

## 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* 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., Elias, V., Espinoza, R. D., Sabatini, P. J., 2003. *Geotechnical Engineering Circular No. 7, Soil Nail Walls*, U.S. Department of Transportation, Federal Highway Administration, FHWA-IF-03-017, 305 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 Division. 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 for Road, Bridge, and Municipal Construction* 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*
  - *Easements*
  - *Backslope and toe slope geometries*
  - *Excavation limits*
  - *Right of way restrictions*
  - *Wetlands*
  - *Materials sources*
  - *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*
  - *Global stability*
  - *Settlement*
  - *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.

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
<p><b>Fill Walls/ Reinforced Soil Slopes</b></p>	<ul style="list-style-type: none"> <li>• internal stability</li> <li>• external stability</li> <li>• global and compound stability</li> <li>• limitations on rate of construction</li> <li>• settlement</li> <li>• horizontal deformation?</li> <li>• lateral earth pressures?</li> <li>• bearing capacity?</li> <li>• chemical compatibility with soil, groundwater, and wall materials?</li> <li>• pore pressures behind wall</li> <li>• borrow source evaluation (available quantity and quality of borrow soil)</li> <li>• liquefaction</li> <li>• potential for subsidence (karst, mining, etc.)</li> <li>• constructability</li> <li>• scour</li> </ul>	<ul style="list-style-type: none"> <li>• subsurface profile (soil, ground water, rock)</li> <li>• horizontal earth pressure coefficients</li> <li>• interface shear strengths</li> <li>• foundation soil/wall fill shear strengths?</li> <li>• compressibility parameters? (including consolidation, shrink/swell potential, and elastic modulus)</li> <li>• chemical composition of fill/ foundation soils?</li> <li>• hydraulic conductivity of soils directly behind wall?</li> <li>• time-rate consolidation parameters?</li> <li>• geologic mapping including orientation and characteristics of rock discontinuities?</li> <li>• design flood elevations</li> <li>• seismicity</li> </ul>	<ul style="list-style-type: none"> <li>• SPT</li> <li>• CPT</li> <li>• dilatometer</li> <li>• vane shear</li> <li>• piezometers</li> <li>• test fill?</li> <li>• nuclear density?</li> <li>• pullout test (MSEW/RSS)</li> <li>• rock coring (RQD)</li> <li>• geophysical testing</li> </ul>	<ul style="list-style-type: none"> <li>• 1-D Oedometer</li> <li>• triaxial tests</li> <li>• unconfined compression</li> <li>• direct shear tests</li> <li>• grain size distribution</li> <li>• Atterberg limits</li> <li>• specific gravity</li> <li>• pH, resistivity, chloride, and sulfate tests?</li> <li>• moisture content?</li> <li>• organic content</li> <li>• moisture-density relationships</li> <li>• hydraulic conductivity</li> </ul>
<p><b>Cut Walls</b></p>	<ul style="list-style-type: none"> <li>• internal stability</li> <li>• external stability</li> <li>• excavation stability</li> <li>• global and compound stability</li> <li>• dewatering</li> <li>• chemical compatibility of wall/soil</li> <li>• lateral earth pressure</li> <li>• down-drag on wall</li> <li>• pore pressures behind wall</li> <li>• obstructions in retained soil</li> <li>• liquefaction</li> <li>• see page</li> <li>• potential for subsidence (karst, mining, etc.)</li> <li>• constructability</li> </ul>	<ul style="list-style-type: none"> <li>• subsurface profile (soil, ground water, rock)</li> <li>• shear strength of soil</li> <li>• horizontal earth pressure coefficients</li> <li>• interface shear strength (soil and reinforcement)</li> <li>• hydraulic conductivity of soil</li> <li>• geologic mapping including orientation and characteristics of rock discontinuities</li> <li>• seismicity</li> </ul>	<ul style="list-style-type: none"> <li>• test cut to evaluate stand-up time</li> <li>• well pumping tests</li> <li>• piezometers</li> <li>• SPT</li> <li>• CPT</li> <li>• vane shear</li> <li>• dilatometer</li> <li>• pullout tests (anchors, nails)</li> <li>• geophysical testing</li> </ul>	<ul style="list-style-type: none"> <li>• triaxial tests</li> <li>• unconfined compression</li> <li>• direct shear</li> <li>• grain size distribution</li> <li>• Atterberg limits</li> <li>• specific gravity</li> <li>• pH, resistivity tests</li> <li>• organic content</li> <li>• hydraulic conductivity</li> <li>• moisture content</li> <li>• unit weight</li> </ul>

**Summary of Information Needs and Testing Considerations**  
Table 15-1

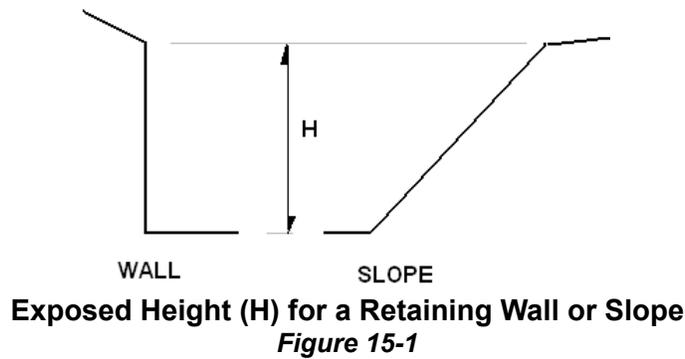
### 15.3.3 Site Reconnaissance

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 of 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 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.



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.

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.

### **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 (fine grained soils only). 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., shelly 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  $36^\circ$  to  $38^\circ$  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.

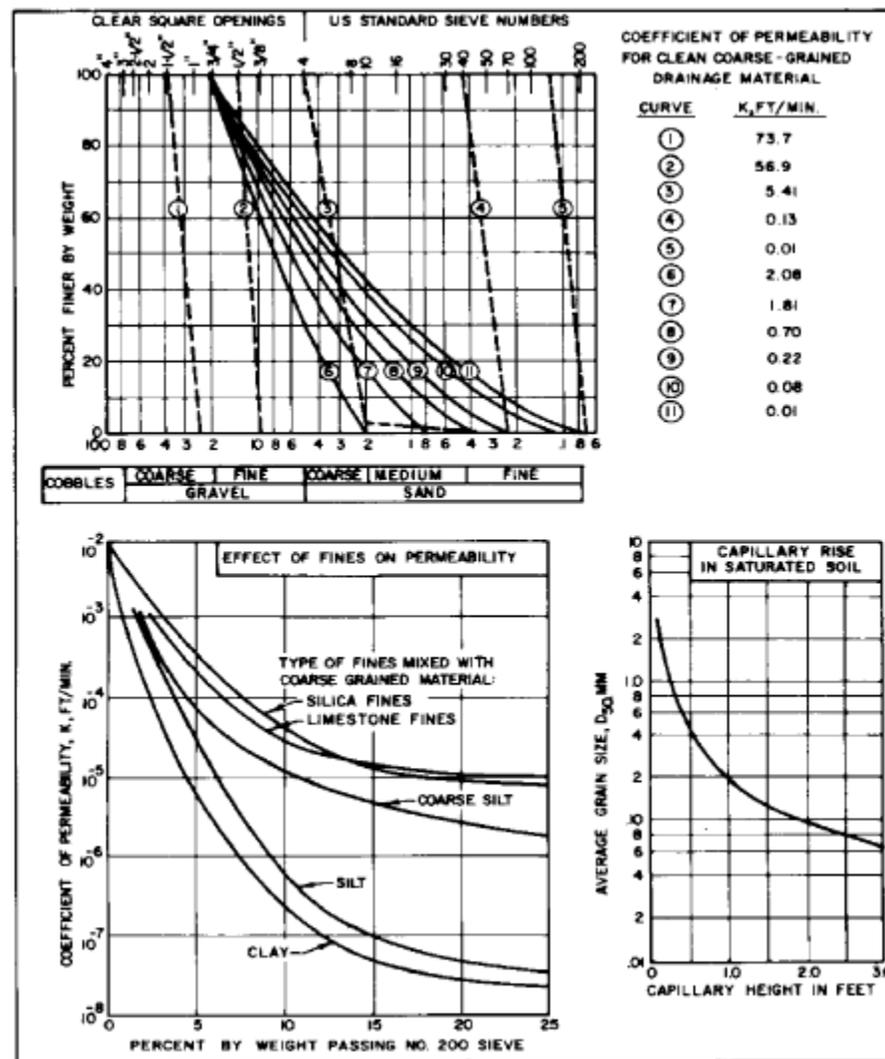
Other typical soil design properties for various types of backfill and native soil units are provided in [Chapter 5](#).

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.



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

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  $P_f$  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 or required 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 are tied to both wall faces, the walls shall be considered unable to slide to reduce the acceleration to be applied. Therefore, the full ground acceleration shall be used in the walls in that case.

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 15.5.3.5), flexible faced MSE walls with a vegetated face (Section 15.5.3.6), and MSE wall supported bridge abutments (Section 15.5.3.4), and modular dry cast concrete block faced systems (Section 15.5.3.8). 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):

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

<sup>1</sup>Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

**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)**

*Table 15-2*

Total Settlement	Differential Settlement Over 100 Feet	Action
$\Delta H \leq 2$ in	$\Delta H_{100} \leq 1.5$ in	Design and Construct
$2 \text{ in} < \Delta H \leq 4$ in	$1.5 \text{ in} < \Delta H_{100} \leq 3$ in	Ensure structure can tolerate settlement
$\Delta H > 4$ in	$\Delta H_{100} > 3$ in	Obtain Approval <sup>1</sup> prior to proceeding with design and Construction

<sup>1</sup>Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

**Settlement Criteria for MSE Walls With Modular (Segmental) Block Facings, Prefabricated Modular Walls, and Rock Walls**

*Table 15-3*

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

<sup>1</sup>Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

**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**

*Table 15-4*

For MSE walls with precast panel facings up to 75 feet<sup>2</sup> 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 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 concrete walls, MSE walls, soil nail walls, soldier pile walls and anchored walls are generally considered as 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 if 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.

#### **15.4.12 Overall Stability**

All retaining walls and reinforced slopes 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. All abutments and those retaining walls and reinforced slopes that support structures such as bridges, other retaining walls, buildings, pipelines or other critical utilities shall have a resistance factor of 0.65 (i.e., a safety factor of 1.5). See Section 8.6.5.2 and the AASHTO LRFD Bridge Design Specifications, Article 11.6.2.3 and commentary for additional background and guidance regarding the assessment of overall stability.

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 should be provided for all walls. 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.

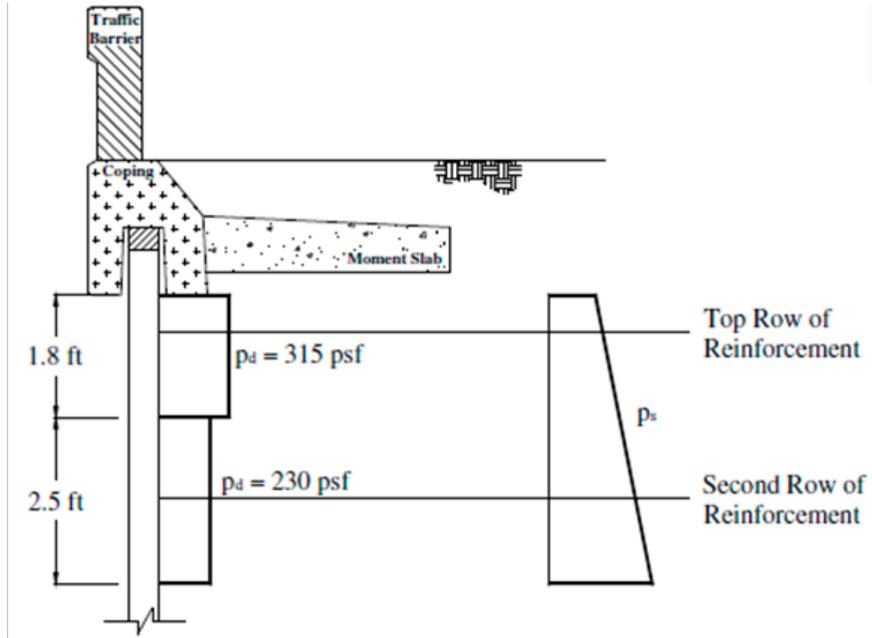
#### 15.4.14 Utilities

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.

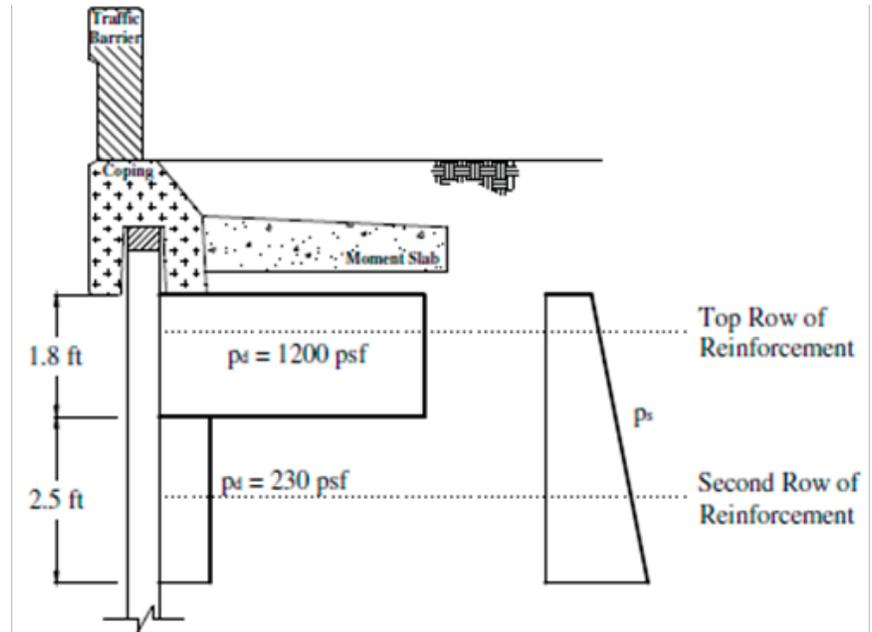
#### 15.4.15 Guardrail and Barrier

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 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.



(a) Pressure distribution for reinforcement pullout



(b) Pressure distribution for reinforcement rupture.

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

Figure 15-3

## 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. Instead of the resistance factor for passive resistance of the vertical wall elements provided in the AASHTO LRFD specifications, a resistance factor for passive resistance of 0.75 shall be used.

#### 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 ( $\tau_n$ ) 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, or clays in general, the level of safety obtained in past WSDOT practice shall continue to be used (i.e., FS = 1.5). 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  $\gamma_{EH}$  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,  $\gamma_{EH}$ , 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.
<b>1.15FDL</b>	<b>60 Min.</b>
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.
<b>1.15FDL</b>	<b>10 Min.</b>
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.
<b>1.00FDL</b>	<b>10 Min.</b>
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	
				<b>1.15FDL</b>	

\*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).

