEOGRID STRANDS THAT RUN PARALLEL TO WALL FACE AND CROSS OVER COUNTERFORT.
PLACE BACK FILL IN A WINDROW SLIGHTLY GREATER THAN FULL LIFT AGAINST LOCK + LOAD PANEL TO HOLD GEOGRID IN PLACE.
ROLL GRID INTO PLACE. STAKE OR HOLD THE GEOGRID TAUT AND FREE OF WRINKLES WHILE PLACING BACKFILL. 12 INCHES AT BACK OF PANEL AND 8 INCHES OVER THE TAIL OF THE COUNTERFORT BACK.
(3) PLACE THE GEOGROD TAIL OVER THE WINDROW, SLOPE TAIL TO TOP OF REAR OF COUNTERFORT. LOCK INTO PLACE WITH BACKFILL.
ENTIRE REINFORCED ZONE TO BE COMPACTED TO WSDOT STANDARD SPECIFICATION FOR MSE REINFORCED ZONE.
GEO GRID
SPECIFIED BACKFILL
GAVEL BARROW, WSDOT SPEC 9-03.14(4) ¾ MAX PARTICAL SIZE 18" AVERAGE DEPTH (MIN)
(1)
PANEL
GEOGRID ROLLED UP BEHIND COUNTERFORT TO PROTECT GEOGRID WHILE COUNTERFORT BACKFILL IS PLACED.
GEO GRID
MINIMUM OVERLAP 4 FEET
(2)
PANEL
BACK FILL IN COMPACTED LIFTS
GEO GRID TAIL MINIMUM OVERLAP 4 FEET COMPACTED IN BACKFILL
(3)
PANEL
FINISHED LOCK+LOAD COURSE INSTALLATION
(4)
PANEL
LOCK+LOAD LIFT ASSEMBLY DETAIL



REV #
DESCRIPTION
DATE
LOCK+LOAD RETAINING WALL SYSTEMS
GEO SYNTHETIC WALL W/LOCK + LOAD
TYPICAL DETAILS
01/06/12
KJA
RC/DA
PREPARED FOR
N.T.S.
OUR PROJECT NUMBER
CUSTOMER DRAWING
OUR DRAWING NUMBER
LL-WSDOT-1B-GEO-A2
CUT GEOGRID ON BOTH SIDE OF COUNTERFORT TO RELIEVE TENSION ON GRID.
CUT ONLY PERPENDICULER TO WALL PANEL
GEO GRID
LOCK+LOAD PANEL
UNIFORMED COMPACTION FROM PANEL TO END OF GEOGRID
GRID DETAIL
FULL UNIFORMED COMPACTION IS MANDATORY.
(LOCK + LOAD CANTILEVER SYSTEM MAKES IT POSSIBLE TO ACHIEVE 95% PROCTOR AT WALL FACE)
* ALL REINFORCEMENT PRODUCTS FROM WSDOT QPL LIST
PANEL LOOP
COUNTERFORT
GAVEL BARROW, WSDOT SPEC 9-03.14(4) ¾ MAX PARTICAL SIZE
3/4" - 1" VERTICAL GAP CONSOLIDATION GAP
PANEL
ASSEMBLY DETAIL
GEO GRID WRAP (MINIMUM 4 FEET)
GAVEL BARROW, WSDOT SPEC 9-03.14(4) ¾ MAX PARTICAL SIZE
GEO GRID
PROJECT SPECIFIED REINFORCED BACKFILL ZONE.
PANEL
TYPICAL SECTION AT WALL FACE
NOTE.
ENTIRE REINFORCED ZONE TO BE COMPACTED TO WSDOT STANDARD SPECIFICATION FOR MSE REINFORCED ZONE.