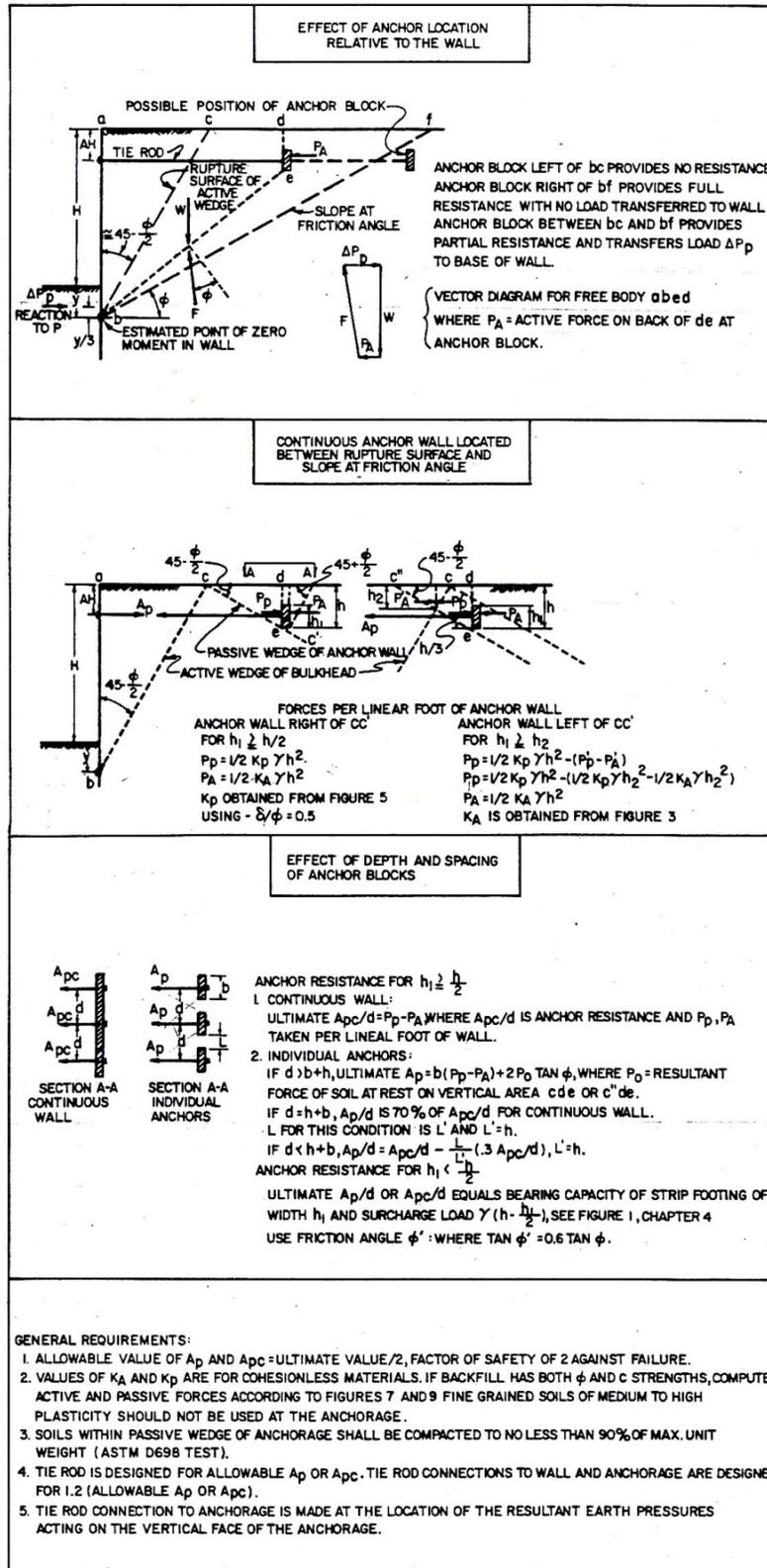
#### **15.5.3 Mechanically Stabilized Earth Walls**

Wall design shall be in accordance with the AASHTO *LRFD Bridge Design Specifications*, except as noted below regarding the use of the K-Stiffness Method for internal stability design.

### 15.5.3.1 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.



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

### 15.5.3.2 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). 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 K-Stiffness Method (see Section 15.5.3.1) 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  $c-\phi$  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.

### 15.5.3.3 Compound Stability Assessment for MSE Walls

If the MSE wall is located over a soft foundation soil or on a relatively steep slope, compound stability of the wall and slope combination should be evaluated as a service limit state in accordance with the AASHTO LRFD Specifications. 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 (2003), 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.

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 15-5. 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, 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.

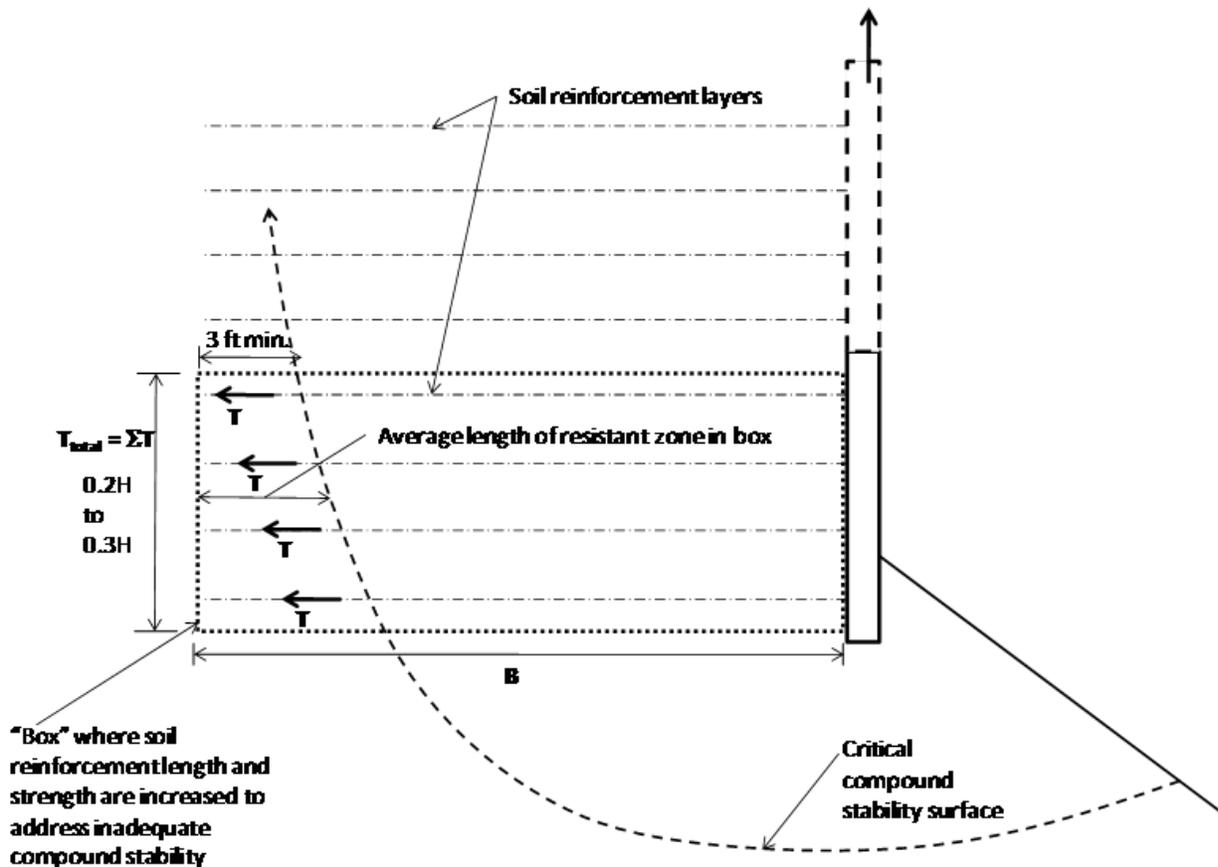
Note that for taller walls, it may be desirable to define more than one box at the wall base to improve the accuracy of the pullout length for the intersected reinforcement layers. If the wall is tiered, a box may need to be provided at the base of each tier, depending on the horizontal separation between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

#### 15.5.3.4 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 15-6 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:

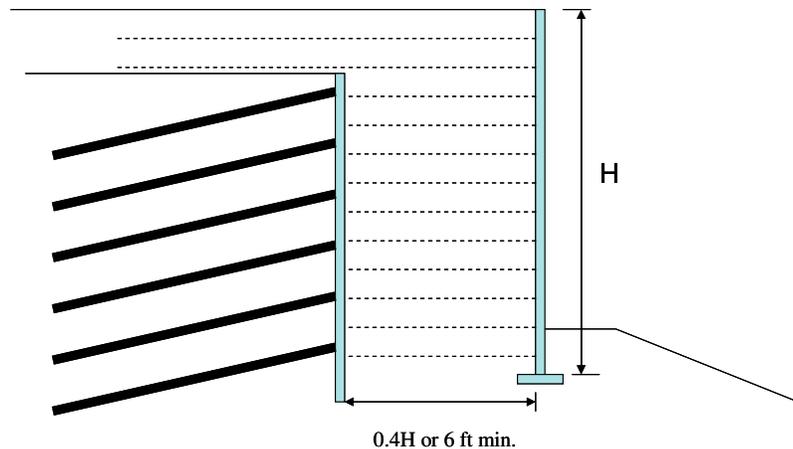
- The minimum base width is  $0.4H$  or 6 feet, whichever is greater, where  $H$  is the total height of the new wall. Note that for soil reinforcement lengths that are less than 8 feet, the weight and size of construction equipment used to place and compact the soil backfill will need to be limited 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, but no less than 3 feet of reinforced soil, 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 near vertical 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.,  $T_{\max}$ ).



### Compound Stability Assessment Concept for MSE Wall Design

Figure 15-5

- 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.



**Example of Steep Shored MSE Wall**  
**Figure 15-6**

### 15.5.3.5 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 hierarchical order:

1. This *Geotechnical Design Manual*
2. The *Bridge Design Manual* and Bridge Office design policy update provided in the Bridge Office Design Memorandum entitled “Bridges with MSE wall supported abutments,” dated June 25, 2013
3. AASHTO (2012), AASHTO LRFD Bridge Design Specifications, 6th Edition.
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/](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:

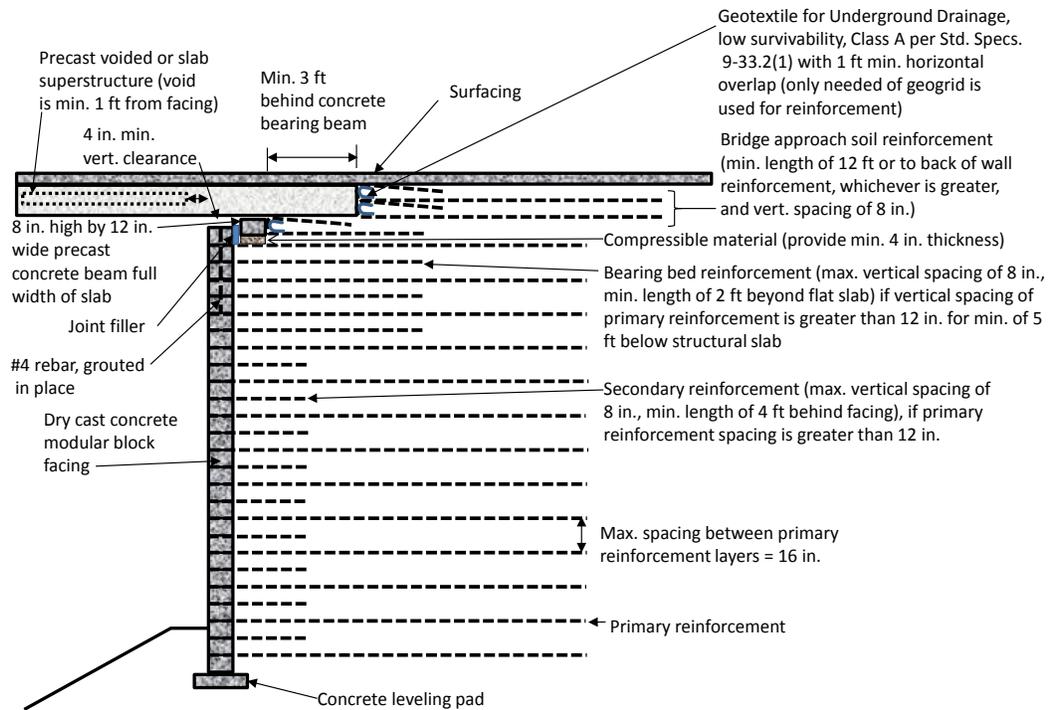
- For single or multi-span bridges, use of a footing foundation placed directly above the MSE wall reinforced soil zone, or
- For flat slab single span bridges with a span length of up to 60 feet, the end of the flat slab itself bears directly on the surface of the MSE wall reinforced soil zone.

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 MSE 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.

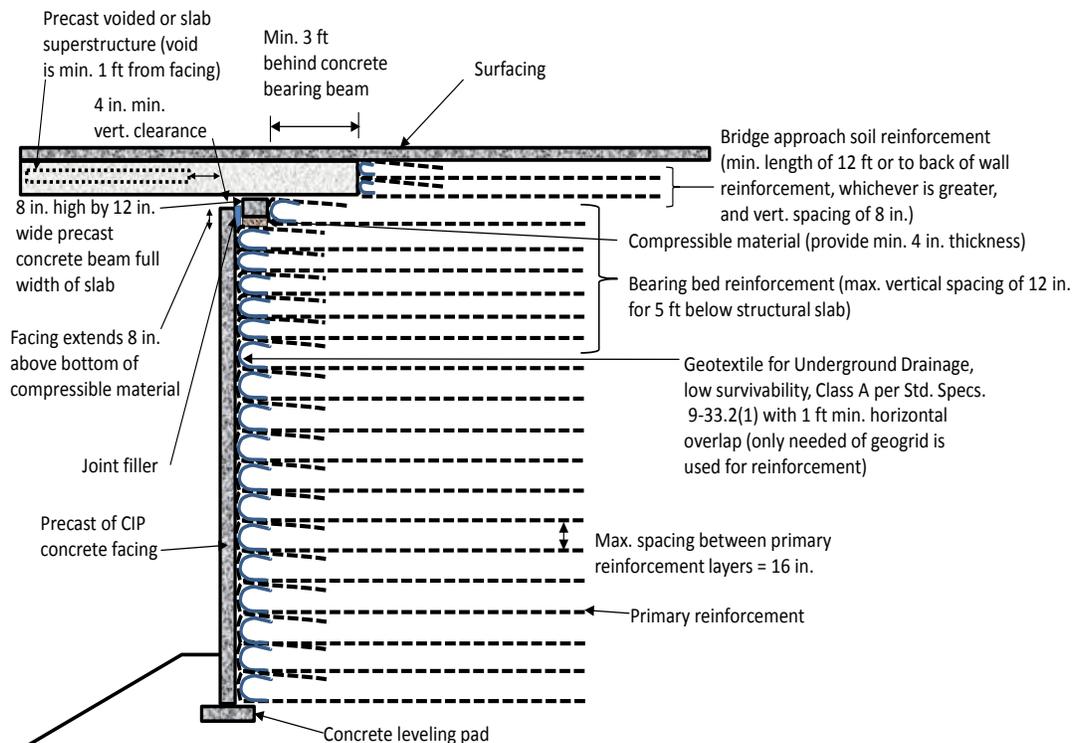
For this MSE wall application, only the following MSE wall/facing types shall be used:

- 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 15D), and
- WSDOT preapproved proprietary MSE walls identified as such (see Appendix 15D), 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 15D as preapproved proprietary walls shall not be considered preapproved for the MSE wall supported bridge abutment application (i.e., a special design is required).

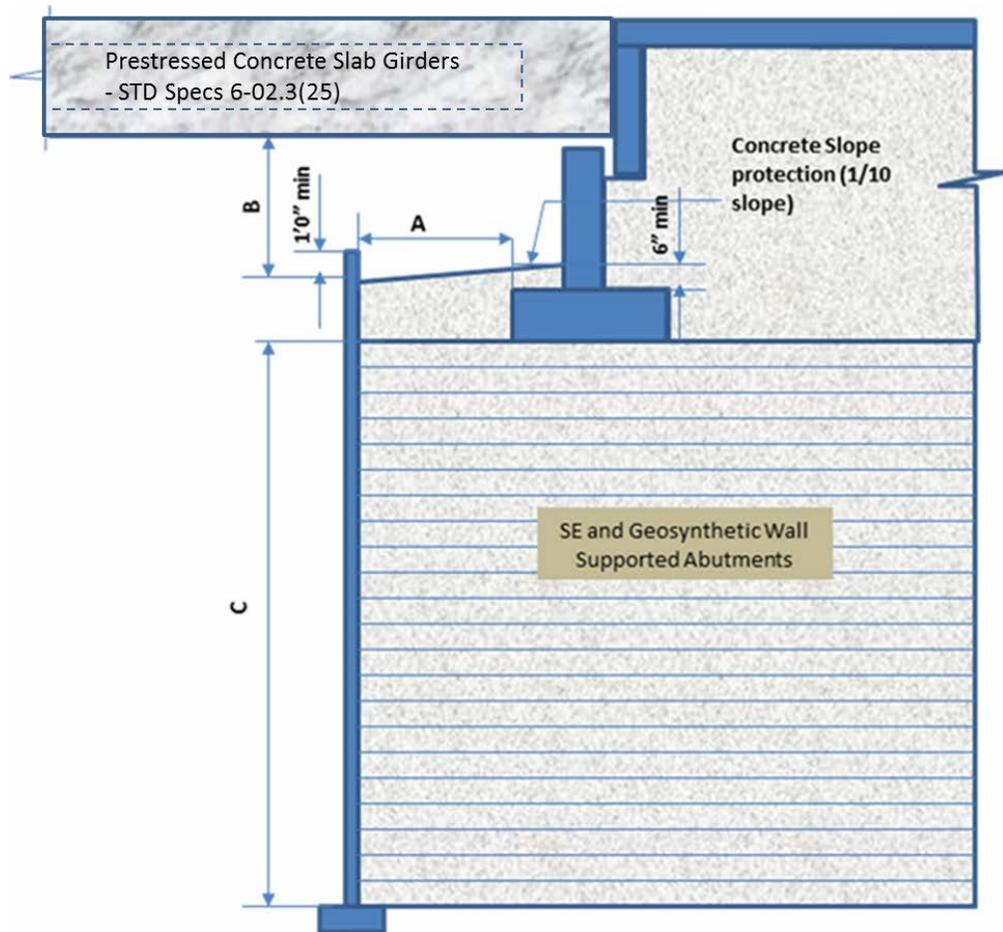
Figures 15-7, 15-8, and 15-9 provide typical sections that should be used in the design of MSE wall bridge abutments. The base of the wall may be truncated to reduce excavation needs subject to the limitations provided in Section 11 of the AASHTO LRFD Bridge Design Specifications. Figure 15-8 is similar 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 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.



**Typical Section for MSE Wall Supported Abutment – Flat Slab Superstructure With no Footing and Dry-Cast Modular Block Wall Facing**  
 Figure 15-7



**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 15-8



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

**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)**

**Figure 15-9**

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 15-7 and 15-8 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 15-9, 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 effect 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.

### 15.5.3.6 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.

### 15.5.3.7 Flexible Faced MSE Walls With Vegetation

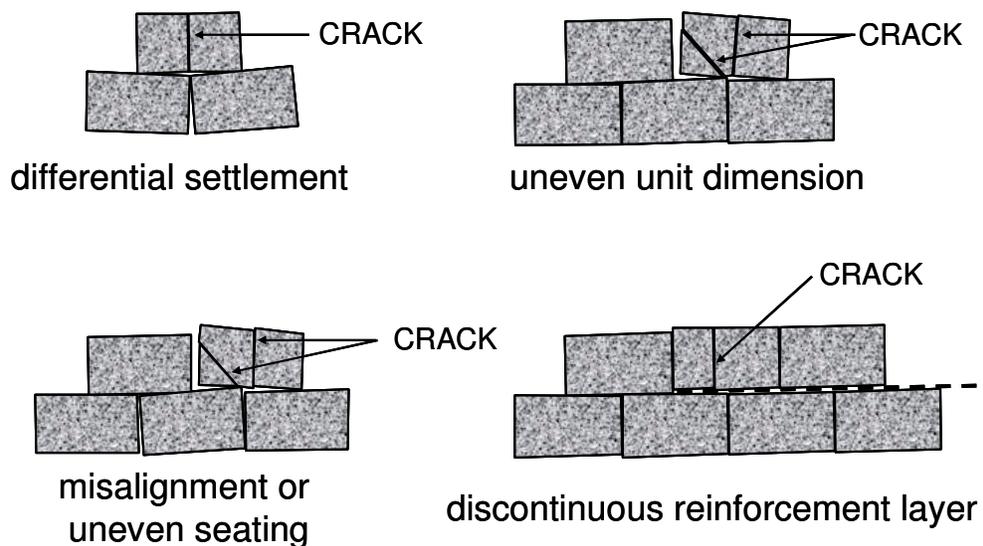
If a vegetated face is to be used with an MSE wall, the exposed (i.e., above ground wall height shall be limited to 20 feet or less, and the wall face batter shall be no steeper than 1H:6V, unless the facing is battered at 1H:2V or flatter, in which case the maximum height could be extended to 30 feet). 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.

### 15.5.3.8 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 15-10 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.



**Example Causes of Cracking in Modular Dry Cast Concrete Block Wall Facings**

**Figure 15-10**

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.

#### 15.5.3.9 Internal Stability Using K-Stiffness Method

The K-Stiffness Method, as described by Allen and Bathurst (2003) and as updated by Bathurst et al. (2008b), may be used as an alternative to the Simplified Method provided in the AASHTO *LRFD Bridge Design Specifications* (Sections 3 and 11) to design the internal stability for walls up to 35 feet in height that are not directly supporting other structures and that are not in high settlement areas (i.e., total settlement beneath the wall of 6 in or more). Use of the K-Stiffness Method for greater wall heights, in locations where settlement is anticipated to be 6 in or more, or for walls that support other structures shall be considered experimental, will require special monitoring of performance, and will require the approval of the State Geotechnical Engineer. 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.

##### 15.5.3.9.1 K-Stiffness Method Loads and Load Factors

The methods used in historical design practice for calculating the load in the reinforcement to accomplish internal stability design include the Simplified Method, the Coherent Gravity Method, and the FHWA Structure Stiffness Method. All of these methods are empirically derived, relying on limit equilibrium concepts for their formulation, whereas, the K-Stiffness Method, also empirically derived, relies on the difference in stiffness of the various wall components to distribute a total lateral earth pressure derived from limit equilibrium concepts 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 K-Stiffness Method can be used to directly evaluate the potential for soil backfill failure and to design the wall internally for the service limit state. These other

methods used in historical practice indirectly account for soil failure and service limit state conditions 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).

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

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) which supports the acceptability of larger vertical spacings. Allen and Bathurst (2003) do report that based on data from a number of wall case histories, the correlation between vertical spacing and reinforcement load appears to remain linear for vertical spacings ranging from 1 to 5 feet, though the data at vertical spacings greater than 2.7 feet are very limited. However, larger vertical spacings 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.

The factored vertical stress,  $\sigma_v$ , at each reinforcement level shall be calculated as:

$$\sigma_v = \gamma_p \gamma_r H + \gamma_p \gamma_f S + \gamma_{LL} q \quad (15-1)$$

Where:

- $\sigma_v$  = the factored pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present (KSF)
- $\gamma_p$  = the load factor for vertical earth pressure EV in Table 15-5
- $\gamma_{LL}$  = the load factor for live load surcharge per the AASHTO LRFD Specifications
- $q$  = live load surcharge (KSF)
- $H$  = the total vertical wall height at the wall face (FT)
- $S$  = average soil surcharge depth above wall top (FT)
- $\gamma_r$  = the unit weight of the reinforced soil backfill (KCF)
- $\gamma_f$  = the unit weight of the soil backfill behind and above the reinforced soil zone (KCF)

Note that sloping soil surcharges are taken into account through an equivalent uniform surcharge and assuming a level backslope condition. For these calculations, the wall height “H” is referenced from the top of the wall at the wall face to the top of the bearing pad, excluding any copings and appurtenances.

Methods used in historical practice (e.g., the Simplified Method) calculate the vertical stress resulting from gravity forces within the reinforced backfill at each level, resulting in a linearly increasing gravity force with depth and a triangular lateral stress distribution. The K-Stiffness Method instead calculates the maximum gravity force resulting from the gravity forces within the reinforced soil backfill to determine the maximum reinforcement load within the entire wall reinforced backfill,  $T_{\text{max}}$ , and then adjusts that maximum reinforcement load with depth for each of the layers using

a load distribution factor,  $D_{tmax}$  to determine  $T_{max}$ . This load distribution factor was derived empirically based on a number of full scale wall cases and verified through many numerical analyses (see Allen and Bathurst, 2003).

For the K-Stiffness Method, the load in the reinforcements is obtained by multiplying the factored vertical earth pressure by a series of empirical factors which take into account the reinforcement global stiffness for the wall, the facing stiffness, the facing batter, the local stiffness of the reinforcement, the soil strength and stiffness, and how the load is distributed to the reinforcement layers. The maximum factored load in each reinforcement layer shall be determined as follows:

$$T_{max} = 0.5S_v K \sigma_v D_{tmax} \Phi_g \Phi_{local} \Phi_{fb} \Phi_{fs} \Phi_c \quad (15-2)$$

Where:

- $S_v$  = tributary area (assumed equivalent to the average vertical spacing of the reinforcement at each layer location when analyses are carried out per unit length of wall), in FT
- $K$  = is an index lateral earth pressure coefficient for the reinforced backfill, and shall be set equal to  $K_0$  as calculated per Article 3.11.5.2 of the AASHTO LRFD Specifications.  $K$  shall be no less than 0.3 for steel reinforced systems.
- $\sigma_v$  = the factored pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present, as calculated in Equation 15-1 (KSF)
- $D_{tmax}$  = distribution factor to estimate  $T_{max}$  for each layer as a function of its depth below the wall top relative to  $T_{mmax}$  (the maximum value of  $T_{max}$  within the wall)
- $S_{global}$  = global reinforcement stiffness (KSF)
- $\Phi_g$  = global stiffness factor
- $\Phi_{local}$  = local stiffness factor
- $\Phi_{fb}$  = facing batter factor
- $\Phi_{fs}$  = facing stiffness factor
- $\Phi_c$  = soil backfill cohesion factor

$D_{tmax}$  shall be determined from Figure 15-6.

The global stiffness,  $S_{global}$ , considers the stiffness of the entire wall section, and it shall be calculated as follows:

$$S_{global} = \frac{J_{ave}}{(H/n)} = \frac{\sum_{i=1}^n J_i}{H} \quad (15-3)$$

Where:

$J_{ave}$  is the average stiffness of all the reinforcement layers within the entire wall section on a per FT of wall width basis (KIPS/FT),  $J_i$  is the stiffness of an individual reinforcement layer on a per FT of wall width basis (KIPS/FT),  $H$  is the total wall height (FT), and  $n$  is the number of reinforcement layers within the entire wall section.

$$\Phi_g = 0.25 \left( \frac{S_{global}}{p_a} \right)^{0.25} \quad (15-4)$$

Where:

$p_a$  = atmospheric pressure (a constant equal to 2.11 KSF), and the other variables are as defined previously.

The local stiffness considers the stiffness and reinforcement density at a given layer and is calculated as follows:

$$S_{local} = \frac{J}{S_v} \quad (15-5)$$

Where:

$J$  is the stiffness of an individual reinforcement layer (KIPS/FT), and  $S_v$  is the vertical spacing of the reinforcement layers near a specific layer (FT). The local stiffness factor,  $\Phi_{local}$ , is then defined as follows:

$$\Phi_{local} = \left( \frac{S_{local}}{S_{global}} \right)^a \quad (15-6)$$

Where:

$a$  = a coefficient which is also a function of stiffness. Based on observations from the available data, set  $a = 1.0$  for geosynthetic walls and  $= 0.0$  for steel reinforced soil walls.

The wall face batter factor,  $\Phi_{fb}$ , which accounts for the influence of the reduced soil weight on reinforcement loads, is determined as follows:

$$\Phi_{fb} = \left( \frac{K_{abh}}{K_{avh}} \right)^d \quad (15-7)$$

Where:

$K_{abh}$  is the horizontal component of the active earth pressure coefficient accounting for wall face batter, and  $K_{avh}$  is the horizontal component of the active earth pressure coefficient assuming that the wall is vertical, and  $d$  = a constant coefficient (recommended to be 0.5 to provide the best fit to the empirical data).

$K_{abh}$  and  $K_{avh}$  are determined from the Coulomb equation, assuming no wall/soil interface friction and a horizontal backslope (AASHTO 2010), as follows:

$$K_{ab} = \frac{\cos^2(\phi + \omega)}{\cos^3 \omega \left[ 1 + \frac{\sin \phi}{\cos \omega} \right]} \quad (15-8)$$

Where:

$\phi$  = peak soil friction angle ( $\phi_{peak}$ ), and  $\omega$  = wall/slope face inclination (positive in a clockwise direction from the vertical). The wall face batter  $\omega$  is set equal to 0 to determine  $K_{av}$  using Equation 15-8. The horizontal component of the active earth pressure coefficient, assuming no wall/soil interface friction, is determined as follows:

$$K_{abh} = K_{ab} \cos(\omega) \quad (15-9)$$

Since for a vertical wall,  $\omega = 0^\circ$ ,  $K_{av} = K_{avh}$ .

The facing stiffness factor,  $\Phi_{fs}$ , was empirically derived to account for the significantly reduced reinforcement stresses observed for geosynthetic walls with segmental concrete block and propped panel wall facings. It is not yet known whether this facing stiffness correction is fully applicable to steel reinforced wall systems. On the basis of data available at the time of this report, Allen and Bathurst (2003) recommend that this facing stiffness factor be determined as a function of a non-dimensional facing column stiffness parameter  $F_f$ :

$$F_f = \frac{1.5H^3 P_a}{Eb_w^3 \left( \frac{h_{eff}}{H} \right)} \quad (15-10)$$

and

$$\Phi_{fs} = \eta (F_f)^\kappa \quad (15-11)$$

Where:

$b_w$  is the thickness of the facing column,  $H$  = the total wall face height,  $E$  = the modulus of the facing material,  $h_{eff}$  is the equivalent height of an un-jointed facing column that is 100 percent efficient in transmitting moment throughout the facing column, and  $p_a$ , used to preserve dimensional consistency, is atmospheric pressure (equal to 2.11 KSF). The dimensionless coefficients  $\eta$  and  $\kappa$  were determined from an empirical regression of the full-scale field wall data to be 0.69 and 0.11, respectively.

Equation 15-10 was developed by treating the facing column as an equivalent uniformly loaded cantilever beam. It is recognized that Equation 15-10 represents a rather crude model of the stiffness of a retaining wall facing column, considering that the wall toe may not be completely fixed, the facing column often contains joints (i.e., the beam is not continuous), and the beam is attached to the reinforcement at various points. Since this analysis is being used to isolate the contribution of the facing to the load carrying capacity of the wall system, a simplified model that treats the facing as an isolated beam can be used. Once significant deflection occurs in the facing column, the reinforcement is then forced to carry a greater percentage of the load in the wall system. The full-scale wall data was used by Allen and Bathurst (2003) to empirically determine the percentage of load carried by these two wall components. Due to these complexities, these equations have been used in this analysis only to set up the form of a parameter that can be used to represent the approximate stiffness of the facing column.

For modular block faced wall systems, due to their great width,  $h_{eff}$  can be considered approximately equal to the average height of the facing column between reinforcement layers, and that the blocks between the reinforcement layers behave as if continuous. The blocks are in compression, partially due to self weight and partially due to downdrag forces on the back of the facing (Bathurst, et al. 2000), and can effectively transmit moment throughout the height of the column between the reinforcement layers that are placed between the blocks where the reinforcement is connected to

the facing. The compressibility of the reinforcement layer placed between the blocks, however, can interfere with the moment transmission between the blocks above and below the reinforcement layer, effectively reducing the stiffness of the facing column. Therefore,  $h_{\text{eff}}$  should be set equal to the average vertical reinforcement spacing for this type of facing. Incremental panel faced systems are generally thinner (a thickness of approximately 4 to 5.5 in) and the panel joints tend to behave as a pinned connection. Therefore,  $h_{\text{eff}}$  should be set equal to the panel height for this type of facing. The stiffness of flexible wall facings is not as straight-forward to estimate. Until more is known, a facing stiffness factor  $\Phi_{\text{fs}}$  of 1.0 should be used for all flexible faced walls (e.g., welded wire facing, geosynthetic wrapped facings, including such walls where a precast or cast-in-place concrete facing is placed on the wall after the wall is built).