NOTE:
1) MINIMUM CONCRETE COMPRESSIVE STRENGTH AT 28 DAYS IS 5500 PSI
2) 6% AIR ENTRAINMENT
3) 3 LBS STRUCTURAL FIBER PER CUBIC YARD
4) FACING TEXTURE AS SPECIFIED IN CONTRACT
5) LOOP EMBEDMENT STAYS THE SAME FOR ALL TEXTURES
FRONT VIEW
CONNECTING LOOP
BACK VIEW
WEIGHT REDUCTION BLOCKOUTS
LOOP STEEL
CONNECTING LOOP
32"
22"
5"
5"
6"
6"
4"
16"
TOP VIEW
FACING TEXTURE
3 3/4"
CONNECTING LOOP
6"
SIDE VIEW
WEIGHT REDUCTION BLOCKOUTS
"LOOP HEIGHT" RIGHT ANGLE DISTANCE FROM COUNTERFORT BEARING AREA ON BACK OF PANEL TO INSIDE OF LOOP
CONNECTING LOOP
FACING TEXTURE
1.25"
16"
6"
1"
0.75"
1.40"
1.60"
6.75"
TOLERENCE -0 + .25"
BOTTOM VIEW
CONNECTING LOOP
CONNECTING LOOP- DETAIL
FRONT VIEW
22
6"
6.0"
RSW
TOP VIEW
R3.0"
9.0"
SIDE VIEW
85°
NOTE:
MATERIAL IS 1/4" DIA. T304 STAINLESS STEEL WIRE ASTM A580
ALL DIMENSIONS OUTSIDE TO OUTSIDE INCLUDING RADIUS
WELDING IN ACCORDANCE WITH AASHTO/AWS D1.5M/D1.5
REV #
DESCRIPTION
DATE
LOCK+LOAD RETAINING WALL SYSTEMS
STANDARD PANEL / CONNECTION DETAIL
TYPICAL DETAILS
KJA
RC/DA
N.T.S.
01/06/12
PREPARED FOR
OUR PROJECT NUMBER
CUSTOMER DRAWING
OUR DRAWING NUMBER
LL-WSDOT-1C PANEL


REV #
DESCRIPTION
DATE
LOCK+LOAD RETAINING WALL SYSTEMS
HALF WIDE PANEL
TYPICAL DETAILS
01/06/12
KJA
RC/DA
N.T.S.
PREPARED FOR
OUR PROJECT NUMBER
CUSTOMER DRAWING
OUR DRAWING NUMBER
LL-WSDOT-1D-HALF PANEL
FACING TEXTURE
1.25"
6.0"
1.00"
1.4"
0.7"
1.6"
RIGHT ANGLE DISTANCE FROM COUNTERFORT BEARING AREA ON BACK OF PANEL TO INSIDE OF LOOP
6.75" TOLERENCE -0 + .25"
CONNECTING LOOP
SIDE VIEW
6.75"
CONNECTING LOOP
BOTTOM VIEW
FACING TEXTURE 16"X16"
3.75"
CONNECTING LOOP
TOP VIEW
OUTLINE OF CONNECTING LOOP
16"
4"
5"
6"
6"
16"
BACK VIEW
OUTLINE OF CONNECTING LOOP
FRONT VIEW
NOTE:
1) MINIMUM CONCRETE COMPRESSIVE STRENGTH AT 28 DAYS IS 5500PSI
2) 6% AIR ENTRAINMENT
3) 3 LBS STRUCTURAL FIBER PER CUBIC YARD
4) FACING TEXTURE AS SPECIFIED IN THE CONTRACT
85°
SIDE VIEW
R3.0"
9.0"
TOP VIEW
10.5"
5.75"
6.0"
1/4 (1)
RSW
FRONT VIEW
HALF PANEL CONNECTING LOOP DETAIL
NOTE:
MATERIAL IS 1/4" DIA. T304 115 KSI STAINLESS STEEL WIRE ASTM A580
ALL DIMENSIONS ARE OUTSIDE TO OUTSIDE INCLUDING RADIUS
WELDING IN ACCORDANCE WITH AASHTO/AWS D1.5M/D1.5