The maximum wall height available where facing stiffness effects could be observed was approximately 35 feet. Data from taller stiff faced walls were not available. It is possible that this facing stiffness effect may not be as strong for much taller walls. Therefore, for walls taller than approximately 35 feet, approval for use of the K-Stiffness Method by the State Geotechnical Engineer is required.

Allen and Bathurst (2003) also discovered that the magnitude of the facing stiffness factor may also be a function of the amount of strain the soil reinforcement allows to occur. It appears that once the maximum reinforcement strain in the wall exceeds approximately 2 percent strain, stiff wall facings tend to reach their capacity to restrict larger lateral earth pressures. To accommodate this strain effect on the facing stiffness factor, for stiff faced walls, the facing stiffness factor increases for maximum reinforcement strains above 2 percent. Because of this, it is recommended that stiff faced walls be designed for maximum reinforcement strains of approximately 2 percent or less, if a facing stiffness factor  $\Phi_{\text{fs}}$  of less than 0.9 is used.

For steel reinforced walls, this facing stiffness effect has not been verified, though preliminary data indicates that facing stiffness does not affect reinforcement load significantly for steel reinforced systems. Therefore, a facing stiffness factor  $\Phi_{\text{fs}}$  of 1.0 shall be used for all steel reinforced MSE wall systems.

The backfill soil cohesion factor,  $\Phi_c$ , is calculated as:

$$\Phi_c = 1 - \lambda \frac{c}{\gamma H} \quad (15-12)$$

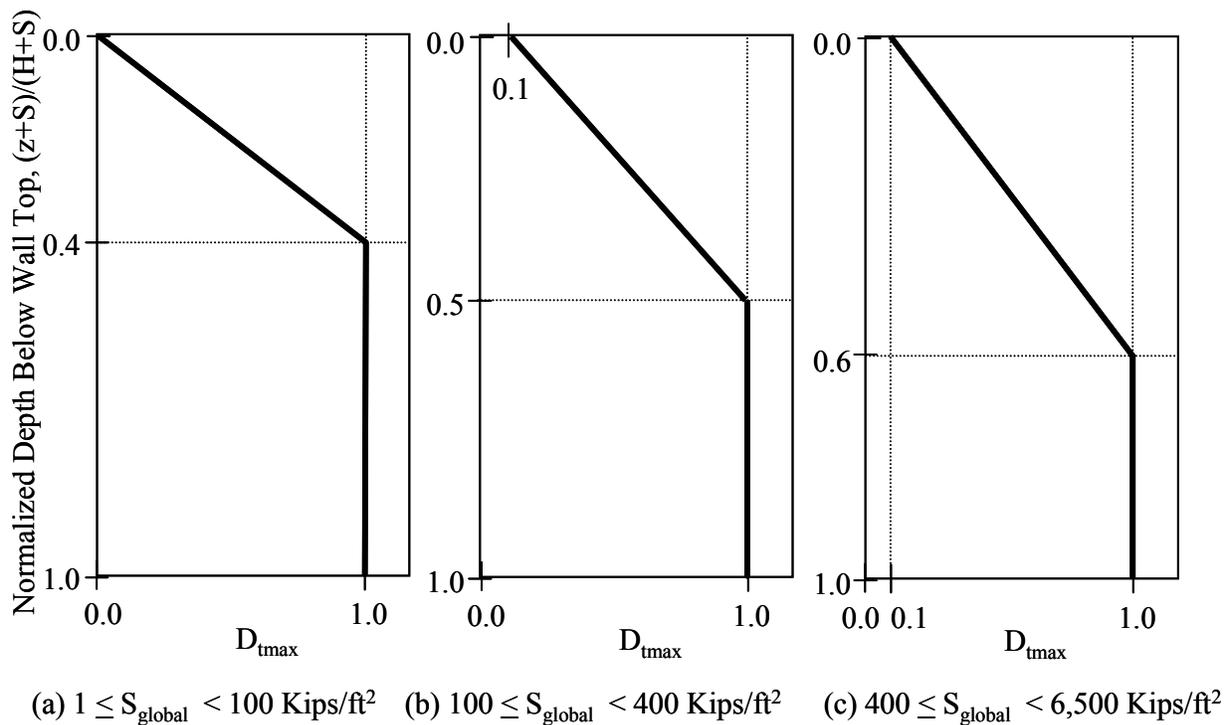
Where:

the cohesion coefficient  $\lambda = 6.5$ ,  $c$  is the soil cohesion,  $\gamma$  is the soil unit weight, and  $H$  is the wall height. The practical limit  $0 \geq \Phi_c \geq 1$  requires  $c/\gamma H \leq 0.153$ . It is possible that a combination of a short wall height and high cohesive soil strength could lead to  $\Phi_c = 0$ . In practical terms this means that no reinforcement is required for internal stability. However, this does not mean that the wall will be stable at the facing (e.g., connection over-stressing may still occur).

Note that soil cohesion should not be relied upon for final wall design (i.e., set  $c = 0$ ). If a backfill soil with significant cohesion must be used, with the use of such backfill soils subject to the approval of the State Geotechnical Engineer, the loss of cohesion over time due to backfill moisture gain, or possibly other reasons, should be considered during the design to estimate the long-term performance of the wall, and the potential for long-term deformations. Limited full scale wall data indicate that reinforcement loads could increase over time for soils with a significant cohesion component.

$D_{tmax}$  shall be determined as shown in Figure 15-11. Allen and Bathurst (2003) found that as the reinforcement stiffness increases, the load distribution as a function of depth below the wall top becomes more triangular in shape.  $D_{tmax}$  is the ratio of  $T_{max}$  in a reinforcement layer to the maximum reinforcement load in the wall,  $T_{mxmx}$ . Note that the empirical distributions provided in Figure 15-7 apply to walls constructed on a firm soil foundation. The distributions that would result for a rock or soft soil foundation may be different from those shown in this figure, and in general will tend to be more triangular in shape as the foundation soils become more compressible.

The factored tensile load applied to the soil reinforcement connection at the wall face,  $T_o$ , shall be equal to the maximum factored reinforcement tension,  $T_{max}$ , for all wall systems regardless of facing and reinforcement type.



**$D_{tmax}$  as a Function of Normalized Depth Below Wall Top Plus Average Surcharge Depth: (A) Generally Applies to Geosynthetic Walls, (B) Generally Applies to Polymer Strap Walls and Extensible or Very Lightly Reinforced Steel Reinforced Systems, and (C) Generally Applies to Steel Reinforced Systems**

**Figure 15-11**

Triaxial or direct shear soil friction angles should be used with the Simplified Method provided in the AASHTO LRFD Specifications, to be consistent with the current specifications and empirical derivation for the Simplified Method, whereas plane strain soil friction angles should be used with the K-Stiffness Method, to be consistent with the empirical derivation and calibration for that method. The following equations maybe used to make an approximate estimate of the plane strain soil friction angle based on triaxial or direct shear test results.

For triaxial test data (Lade and Lee, 1976):

$$\phi_{ps} = 1.5\phi_{tx} - 17 \quad (15-13)$$

For direct shear test data (based on interpretation of data presented by Bolton (1986) and Jewell and Wroth (1987)):

$$\phi_{ps} = \tan^{-1} (1.2 \tan \phi_{ds}) \quad (15-14)$$

All soil friction angles are in degrees for both equations. Direct shear or triaxial soil friction angles may be used for design using the K-Stiffness Method, if desired, but it should be recognized that doing so could add some conservatism to the resulting load prediction. Note that if presumptive design parameters are based on experience from triaxial or direct shear testing of the backfill, a slight increase in the presumptive soil friction angle based on Equations 15-13 or 15-14 is appropriate to apply.

#### 15.5.3.9.2 **K-Stiffness Method Load Factors**

In addition to the load factors provided in Section 3.4.1 of the AASHTO LRFD specifications, the load factors provided in Table 15-5 shall be used as minimum values for the K-Stiffness Method. The load factor  $\gamma_p$  to be applied to maximum load carried by the reinforcement  $T_{max}$  due to the weight of the backfill for reinforcement strength, connection strength, and pullout calculations shall be EV, for vertical earth pressure. The load factors presented in Table 15-5 were developed using the soil reinforcement load data presented by Allen and Bathurst (2003), Allen et al. (2003, 2004), and Bathurst et al. (2008b), and the load factor calibration methodology as described in Allen, et al. (2005) and Bathurst, et al. (2008a).

Loads carried by the soil reinforcement in mechanically stabilized earth walls are the result of vertical and lateral earth pressures which exist within the reinforced soil mass, reinforcement extensibility, facing stiffness, wall toe restraint, and the stiffness and strength of the soil backfill within the reinforced soil mass. The calculation method for  $T_{max}$  is empirically derived, based on reinforcement strain measurements, converted to load based on the reinforcement stiffness, from full scale walls at working stress conditions (see Allen and Bathurst, 2003; and Bathurst, et al., 2008). Research by Allen and Bathurst (2003) indicates that the working loads measured in MSE wall reinforcement remain relatively constant throughout the wall life, provided the wall is designed for a stable condition, and that the load statistics remain constant up to the point that the wall begins to fail. Therefore, the load factors for MSE wall reinforcement loads provided in Table 15-5 can be considered valid for strength limit states.

Another strength limit state that needs to be considered for these walls is the prevention of soil failure. Soil failure is defined as contiguous or near-contiguous zones of soil with shear strains in excess of the strain at peak strength. Contiguous shear zones have been observed in test walls taken to collapse under uniform surcharge loading (Bathurst 1990, Bathurst et al. 1993b, Allen and Bathurst 2002b). Allen and Bathurst (2002b) found that once a wall goes beyond working stress conditions, the load levels in the reinforcement begin to increase as internal soil shear surfaces continue to develop and the soil approaches a residual strength. Once the soil has exceeded its peak shear strain and begins to approach its residual shear strength, for all practical purposes the wall has failed and an internal strength limit state for the soil achieved.

The key to prevent reaching the soil failure limit state is to estimate how much strain can be allowed in the reinforced wall system (i.e., the soil reinforcement) without causing the soil to reach what is defined above as a soil failure condition. Preventing the reinforcement strain from exceeding a 3 to 3.5 percent design value will be adequate for the high shear strength granular backfill soils typically specified for walls in Washington State and likely conservative for weaker backfill soils. Since the maximum reinforcement strain to prevent soil failure was derived from high shear strength soils, the 3 to 3.5 percent strain value represents what is effectively a lower bound value. For geosynthetic wall design, the maximum strain in the reinforcement is kept below 3 percent everywhere in the wall; therefore, only the maximum reinforcement strain in the wall must be estimated, and the distribution of the load among the reinforcement layers is not relevant to this calculation. For the K-Stiffness Method, much of the uncertainty in the prediction accuracy of the method is in the distribution of the loads among the reinforcement layers relative to the maximum load in all the reinforcement layers, i.e., the maximum reinforcement load can be predicted more accurately and the loads in all the reinforcement layers. Therefore, a smaller load factor can be used for this limit state for geosynthetic walls. Note that this approach is conservative in that many of the reinforcement layers will be at a strain level that is much less than the maximum value.

For steel reinforced walls, the key to preventing soil failure is to prevent the steel from exceeding its yield strength. Assuming that is accomplished in the design, the strain in the reinforcement and soil will be far below the strain that would allow soil failure to occur. Past design practice has been to ensure that the stress in all the layers of steel reinforcement does not exceed the yield strength of the steel. Since all the reinforcement layers must be checked and designed so that they do not exceed yield, the full distribution of load to each reinforcement layer is important for this calculation. Therefore, the load factor for reinforcement rupture for steel reinforced walls is also used for designing the wall reinforcement layers to not exceed yield.

Type of Load	Load Factor	
	Maximum	Minimum
EV: Vertical Earth Pressure:		
MSE Wall soil reinforcement loads (K-Stiffness Method, steel strips and grids)	1.55	N/A
MSE Wall soil reinforcement/facing connection loads (K-Stiffness Method, steel grids attached to rigid facings)	1.80	N/A
MSE Wall soil reinforcement loads (K-Stiffness Method, geosynthetics, reinforcement rupture)	1.55	N/A
MSE Wall soil reinforcement loads (K-Stiffness Method, geosynthetics, soil failure)	1.40	N/A
MSE Wall soil reinforcement/facing connection loads (K-Stiffness Method, geosynthetics)	1.80	N/A

**Load Factors for Permanent Loads for Internal Stability of MSE Walls Designed Using the K-Stiffness Method,  $\gamma_p$ , for the Strength Limit State**

*Table 15-5*

The load factors provided in Table 15-5 were determined assuming that the appropriate mean soil friction angle is used for design. In practice, since the specific source of material for wall backfill is typically not available at the time of design, presumptive design parameters based on previous experience with the material that is typically supplied to meet the backfill material specification (e.g., Gravel Borrow per the WSDOT Standard Specifications for construction) are used (see Chapter 5). It is likely that these presumptive design parameters are lower bound conservative values for the backfill material specification selected.

Other loads appropriate to the load groups and limit states to be considered as specified in the AASHTO LRFD specifications for wall design are applicable when using the K-Stiffness Method for design. Note that for seismic design (Extreme Event I), a load factor of 1.0 should be used for the total load combination (static plus seismic loads) acting on the soil reinforcement.

#### **15.5.3.9.3 K-Stiffness Method Resistance Factors**

For the service limit state, a resistance factor of 1.0 should be used, except for the evaluation of overall slope stability as prescribed by the AASHTO LRFD specifications (see also Section 15.4.12). For the strength and extreme event limit states for internal stability using the K-Stiffness Method, the resistance factors provided in Table 15-6 shall be used as maximum values. These resistance factors were derived using the data provided in Allen and Bathurst (2003). Reliability theory, using the Monte Carlo Method as described in Allen, et al. (2005) was applied to statistically characterize the data and to estimate resistance factors. The load factors provided in Table 15-5 were used for this analysis.

The resistance factors, specified in Table 15-6 are consistent with the use of select granular backfill in the reinforced zone, homogeneously placed and carefully controlled in the field for conformance with the WSDOT Standard Specifications. The resistance factors provided in Table 15-6 have been developed with consideration to the redundancy inherent in MSE walls due to the multiple reinforcement layers and the ability of those layers to share load one with another. This is accomplished by using a target reliability index,  $\beta$ , of 2.3 (approximate probability of failure,  $P_f$ , of 1 in 100 for static conditions) and a  $\beta$  of 1.65 (Approximate  $P_f$  of 1 in 20) for seismic conditions. A  $\beta$  of 3.5 (approximate  $P_f$  of 1 in 5,000) is typically used for structural design when redundancy is not considered or not present; see Allen et al. (2005) for additional discussion on this issue. Because redundancy is already taken into account through the target value of  $\beta$  selected, the factor  $\eta$  for redundancy prescribed in the AASHTO LRFD specifications should be set equal to 1.0. The target value of  $\beta$  used herein for seismic loading is consistent with the overstress allowed in previous practice as described in the AASHTO *Standard Specifications for Highway Bridges* (AASHTO 2002).

#### **15.5.3.9.4 Safety Against Structural Failure (Internal Stability)**

Safety against structural failure shall consider all components of the reinforced soil wall, including the soil reinforcement, soil backfill, the facing, and the connection between the facing and the soil reinforcement, evaluating all modes of failure, including pullout and rupture of reinforcement.

A preliminary estimate of the structural size of the stabilized soil mass may be determined on the basis of reinforcement pullout beyond the failure zone, for which resistance is specified in Article 11.10.6.3 of the AASHTO *LRFD Bridge Design Specifications*.

The load in the reinforcement shall be determined at two critical locations: the zone of maximum stress and the connection with the wall face. Potential for reinforcement rupture and pullout are evaluated at the zone of maximum stress, which is assumed to be located at the boundary between the active zone and the resistant zone in Figure 11.10.2-1 of the AASHTO *LRFD Bridge Design Specifications*. Potential for reinforcement rupture and pullout are also evaluated at the connection of the reinforcement to the wall facing. The reinforcement shall also be designed to prevent the backfill soil from reaching a failure condition.

Limit State and Reinforcement Type			Resistance Factor
Internal Stability of MSE Walls, K-Stiffness Method			
$\phi_{rr}$	Reinforcement Rupture	Metallic Geosynthetic	0.85 0.85 <sup>(3)</sup>
$\phi_{sf}$	Soil Failure	Metallic Geosynthetic	0.85 1.00 <sup>(1)</sup>
$\phi_{cr}$	Connection rupture	Metallic Geosynthetic	0.85 0.80 <sup>(3)</sup>
$\phi_{po}$	Pullout <sup>(2)</sup>	Steel ribbed strips (at $z < 2$ m)	1.10
		Steel ribbed strips (at $z > 2$ m)	1.00
		Steel smooth strips	1.00
		Steel grids	0.60
		Geosynthetic	0.80
$\phi_{EQr}$	Combined static/earthquake loading (reinforcement and connector rupture)	Metallic Geosynthetic	1.00 0.85 <sup>(3)</sup>
$\phi_{EQp}$	Combined static/earthquake loading (pullout) <sup>(2)</sup>	Steel ribbed strips (at $z < 2$ m)	1.25
		Steel ribbed strips (at $z > 2$ m)	1.15
		Steel smooth strips	1.15
		Steel grids	0.75
		Geosynthetic	0.80

<sup>(1)</sup>If default value for the critical reinforcement strain of 3.0 percent or less is used for flexible wall facings, and 2.0 percent or less for stiff wall facings (for a facing stiffness factor of less than 0.9).

<sup>(2)</sup>Resistance factor values in table for pullout assume that the default values for  $F^*$  and  $\alpha$  provided in Article 11.10.6.3.2 of the AASHTO LRFD Specifications are used and are applicable.

<sup>(3)</sup>This resistance factor applies if installation damage is not severe (i.e.,  $RF_{ID} < 1.7$ ). Severe installation damage is likely if very light weight reinforcement is used. Note that when installation damage is severe, the resistance factor needed for this limit state can drop to approximately 0.15 or less due to greatly increased variability in the reinforcement strength, which is not practical for design.

### Resistance Factors for the Strength and Extreme Event Limit States for MSE Walls Designed Using the K-Stiffness Method

Table 15-6

Loads carried by the soil reinforcement in mechanically stabilized earth walls are the result of vertical and lateral earth pressures, which exist within the reinforced soil mass, reinforcement extensibility, facing stiffness, wall toe restraint, and the stiffness and strength of the soil backfill within the reinforced soil mass. The soil reinforcement extensibility and material type are major factors in determining reinforcement load. In general, inextensible reinforcements consist of metallic strips, bar mats, or welded wire mats, whereas extensible reinforcements consist of geotextiles or geogrids. Internal stability failure modes include soil reinforcement rupture or failure of the backfill soil (strength or extreme event limit state), and excessive reinforcement elongation under the design load (service limit state). Internal stability is determined by equating the factored tensile load applied to the reinforcement to the factored tensile resistance of the reinforcement, the tensile resistance being governed by reinforcement rupture and pullout. Soil backfill failure is prevented by keeping the soil shear strain below its peak shear strain.

#### **15.5.3.9.5 Strength Limit State Design for Internal Stability Using the K-Stiffness Method – Geosynthetic Walls**

For geosynthetic walls, four strength limit states (soil failure, reinforcement failure, connection failure, and reinforcement pullout) must be considered for internal reinforcement strength and stiffness design. The design steps, and related considerations, are as follows:

1. Select a trial reinforcement spacing,  $S_v$ , and stiffness,  $J_{EOC}$ , based on the time required to reach the end of construction (EOC). If the estimated time required to construct the wall is unknown, an assumed construction time of 1,000 hours should be adequate. Note that at this point in the design, it does not matter how one obtains the stiffness. It is simply a value that one must recognize is an EOC stiffness determined through isochronous stiffness curves at a given strain and temperature, and that it represents the stiffness of a continuous reinforcement layer on a per foot of wall width basis. Use the selected stiffness to calculate the trial global stiffness of the wall,  $S_{global}$ , using Equation 15-3, with  $J_{EOC}$  equal to  $J_i$  for each layer. Also select a soil friction angle for design (see Section 15.5.3.9.1). Once the design soil friction angle has been obtained, the lateral earth pressure coefficients needed for determination of  $T_{max}$  (Step 4) can be determined (see Section 15.5.3.9.1). Note that if the reinforcement layer is intended to have a coverage ratio,  $R_c$ , of less than 1.0 (i.e., the reinforcement is to be discontinuous), the actual product selected based on the K-Stiffness design must have a stiffness of  $J_{EOC}(1/R_c)$ .
2. Begin by checking the strength limit state for the backfill soil. The goal is to select a stiffness that is large enough to prevent the soil from reaching a failure condition.
3. Select a target reinforcement strain,  $\epsilon_{targ}$ , to prevent the soil from reaching its peak shear strain. The worst condition in this regard is a very strong, high peak friction angle soil, as the peak shear strain for this type of soil will be lower than the peak shear strain obtained from most backfill soils. The results of full-scale wall laboratory testing showed that the reinforcement strain at which the soil begins to exhibit signs of failure is on the order of 3 to 4 percent for high shear strength sands (Allen and Bathurst, 2003). This empirical evidence reflects very high shear strength soils and is probably a worst case for design purposes, in that most soils will have larger peak shear strain values than the soils tested in the full-scale walls.

A default value for  $\epsilon_{\text{targ}}$  adequate for granular soils is 3 percent for flexible faced walls, and 2 percent for stiff faced walls if a  $\Phi_{\text{fs}}$  of less than 0.9 is used for design. Lower target strains could also be used, if desired.

4. Calculate the factored load  $T_{\text{max}}$  for each reinforcement layer (Equation 15-2). To determine  $T_{\text{max}}$ , the facing type, dimensions, and properties must be selected to determine  $\Phi_{\text{fs}}$ . The local stiffness factor  $\Phi_{\text{local}}$  for each layer can be set to 1.0, unless the reinforcement spacing or stiffness within the design wall section is specifically planned to be varied. The global wall stiffness,  $S_{\text{global}}$ , and global stiffness factor,  $\Phi_{\text{g}}$ , must be estimated from  $J_{\text{EOC}}$  determined in Step 1.
5. Estimate the factored strain in the reinforcement at the end of the wall design life,  $\epsilon_{\text{rein}}$ , using the K-Stiffness Method as follows:

$$\epsilon_{\text{rein}} = \left( \frac{T_{\text{max}}}{J_{\text{DL}} \phi_{\text{sf}}} \right) \quad (15-15)$$

Where:

$T_{\text{max}}$  is the factored reinforcement load from Step 4,  $J_{\text{DL}}$  is the reinforcement layer stiffness at the end of the wall design life (typically 75 years for permanent structures) determined with consideration to the anticipated long-term strain in the reinforcement (i.e.,  $\epsilon_{\text{targ}}$ ),  $\phi_{\text{sf}}$  is the resistance factor to account for uncertainties in the target strain, and other variables are as defined previously. If a default value of  $\epsilon_{\text{targ}}$  is used, a resistance factor of 1.0 will be adequate.

6. If  $\epsilon_{\text{rein}}$  is greater than  $\epsilon_{\text{targ}}$ , increase the reinforcement layer stiffness  $J_{\text{EOC}}$  and recalculate  $T_{\text{max}}$  and  $\epsilon_{\text{rein}}$ .  $J_{\text{EOC}}$  will become the stiffness used for specifying the material if the reinforcement layer is continuous (i.e.,  $R_c = 1$ ). Note that if the reinforcement layer is intended to have a coverage ratio,  $R_c$ , of less than 1.0 (i.e., the reinforcement is to be discontinuous), the actual product selected based on the K-Stiffness design must have a stiffness of  $J_{\text{EOC}}(1/R_c)$ . For final product selection,  $J_{\text{EOC}}(1/R_c)$  shall be based product specific isochronous creep data obtained in accordance with AASHTO PP66-10 at the estimated wall construction duration (1,000 hours is an acceptable default time if a specific construction duration of the wall cannot be estimated at time of design) and site temperature. Select the stiffness at the anticipated maximum working strains for the wall, as the stiffness is likely to be strain level dependent. For design purposes, a 2 percent secant stiffness at the wall construction duration from the beginning of wall construction to the end of wall construction (EOC) is the default strain. If strains of 3 percent are anticipated, determine the stiffness at the higher strain level. If strains of significantly less than 2 percent are anticipated, and a geosynthetic material is being used that is known to have a highly non-linear load-strain curve over the strain range of interest (e.g., some PET geosynthetics), then a stiffness value determined at a lower strain should be obtained. Otherwise, just determine the stiffness at 2 percent strain. This recognizes the difficulties of accurately measuring the stiffness at very low strains. Note that for calculating  $T_{\text{max}}$ , if multifilament woven geotextiles are to be used as the wall reinforcement, the stiffness values obtained from laboratory isochronous creep data should be increased by 15 percent to account for soil confinement effects. If nonwoven geotextiles are planned to be used as wall

reinforcement,  $J_{EOC}$  and  $J_{DL}$  shall be based on confined in soil isochronous creep data, and use of nonwoven geotextiles shall be subject to the approval of the State Geotechnical Engineer.

7. Next, check the strength limit state for reinforcement rupture in the backfill. The focus of this limit state is to ensure that the long-term factored rupture strength of the reinforcement is greater than the factored load calculated from the K-Stiffness Method.  $T_{max}$  calculated from Step 4 is a good starting point for evaluating this limit state. Note that the global wall stiffness for this calculation is based on the EOC stiffness of the reinforcement, as the reinforcement loads should still be based on EOC conditions, even though the focus of this calculation is at the end of the service life for the wall.
8. Calculate the strength reduction factors  $RF_{ID}$ ,  $RF_{CR}$ , and  $RF_D$  for the reinforcement type selected using the approach prescribed in AASHTO PP66-10. Because the focus of this calculation is to prevent rupture, these factors must be based on reinforcement rupture. Applying a resistance factor to address uncertainty in the reinforcement strength, determine  $T_{ult}$ , the ultimate tensile strength of the reinforcement as follows:

$$T_{max} \leq \frac{T_{ult} \phi_{rr} R_c}{RF_{ID} RF_{CR} RF_D} \quad (15-16)$$

Where:

$T_{max}$  is the factored reinforcement load,  $\phi_{rr}$  is the resistance factor for reinforcement rupture,  $R_c$  is the reinforcement coverage ratio,  $RF_{ID}$ ,  $RF_{CR}$ , and  $RF_D$  are strength reduction factors for installation damage, creep, and durability, respectively, and the other the variables are as defined previously. The strength reduction factors should be determined using product and site specific data when possible (AASHTO, 2010; WSDOT, 2009).  $T_{ult}$  is determined from an index wide-width tensile test such as ASTM D4595 or ASTM D6637 and is usually equated to the MARV for the product.

9. Step 8 assumes that a specific reinforcement product will be selected for the wall, as the strength reduction factors for installation damage, creep, and durability are known at the time of design. If the reinforcement properties will be specified generically to allow the contractor or wall supplier to select the specific reinforcement after contract award, use the following equation the long-term design strength of the reinforcement,  $T_{al\text{design}}$ :

$$T_{al\text{design}} = \frac{T_{max}}{\phi_{rr} R_c} \quad (15-17)$$

Where:

$T_{max}$  is the factored reinforcement load from Step 6. The contractor can then select a product with the required  $T_{al\text{design}}$ .

10. If the geosynthetic reinforcement is connected directly to the wall facing (this does not include facings that are formed by simply extending the reinforcement mat), the reinforcement strength needed to provide the required long-term connection strength must be determined. Determine the long-term connection strength ratio

$CR_{cr}$  at each reinforcement level, taking into account the available normal force between the facing blocks, if the connection strength is a function of normal force.  $CR_{cr}$  is calculated or measured directly per the AASHTO LRFD Specifications.

- Using the unfactored reinforcement load from Step 6 and an appropriate load factor for the connection load to determine  $T_{max}$  (factored) at the connection, determine the adequacy of the long-term reinforcement strength at the connection. Compare the factored connection load at each reinforcement level to the available factored long-term connection strength as follows:

$$T_{max} \leq \phi_{cr} T_{ac} R_c = \frac{\phi_{cr} T_{ult} CR_{cr} R_c}{RF_D} \quad (15-18)$$

- It must be recognized that the strength ( $T_{ult}$  and  $T_{al}$ ) and stiffness ( $J_{EOC}$ ) determined from the K-Stiffness Method could result in the use of very light weight geosynthetics. In no case shall geosynthetic reinforcement be used that has an  $RF_{ID}$  applicable to the anticipated soil backfill gradation and installation conditions anticipated of greater than 1.7, as determined per AASHTO PP66-10. Furthermore, reinforcement coverage ratios,  $R_c$ , of less than 1.0 may be used provided that it can be demonstrated the facing system is fully capable of transmitting forces from unreinforced segments laterally to adjacent reinforced sections through the moment capacity of the facing elements. For walls with modular concrete block facings, the gap between soil reinforcement sections or strips at a horizontal level shall be limited to a maximum of one block width in accordance with the AASHTO LRFD Specifications, to limit bulging of the facing between reinforcement levels or build-up of unacceptable stresses that could result in performance problems. Also, vertical spacing limitations in the AASHTO LRFD Specifications for MSE walls apply to walls designed using the K-Stiffness method.
- Determine the length of the reinforcement required in the resisting zone by comparing the factored  $T_{max}$  value to the factored pullout resistance available as calculated per the AASHTO LRFD Specifications. If the length of the reinforcement required is greater than desired (typically, the top of the wall is most critical), decrease the spacing of the reinforcement, recalculate the global wall stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.

#### **15.5.3.9.6 Strength Limit State Design for Internal Stability Using the K-Stiffness Method – Steel Reinforced Walls**

For steel reinforced soil walls, four strength limit states (soil failure, reinforcement rupture, connection rupture, and pullout) shall be evaluated for internal reinforcement strength and stiffness design. The design steps and related considerations are as follows:

- Select a trial reinforcement spacing and steel area that is based on end-of-construction (EOC) conditions (i.e., no corrosion). Once the trial spacing and steel area have been selected, the reinforcement layer stiffness on a per foot of wall width basis,  $J_{EOC}$ , and wall global stiffness,  $S_{global}$ , can be calculated (Equation 15-3). Note that at this point in the design, it does not matter how one obtains the reinforcement spacing and area. They are simply starting points for the calculation. Also select a design soil friction angle to calculate K (see Section 15.5.3.9.1). Note

that for steel reinforced wall systems, the reinforcement loads are not as strongly correlated to the peak plane strain soil friction angle as are the reinforcement loads in geosynthetic walls (Allen and Bathurst, 2003). This is likely due to the fact that the steel reinforcement is so much stiffer than the soil. The K-Stiffness Method was calibrated to a mean value of  $K_0$  of 0.3 (this results from a plane strain soil friction angle of  $44^\circ$ , or from triaxial or direct shear testing a soil friction angle of approximately  $40^\circ$ ). Therefore, soil friction angles higher than  $44^\circ$  shall not be used. Lower design soil friction angles should be used for weaker granular backfill materials.

2. Begin by checking the strength limit state for backfill soil failure. The goal is to select a reinforcement density (spacing, steel area) that is great enough to keep the steel reinforcement load below yield ( $A_s F_y R_c / b$ , which is equal to  $A_s F_y / S_h$ ).  $F_y$  is the yield stress for the steel,  $A_s$  is the area of steel before corrosion (EOC conditions), and  $S_h$  is the horizontal spacing of the reinforcement (use  $S_h = 1.0$  for continuous reinforcement). Depending on the ductility of the steel, once the yield stress has been exceeded, the steel can deform significantly without much increase in load and can even exceed the strain necessary to cause the soil to reach a failure condition. For this reason, it is prudent to limit the steel stress to  $F_y$  for this limit state. Tensile tests on corroded steel indicate that the steel does not have the ability to yield to large strains upon exceeding  $F_y$ , as it does in an uncorroded state, but instead fails in a brittle manner (Terre Armee, 1979). Therefore, this limit state only needs to be evaluated for the steel without corrosion effects.
3. Using the trial steel area and global wall stiffness from Step 1, calculate the factored  $T_{\max}$  for each reinforcement layer using Equations 15-1 and 15-2.
4. Apply an appropriate resistance factor to  $A_s F_y / S_h$  to obtain the factored yield strength for the steel reinforcement. Then compare the factored load to the factored resistance, as shown in Equation 15-19 below. If the factored load is greater than the factored yield strength, then increase  $A_s$  and recalculate the global wall stiffness and  $T_{\max}$ . Make sure that the factored yield strength is greater than the factored load before going to the next limit state calculation. In general, this limit state will not control the design. If the yield strength available is well in excess of the factored load, it may be best to wait until the strength required for the other limit states has been determined before reducing the amount of reinforcement in the wall. Check to see that the factored reinforcement load  $T_{\max}$  is greater than or equal to the factored yield resistance as follows:

$$T_{\max} \leq \frac{A_s F_y}{b} R_c \phi_{sf} = \frac{A_s F_y}{S_h} \phi_{sf} \quad (15-19)$$

Where:

$\phi_{sf}$  is the resistance factor for steel reinforcement resistance at yield, and  $S_h$  is the horizontal spacing of the reinforcement. For wire mesh, and possibly some welded wire mats with large longitudinal wire spacing, the stiffness of the reinforcement macro-structure could cause the overall stiffness of the reinforcement to be significantly less than the stiffness of the steel itself. In-soil pullout test data may be used in that case to evaluate the soil failure limit state, and applied to the approach provided for soil failure for geosynthetic walls (see Equation 15-15 in Step 5 for geosynthetic wall design).

5. Next, check the strength limit state for reinforcement rupture in the backfill. The focus of this limit state is to ensure that the long-term rupture strength of the reinforcement is greater than the load calculated from the K-Stiffness Method. Even though the focus of this calculation is at the end of the service life for the wall, the global stiffness for the wall should be based on the stiffness at the end of wall construction, as reinforcement loads do not decrease because of lost cross-sectional area resulting from reinforcement corrosion.  $T_{\max}$  obtained from Step 5 should be an adequate starting point for this limit state calculation.
6. Calculate the strength of the steel reinforcement at the end of its service life, using the ultimate strength of the steel,  $F_u$ , and reducing the steel cross-sectional area,  $A_s$ , determined in Step 5, to  $A_c$  to account for potential corrosion losses. Then use the resistance factor  $\phi_{rr}$ , as defined previously, to obtain the factored long-term reinforcement tensile strength such that  $T_{al}$  is greater than or equal to  $T_{\max}$ , as shown below:

$$T_{al} = \frac{F_u A_c}{S_h} \phi_{rr} \quad (15-20)$$

and

$$T_{\max} \leq \frac{F_u A_c}{b} R_c \phi_{rr} = \frac{F_u A_c}{S_h} \phi_{rr} \quad (15-21)$$

Where:

$F_u$  is the ultimate tensile strength of the steel, and  $A_c$  is the steel cross-sectional area per FT of wall length reduced to account for corrosion loss. The resistance factor is dependent on the variability in  $F_u$ ,  $A_s$ , and the amount of effective steel cross-sectional area lost as a result of corrosion. As mentioned previously, minimum specification values are typically used for design with regard to  $F_u$  and  $A_s$ . Furthermore, the corrosion rates provided in the AASHTO LRFD Specifications are also maximum rates based on the available data (Terre Armee, 1991). Recent post-mortem evaluations of galvanized steel in reinforced soil walls also show that AASHTO design specification loss rates are quite conservative (Anderson and Sankey, 2001). Furthermore, these corrosion loss rates have been correlated to tensile strength loss, so that strength loss due to uneven corrosion and pitting is fully taken into account. Therefore, the resistance factor provided in Table 15-6, which is based on the variability of the un-aged steel, is reasonable to use in this case, assuming that non-aggressive backfill conditions exist.

If  $T_{al}$  is not equal to or greater than  $T_{\max}$ , increase the steel area, recalculate the global wall stiffness on the basis of the new value of  $A_s$ , reduce  $A_s$  for corrosion to obtain  $A_c$ , and recalculate  $T_{\max}$  until  $T_{al}$  based on Equation 15-21 is adequate to resist  $T_{\max}$ .

7. If the steel reinforcement is connected directly to the wall facing (this does not include facings that are formed by simply extending the reinforcement mat), the reinforcement strength needed to provide the required long-term connection strength must be determined. This connection capacity, reduced by the appropriate resistance factor, must be greater than or equal to the factored reinforcement load at the connection. If not, increase the amount of reinforcing steel in the wall, recalculate the global stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.
8. Determine the length of reinforcement required in the resisting zone by comparing the factored  $T_{\max}$  value to the factored pullout resistance available as calculated per Section 11 of the AASHTO LRFD specifications. If the length of reinforcement required is greater than desired (typically, the top of the wall is most critical), decrease the spacing of the reinforcement, recalculate the global wall stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.

#### **15.5.3.9.7 Combining Other Loads With the K-Stiffness Method Estimate of $T_{\max}$ for Internal Stability Design**

**Seismic Loads** – Seismic design of MSE walls when the K-Stiffness Method is used for internal stability design shall be conducted in accordance with Articles 11.10.7.2 and 11.10.7.3 of the AASHTO LRFD Specifications, except that the static portion of the reinforcement load is calculated using the K-Stiffness Method. The seismic load resulting from the inertial force of the wall active zone within the reinforced soil mass ( $T_{\text{md}}$  in AASHTO LRFD Article 11.10.7.3) is added to  $T_{\max}$  calculated using the K-Stiffness Method by superposition. A load factor of 1.0 for the load combination (static plus seismic), and the resistance factors for combined seismic and static loading provided in Table 15-6 shall be used for this Extreme Event Limit State.

**Concentrated Surcharges and Traffic Barrier Impact Loads** – The load increase at each reinforcement layer resulting from the concentrated surcharge and traffic barrier impact loads calculated as specified in the AASHTO LRFD Design Specifications, Articles 3.11.6.3 and 11.10.10 and Sections 15.5.3.4 and 15.4.15, shall be added to the K-Stiffness calculation of  $T_{\max}$  by superposition at each affected reinforcement level, considering the tributary area of the reinforcement. The load factor used for each load due to the surcharge or traffic impact load shall be as specified in the AASHTO LRFD *Bridge Design Specifications*.

#### **15.5.3.9.8 Design Sequence Considerations for the K-Stiffness Method**

A specific sequence of design steps has been proposed herein to complete the internal stability design of reinforced soil walls. Because global wall stiffness is affected by changes to the reinforcement design to meet various limit states, iterative calculations may be necessary. Depending on the specifics of the wall and reinforcement type, certain limit states may tend to control the amount of reinforcement required. It may therefore be desirable to modify the suggested design sequence to first calculate the amount of reinforcement needed for the limit state that is more likely to control the amount of reinforcement. Then perform the calculations for the other limit states to ensure that the amount of reinforcement is adequate for all limit states. Doing this will hopefully reduce the number of calculation iterations.

For example, for geosynthetic reinforced wrap-faced walls, with or without a concrete fascia placed after wall construction, the reinforcement needed to prevent soil failure will typically control the global reinforcement stiffness needed, while pullout capacity is generally not a factor, and connection strength is not applicable. For modular concrete block-faced or precast panel-faced geosynthetic walls, the connection strength needed is likely to control the global reinforcement stiffness. However, it is also possible that reinforcement rupture or soil failure could control instead, depending on the magnitude of the stiffness of a given reinforcement product relative to the long-term tensile strength needed. The key here is that the combination of the required stiffness and tensile strength be realistic for the products available. Generally, pullout will not control the design unless reinforcement coverage ratios are low. If reinforcement coverage ratios are low, it may be desirable to evaluate pullout early in the design process. For steel strip, bar mat, wire ladder, and polymer strap reinforced systems, pullout often controls the reinforcement needed because of the low reinforcement coverage ratios used, especially near the top of the wall. However, connection strength can also be the controlling factor. For welded wire wall systems, the tensile strength of the reinforcement usually controls the global wall reinforcement stiffness needed, though if the reinforcement must be connected to the facing (i.e., the facing and the reinforcement are not continuous), connection strength may control instead. Usually, coverage ratios are large enough for welded wire systems (with the exception of ladder strip reinforcement) that pullout is not a controlling factor in the determination of the amount of reinforcement needed. For all steel reinforced systems, with the possible exception of steel mesh reinforcement, the soil failure limit state does not control the reinforcement design because of the very low strain that typically occurs in steel reinforced systems.

#### **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:

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

### Minimum Rock Sizes for Rock Walls

Table 15-7

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. The design and construction shall be in accordance with the General Special Provisions.

### 15.5.7 Soil Nail Walls

Soil Nail walls are not specifically addressed by the ASHTO *LRFD Bridge Design Specifications*. Soil nail walls shall be designed for internal stability by the geotechnical designer using Gold Nail version 3.11 or SNail version 2.11 or later versions of these programs and the following manuals:

- Lazarte, C. A., Elias, V., Espinoza, R. D., Sabatini, P. J., 2003. *Geotechnical Engineering Circular No. 7, Soil Nail Walls*, U.S. Department of Transportation, Federal Highway Administration, FHWA-IF-03-017, 305 pp.
- Byrne, R. J., Cotton, D., Porterfield, J., Wolschlag, C., and Ueblacker, G., 1996, *Demonstration Project 103, Manual for Design and Construction Monitoring of Soil Nail Walls*, Federal Highway Administration, FHWA-SA-96-069, 468 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.

The LRFD procedures described in the *Manual for Design and Construction Monitoring of Soil Nail Walls*, FHWA-SA-96-069, shall not be used.

For external stability and compound stability analysis, as described in Section 15.5.3.3 and the AASHTO *LRFD Bridge Design Specifications*, limit equilibrium slope stability programs as described in Chapter 7 should be used. The program SNail also has the ability to conduct compound stability analyses and may be used for this type of analysis as well.

When using SNail, the geotechnical designer should use the allowable option and shall pre-factor the yield strength of the nails, punching shear of the shotcrete, and the nail adhesion. Unfactored cohesion and friction angle shall be used and the analysis run to provide the minimum safety factors discussed above for overall stability.

When using GoldNail, the geotechnical designer should utilize the design mode and the safety factor mode of the program with the partial safety factors identified in the FHWA *Manual for Design and Construction Monitoring of Soil Nail Walls*, FHWA-SA-96-069.

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 such as those underpinning abutments.

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. For nail testing, a minimum bond length of 10 feet and a minimum unbonded length of 5 feet is required. 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.

## 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, 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,  $\phi = 36^\circ$  and  $\gamma = 130$  pcf.
- For the foundation soil, for sliding stability analysis,  $\phi = 32^\circ$ .
- 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,  $\phi = 38^\circ$  and  $\gamma = 130$  pcf.
- For the foundation soil, for sliding stability analysis,  $\phi = 36^\circ$ , and interface friction angle of  $0.7 \times 36^\circ = 25^\circ$ .
- For the retained soil behind the soil reinforcement, for external stability analysis,  $\phi = 36^\circ$  and  $\gamma = 130$  pcf.
- Wall settlement criteria are as specified in Table 15-2.

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 for Road, Bridge, and Municipal Construction* M 41-01 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. The resistance factor for global stability should be 0.65 if the temporary shoring system is supporting another structure such as a bridge, building, or major retaining wall (factor of safety of 1.5 for wall types in which LRFD procedures are not available) and 0.75 if the shoring system is not supporting another structure (factor of safety of 1.3 for wall types in which LRFD procedures are not available). For soil nail walls, the safety factors provided in the FHWA manuals identified herein shall be used.

For design of cut slopes that are part of a temporary excavation, assuming that the cut slopes not supporting a structure, 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 surcharge 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 15-8. 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.

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

**WAC 296-155 Allowable Temporary Cut Slopes**

*Table 15-8*

**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 10 feet in height shall be designed by a registered civil engineer (geotechnical engineer) in accordance with the GDM. 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.

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 2010).

### 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-E.

#### 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 seepage 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 cofferdams
- Ground 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-E.

### 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-E 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

AASHTO, 2002, *Standard Specifications for Highway Bridges*, American Association of State Highway and Transportation Officials, 17th Edition, Washington, D.C., USA.

AASHTO, 2010, *LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Fifth Edition, Washington, D.C., USA.

AASHTO, 2010, *Provisional Standard PP66-10: Determination of Long-Term Strength of Geosynthetic Reinforcement*, American Association of State Highway and Transportation Officials, Inc., Washington, D.C.

Allen, T. M., and Bathurst, R. J., 2003, *Prediction of Reinforcement Loads in Reinforced Soil Walls*, WSDOT Research Report WA-RD 513.1 WA-RD 522.2, 364 pp.

Allen, T. M., Bathurst, R. J., Holtz, R. D., Walters, D. L. and Lee, W. F., 2003. "A New Working Stress Method for Prediction of Reinforcement Loads in Geosynthetic Walls," *Canadian Geotechnical Journal*, Vol. 40, No. 5, pp. 976-994.

- Allen, T. M., Bathurst, R. J., Lee, W. F., Holtz, R. D., and Walters, D. L., 2004, "A New Method for Prediction of Loads in Steel Reinforced Walls," *ASCE Journal of Geotechnical and Geo-environmental Engineering*, Vol. 130, No. 11, pp. 1109-1120.
- Allen, T. M., Nowak, A. S., and Bathurst, R. J., 2005, *Calibration to Determine Load and Resistance Factors for Geotechnical and Structural Design*, Transportation Research Board Circular E-C079, Washington, D.C., 93 p.
- Allen, T. M., Christopher, B. R., Elias, V., and DiMaggio, J. D., 2001, *Development of the Simplified Method for Internal Stability Design of Mechanically Stabilized Earth (MSE) Walls*, WSDOT Research Report WA-RD 513.1, 96 pp.
- Anderson, P. L., and Sankey, J., 2001, "The Performance of Buried Galvanized Steel Reinforcements after 20 Years in Service," Kyushu 2001, Japan, pp. \_\_\_.
- Bathurst, R. J., Allen, T. M., and Nowak, A. S., 2008a, "Calibration Concepts for Load and Resistance Factor Design (LRFD) of Reinforced Soil Walls," *Canadian Geotechnical Journal*, Vol. 45, pp. 1377-1392.
- Bathurst, R. J., Miyata, Y., Nernheim, A., and Allen, T. M., 2008b, "Refinement of K-stiffness Method for Geosynthetic Reinforced Soil Walls," *Geosynthetics International*, Vol. 15, No. 4, pp. 269-295.
- Bathurst, R. J., Nernheim, A., and Allen, T. M., 2009. "Predicted Loads in Steel Reinforced Soil Walls Using the AASHTO Simplified Method," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 135, No. 2, pp. 177-184.
- 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.
- Bligh, R. P., Briaud, J.-L., Kim, K. M., and Abu-Odeh, A., 2010, *Design of Roadside Barrier Systems Placed on MSE Retaining Walls*, NCHRP Report 663, Transportation Research Board, Washington, D.C., 195 pp.
- Bolton, M. D. 1986, "The Strength and Dilatancy of Sands," *Geotechnique*, Vol. 36, No. 1, pp. 65-78.
- Byrne, R. J., Cotton, D., Porterfield, J., Wolschlag, C., and Ueblacker, G., 1996, *Demonstration Project 103, Manual for Design & Construction Monitoring of Soil Nail Walls*, Federal Highway Administration, FHWA-SA-96-069, 468 pp.
- Chugh, A. K., 1995, "A Unified Procedure for Earth Pressure Calculations," *Proceedings: Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, Vol. III, St. Luis, Missouri.
- Elias, V., Fishman, K.L., Christopher, B. R., and Berg, R. R., 2009, *Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*, No. FHWA-NHI-09-087, Federal Highway Administration, 144 pp.
- Jewell, R. A., and Wroth, C. P., 1987, "Direct Shear Tests on Reinforced Sand," *Geotechnique*, Vol. 37, No. 1, pp. 53-68.
- Lade and Lee, 1976, "Engineering Properties of Soils," *Report UCLA-ENG-7652*, 145 pp. [As cited by Holtz and Kovacs (1981)].

- Lazarte, C. A., Elias, V., Espinoza, R. D., Sabatini, P. J., 2003. *Geotechnical Engineering Circular No. 7, Soil Nail Walls*, U.S. Department of Transportation, Federal Highway Administration, FHWA-IF-03-017, 305 pp.
- Mack, D.A., Sanders, S.H., Millhone, W.L., Fippin, R.L., and Kennedy, D.G., 2006. *Rockery Design and Construction Guidelines*, FHWA Manual No. FHWA-CFL/TD-06-006, 178 pp.
- Morrison, K. F., Harrison, F. E., Collin, J. G., Dodds, A., and Arndt, B., 2006. *Shored Mechanically Stabilized Earth (SMSE) Wall Systems*, Technical Report, Federal Highway Administration, Washington, D.C., Report No. FHWA-CFL/TD-06-001, 212 pp.
- NAVFAC, 1982, *Design Manual: Foundations, and Earth Structures*, DM-7.2.
- 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.
- 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.
- 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.
- Santamarina, J. C., Altschaeffl, A. G., and Chameau, J. L., 1992, "Reliability of Slopes: Incorporating Qualitative Information," Transportation Research Board, TRR 1343, Washington, D.C., pp. 1-5.
- Terre Armee, 1979, *Experimental Wall Pushed to Break by Corrosion of the Reinforcements*, TAI Research Report R12.
- US Department of Defense, 2005, *Soil Mechanics*, Unified Facilities Criteria (UFC), UFC 3-220-10N,



### 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 PP66-10.

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.

### 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-020. 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* M 41-10 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.

## 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 Table 15-A-1. More stringent requirements will likely be necessary depending on the assessment of backfill needs as described above. This is especially likely in western Washington regarding the fines content and overall gradation; hence Gravel Borrow per the *Standard Specifications* is recommended.

Sieve Size	Percent Passing
100 mm (4 in.) <sup>+</sup>	100
0.42 mm (No. 40)	0-60
0.074 mm (No. 200)	0-15

**Minimum Gradation Requirements for  
MSE Walls and Reinforced Slopes**

*Table 15-A-1*

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  $\geq 3000$  ohm-cm (AASHTO T288)
- Chlorides  $\leq 100$  ppm (AASHTO T291)
- Sulfates  $\leq 200$  ppm (AASHTO T290)
- Organic Content  $\leq 1$  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  $\leq 1.25$  inches, unless full scale installation damage tests are conducted in accordance with AASHTO PP66-10 so that the design can take into account the potential greater degree of damage.

Soils not meeting the requirements provided above shall not be used.

## 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:  $\frac{3}{4}$  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:  $\frac{1}{2}$  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  $\frac{3}{4}$  inch. The maximum allowable offset in any concrete block joint shall be  $\frac{3}{8}$  inch.

The design of the MSE wall (geosynthetic wrapped face, with or without a precast concrete, cast-in-place concrete, or shotcrete fascia 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

## References

AASHTO, 2010, *Provisional Standard PP66-10: Determination of Long-Term Strength of Geosynthetic Reinforcement*, American Association of State Highway and Transportation Officials, Inc., Washington, DC.

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.



# **Preapproved Proprietary Wall/Reinforced Slope Design and Construction Review Checklist**

## **Appendix 15-B**

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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 (note that until the proprietary wall suppliers have fully converted to LRFD, LFD or working stress design may be used as an alternative to the LRFD requirements identified below in the checklist – see [Chapter 15](#), Appendix 15-A for additional information on this issue):

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 (in general, the service limit state is not specifically checked for internal stability, 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, and Extreme Event I should be used for the extreme event limit state – seismic design)? (GT, ST)
    - 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,  $R_c$ , 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 K-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)

### 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 Division 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.

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.

### Part One: Wall System Overview

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

### 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
  - Acute outside 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
  - Precast barrier
  - 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.

### 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](http://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 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
  - Connection strength as a percent of reinforcement strength at various confining pressures for each reinforcement product, connection type, and facing unit.
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.)

## 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.

## 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.

## 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:

Wall Supplier	System Name	System Description	ASD/LFD or LRFD?	Height, or Other Limitations	Year Initially Approved	Last Approved Update
The Reinforced Earth Co. 8614 Westwood Center Dr. Suite 1100 Vienna, VA 22182 703-821-1175	Reinforced Earth Wall	Precast concrete 5'x5' facing panels and steel strip soil reinforcement	ASD/LFD	33 feet	1987	Approved 11/9/04 (submitted 3/29/04)
L.B. Foster Company Foster Geotechnical 1660 Hotel Circle North, Suite 304 San Diego, CA 92108-2803 619-688-2400	Retained Earth Wall	Precast concrete 5'x5' facing panels and steel bar mat soil reinforcement	ASD/LFD	33 feet	Unknown	Approved 11/9/04 (submitted 12/11/03)
SSL, LLC 4740 Scotts Valley Drive, Suite E Scotts Valley, CA 95066 831-430-9300	MSEPlus Wall	Precast concrete 5'x5' facing panels and steel welded wire strip soil reinforcement	<u>LRFD</u>	33 feet	1999	Approved 8/5/13 (submitted 5/28/13)
Tensar Earth Technologies, Inc. 5883 Glenridge Drive, Suite 200 Atlanta, GA 30328 404-250-1290	ARES Wall	Precast concrete 5'x5' facing panels and Tensar geogrid soil reinforcement	ASD/LFD	33 feet	1998	Approved 11/9/04 (submitted 8/6/04)
Hilfiker Retaining Walls 3900 Broadway PO Box 2012 Eureka, CA 95503-5707 707-443-5093	Eureka Reinforced Soil Wall	Precast concrete 5'x5' facing panels and welded wire mat soil reinforcement	ASD/LFD	33 feet	Unknown	Approved 11/9/04 (submitted 10/5/04)
Hilfiker Retaining Walls 3900 Broadway P.O. Box 2012 Eureka, CA 95503-5707 707-443-5093	Welded Wire Retaining Wall	Welded wire facing that is continuous with welded wire soil reinforcement	ASD/LFD	33 feet	Unknown	Approved 11/9/04 (submitted 9/15/03)
Keystone Retaining Wall Systems, Inc. 4444 West 78 <sup>th</sup> Street Minneapolis, MN 55435 952-897-1040	Key System I Wall	Modular dry cast concrete block facing with steel welded wire ladder strip soil reinforcement	ASD/LFD	33 feet	2001	Approved 11/9/04 (submitted 3/31/04)
Tensar Earth Technologies, Inc. 5883 Glenridge Drive, Suite 200 Atlanta, GA 30328 404-250-1290	MESA Wall	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)

Wall Supplier	System Name	System Description	ASD/LFD or LRFD?	Height, or Other Limitations	Year Initially Approved	Last Approved Update
Nelson Wall 12356 Northup Way, Suite 109 Bellevue, WA 98005 425-861-8292	Nelson Wall	Precast concrete gravity wall (similar to Standard Plan Concrete cantilever wall)	ASD/LFD	28 feet	1995	Approved 11/9/04 (submitted 9/12/03)
The Neel Company 8328-D Traford Lane Springfield, VA 22152 703-913-7858	T-WALL	Precast concrete modular wall	ASD/LFD	25 feet	1994	Approved 11/9/04 (submitted 11/05/04)
Tensar Earth Technologies, Inc. 5883 Glenridge Drive, Suite 200 Atlanta, GA 30328 404-250-1290	Welded Wire Form Wall	Tensar geogrid wrapped face wall with welded wire facing form	ASD/LFD	33 feet*	2006	Approved 3/3/06 (submitted 11/26/05)
Anchor Wall Systems, Inc. 5959 Baker Road Suite 390 Minnetonka, MN 55345-5996 952-933-8855	Landmark	Modular dry cast concrete block facing with Miragrid geogrid soil reinforcement	LRFD	33 feet	2012	Approved 4/2/12 (submitted 10/21/11)
Lock and Load Retaining Walls LTD 1681 Chestnut Street Suite 400 Vancouver, BC V6J 4M6 Canada 604-732-9990	Lock + Load Wall	Precast concrete panel facing attached to wrapped face geogrid wall	LRFD	33 feet	2013	Approved 7/10/13 (submitted 5/3/13)
Allan Block Corporation 7424 W. 78th Street Bloomington, MN 55439 952-835-5309	Allan Block Wall (battered face)	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)

\*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.

**Preapproved Proprietary Walls**  
**Table 15-D-1**

# **Description of Typical Temporary Shoring Systems and Selection Considerations**

## **Appendix 15-E**

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### **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.

### **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.

### **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.

## 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.

### 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.

### 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.

## 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.

## 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 area 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.

## 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.

## 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.

## Uncommon Shoring Systems for Cut Applications

The following shoring systems require special, very detailed, expert implementation:

## **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.

## **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.

## **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.

## **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  $-20^{\circ}\text{C}$  to  $-30^{\circ}\text{C}$ . 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.

## **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.

## **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.

## **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.

## **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.

### **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.

### **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).

## Soil Conditions

### **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.

### **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.

### **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.

### **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.

### **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.  $K_0$  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.

### **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.

### **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.

## 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.

## References

- Andrews, G., Squier, L., and Klassel, J., (1966) "Cylinder Pile Retaining Walls," Paper No. 295, Proceedings: ASCE Structural Conference; Miami, Florida, January 31, - February 4, 1966.
- Elias, V., and Christopher, B.R., and Berg, R. R., 2001, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Design and Construction Guidelines, No. FHWA-NHI-00-043, Federal Highway Administration, 394 pp.
- Peck, R., (1963) "Report on Evaluation of Stability Problems, Seattle Freeway South of Olive Way", Report to U.S. Bureau of Public Roads, July 1963.
- Ratay, R., (1996) Handbook of Temporary Structures in Construction; Engineering Standards, Designs, Practices and Procedures (Second Edition), McGraw-Hill, \_\_\_pp.
- Sherif, M., (1966) "Physical Properties of Seattle Freeway Soils," Contract No. GC-1530, Engineering Report No. 2, Prepared for Washington State Highway Department Division of Materials and Research.
- Strazer, R., Bestwick, K., and Wilson, S., (1974) "Design Considerations for Deep Retained Excavations in Over-Consolidated Seattle Clays," Association of Engineering Geology, Vol. XI, No. 4.



# ***Preapproved Wall Appendix: Specific Requirements and Details for LB Appendix 15-F Foster Retained Earth Concrete Panel Walls***

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In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply to the design of the LB Foster Retained 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 September 11, 2003. The design procedures used by LB Foster (specifically Foster Geotechnical) 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.

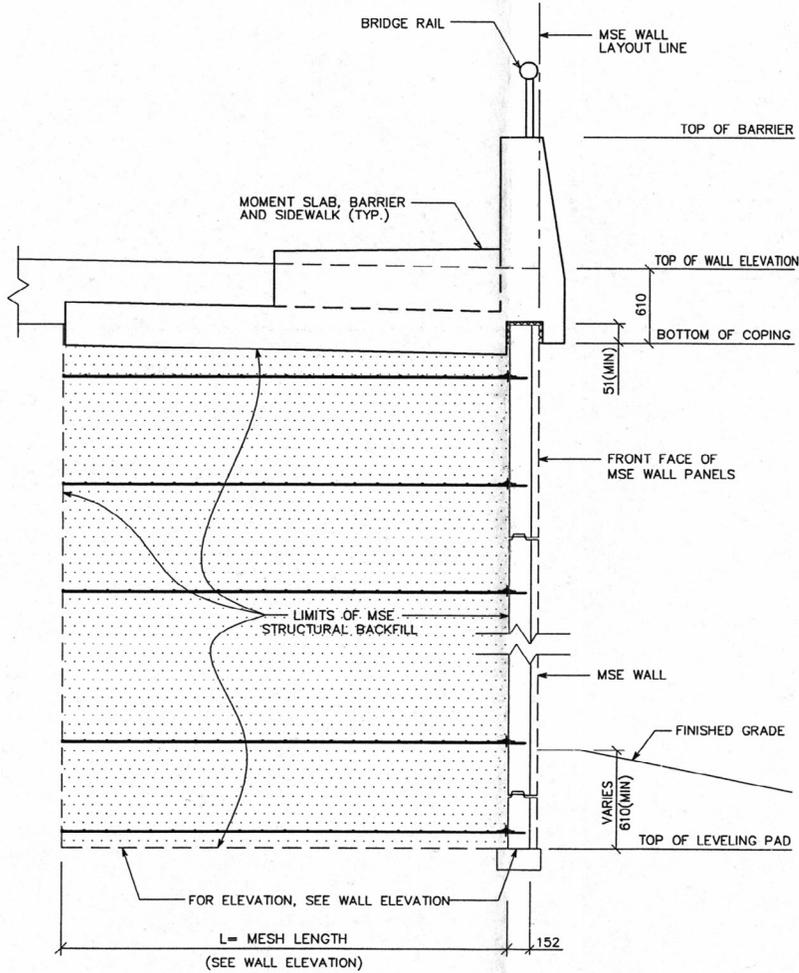
Note the connector shall be designed to have adequate life considering corrosion loss. Furthermore, the connector loops embedded in the facing panels shall be lined up such that the steel grid reinforcement cross bar at the connection is uniformly loaded. Therefore, regarding the alignment of the bearing surfaces of the embedded wire loops, once the steel grid is inserted into the loops, no loop shall have a gap between the loop and the steel 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 LB Foster Retained Earth™ concrete 5 feet × 5 feet panel faced retaining wall system are provided in the following plan sheets. Note that the two stage wall (i.e., welded wire face with concrete panels installed after wall completion) is not approved for WSDOT use. Exceptions and additional requirements regarding the approved details are as follows:

- Several plan sheets that detail panels with larger dimensions than the 5 feet × 5 feet panel. While it is feasible to use larger panels, only the 5 feet × 5 feet panel series is specifically preapproved for use in WSDOT projects. Other panel sizes 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., walls with larger facing panels shall not be submitted as shop drawings in design-bid-build projects).
- Several of the details shown provide only metric dimensions. The closest English system dimensions shall be used, unless the project is a metric project.

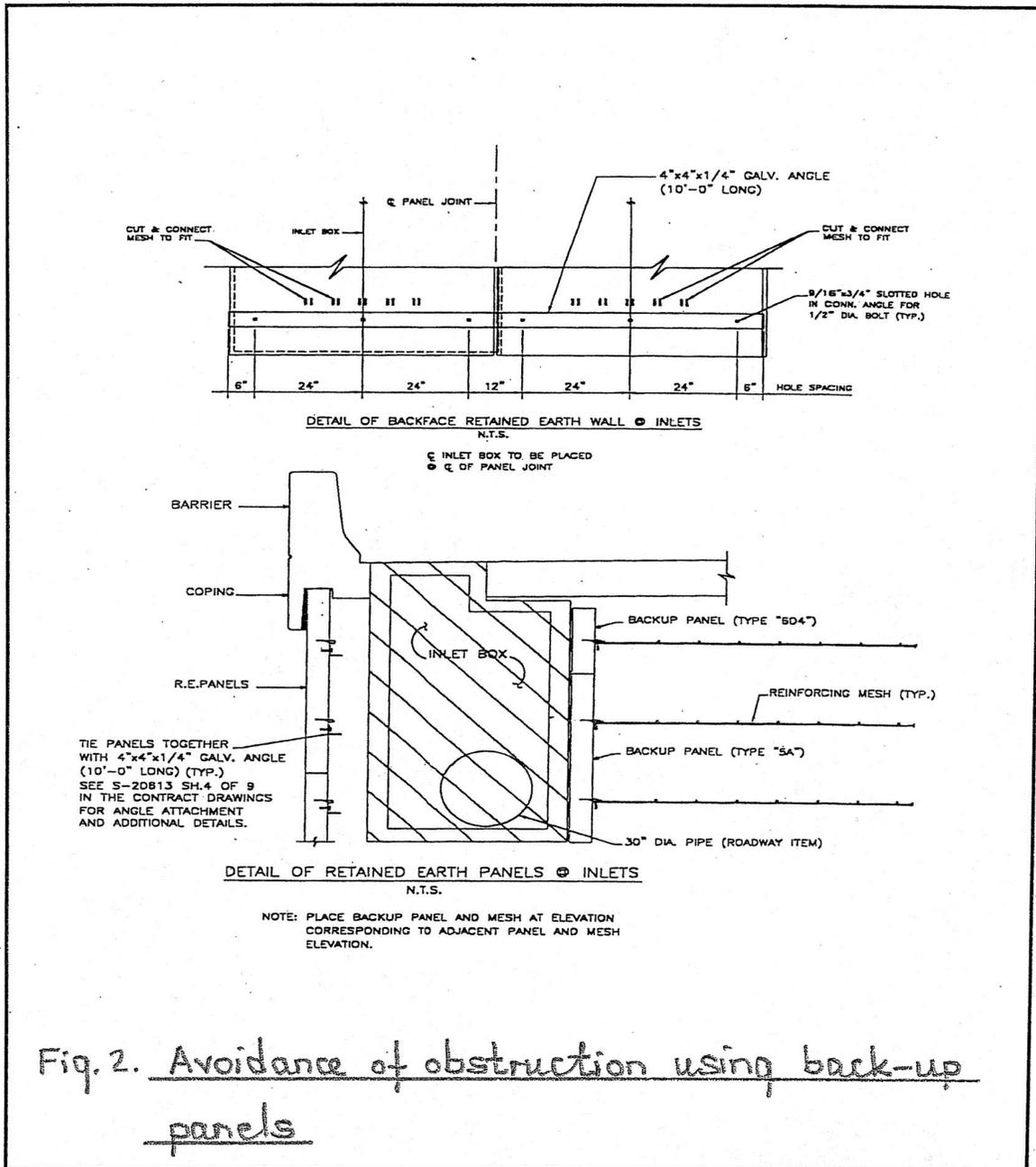
- In the plan sheet on page 7, regarding the filter fabric shown, WSDOT reserves the right to require the use *Standard Specification* materials as specified in *Standard Specification* Section 9-33 that are similar to those specified in this plan sheet.
- In the plan sheets on pages 2 and 6, there should be a minimum cover of 4 inches of soil between the steel grid in the soil and the traffic barrier reaction slab.
- The obstruction avoidance details are preapproved up to a diameter of 5 feet. Larger diameter obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers. However, no details were provided for protrusion of culverts and other objects or conduits through the wall face. Therefore, this wall system is not preapproved for protrusion of culverts and other objects or conduits through the wall face.



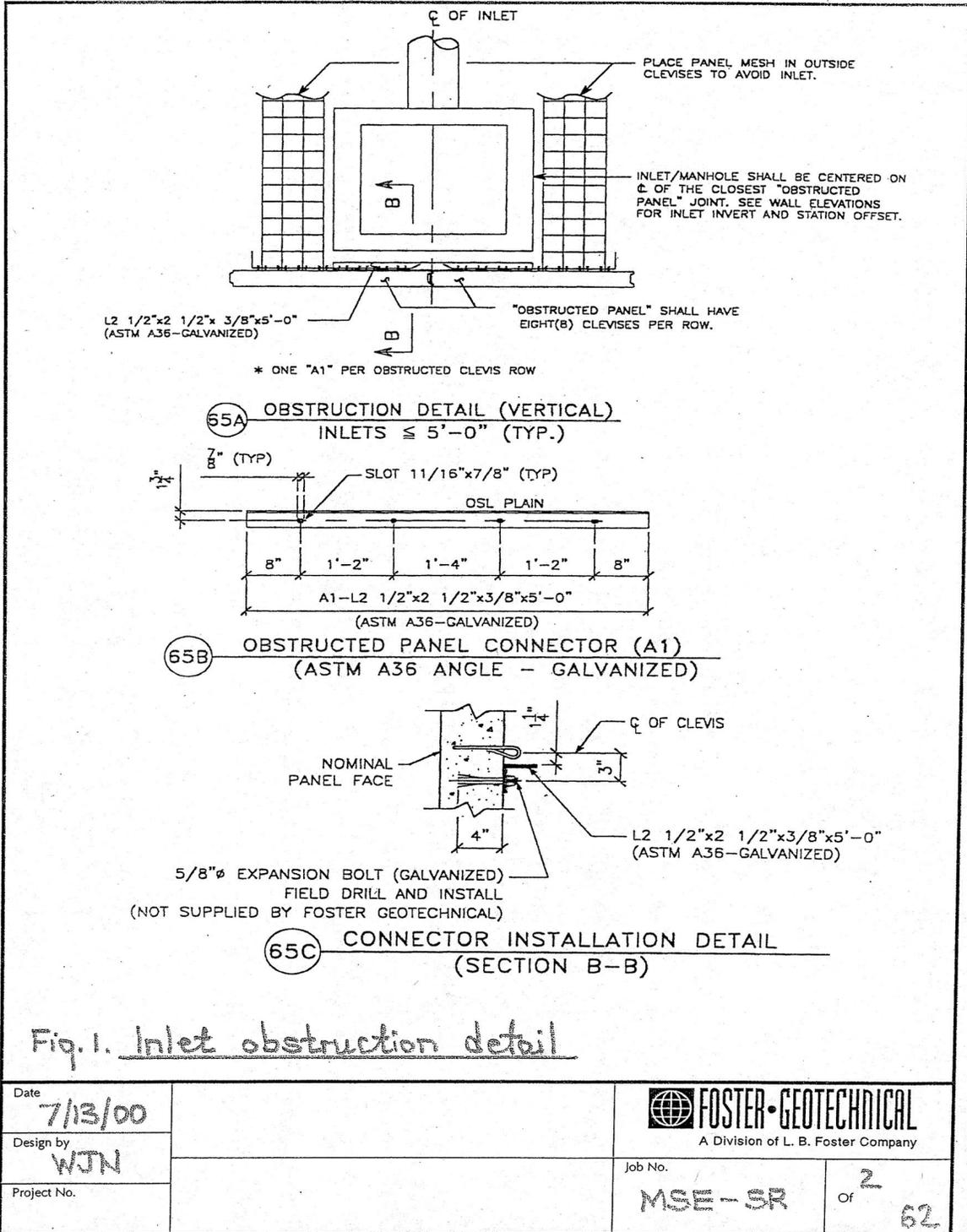
MSE RETAINING WALL: TYPICAL SECTION  
SCALE: N.T.S.

DCS.	DRN.	CHK.	NO.	DATE	BY
					CHK
			RETAINED EARTH™		
A Division of L. B. Foster Company 1860 Hotel Circle North, Suite 304, San Diego, CA 92108 Telephone: (619) 688-2100 / Fax: (619) 688-2499 SAN DIEGO, CA / WOODBRIDGE, VA / HILLSBORO, TX / MELBOURNE, FL / CHICAGO, IL					
Foster Geotechnical design of retaining walls, bridge abutments, and other structures is based on the use of the best available information. The user of such information is responsible for its application. The user of such information is responsible for its application. The user of such information is responsible for its application.					
RETAINED EARTH™ WALLS MSE WALL TYPICAL SECTIONS					
SCALE: AS SHOWN					
JOB NO:					





Date 7/13/00		Job No. MSE-SR	Of 4 52
Design by WJN			
Project No.			







**TYPE-4A**  
SHOW FROM BACK FACE

Q1	Item	Description
1	Welded Wire Fabric - 19 x 192 OAK (W/192) - M(192)	
2	Loop Embeds	1 Top Bar
3	Utility Inset	1 Top Bar

Note: 61 Similar with 8 Loop Embeds Per Row.

**TYPE-4B**  
SHOW FROM BACK FACE

Q1	Item	Description
1	Welded Wire Fabric - 19 x 192 OAK (W/192) - M(192)	
2	Utility Inset	1 Top Bar

Note: 61 Similar with 8 Loop Embeds Per Row.

**TYPE-4C**  
SHOW FROM BACK FACE

Q1	Item	Description
1	Welded Wire Fabric - 19 x 192 OAK (W/192) - M(192)	
2	Utility Inset	1 Top Bar

Note: 61 Similar with 8 Loop Embeds Per Row.

**TYPE-4D**  
SHOW FROM BACK FACE

Q1	Item	Description
1	Welded Wire Fabric - 19 x 192 OAK (W/192) - M(192)	
2	Loop Embeds	1 Top Bar
3	Utility Inset	1 Top Bar

Note: 61 Similar with 8 Loop Embeds Per Row.

**TYPE-4E**  
SHOW FROM BACK FACE

Q1	Item	Description
1	Welded Wire Fabric - 19 x 192 OAK (W/192) - M(192)	
2	Loop Embeds	1 Top Bar
3	Utility Inset	1 Top Bar

Note: 61 Similar with 8 Loop Embeds Per Row.

**TYPE-4F**  
SHOW FROM BACK FACE

Q1	Item	Description
1	Welded Wire Fabric - 19 x 192 OAK (W/192) - M(192)	
2	Loop Embeds	1 Top Bar
3	Utility Inset	1 Top Bar

Note: 61 Similar with 8 Loop Embeds Per Row.

**TYPE-4G**  
SHOW FROM BACK FACE

Q1	Item	Description
1	Welded Wire Fabric - 19 x 192 OAK (W/192) - M(192)	
2	Loop Embeds	1 Top Bar
3	Utility Inset	1 Top Bar

Note: 61 Similar with 8 Loop Embeds Per Row.

**TYPE-4H**  
SHOW FROM BACK FACE

Q1	Item	Description
1	Welded Wire Fabric - 19 x 192 OAK (W/192) - M(192)	
2	Loop Embeds	1 Top Bar
3	Utility Inset	1 Top Bar

Note: 61 Similar with 8 Loop Embeds Per Row.

**TYPE-4I**  
SHOW FROM BACK FACE

Q1	Item	Description
1	Welded Wire Fabric - 19 x 192 OAK (W/192) - M(192)	
2	Loop Embeds	1 Top Bar
3	Utility Inset	1 Top Bar

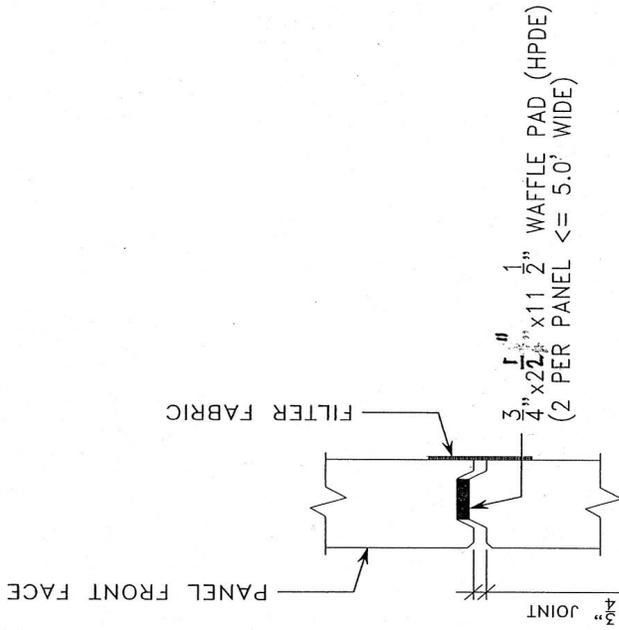
Note: 61 Similar with 8 Loop Embeds Per Row.

**TYPE-4J**  
SHOW FROM BACK FACE

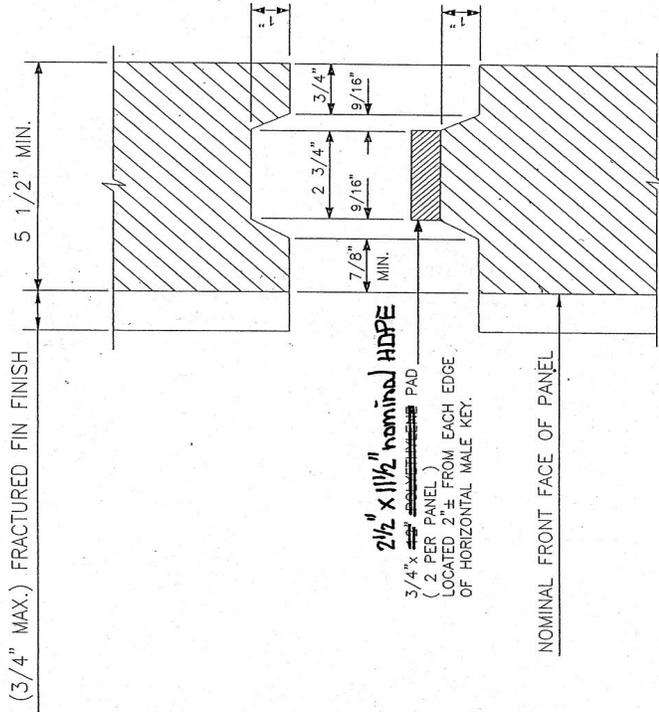
Q1	Item	Description
1	Welded Wire Fabric - 19 x 192 OAK (W/192) - M(192)	
2	Loop Embeds	1 Top Bar
3	Utility Inset	1 Top Bar

Note: 61 Similar with 8 Loop Embeds Per Row.

<b>RETAINED EARTH™ WALLS STANDARD PANEL DETAILS</b>	Foster Geotechnical is a registered professional engineer and geotechnical engineer. The designs shown herein are the property of Foster Geotechnical, Inc. and are not to be used for any other project without the written consent of Foster Geotechnical, Inc.	<p><b>Foster Geotechnical</b> A Division of L. B. Foster Company 1650 Hotel Circle North, Suite 304, San Diego, CA, 92108 Telephone: (619) 484-4400 / Fax: (619) 484-2489 SAN DIEGO, CA / WOODBRIDGE, VA / HILLSBORO, TX / MELBOURNE, FL / CHICAGO, IL</p>	DES. _____ DRN. _____ CHK. _____ RETAINED EARTH™	NO. _____ DATE _____ REVISION _____ BY: JHK
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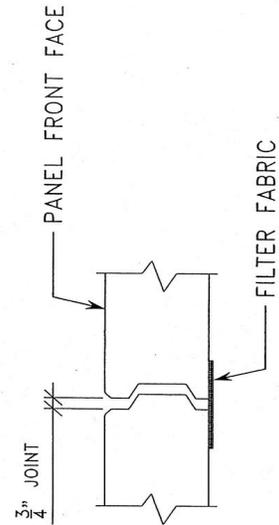


WAFFLE PAD AT HORIZONTAL JOINT



PANEL JOINT DETAIL

N.T.S.



HORIZONTAL & VERTICAL JOINT DETAIL

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.

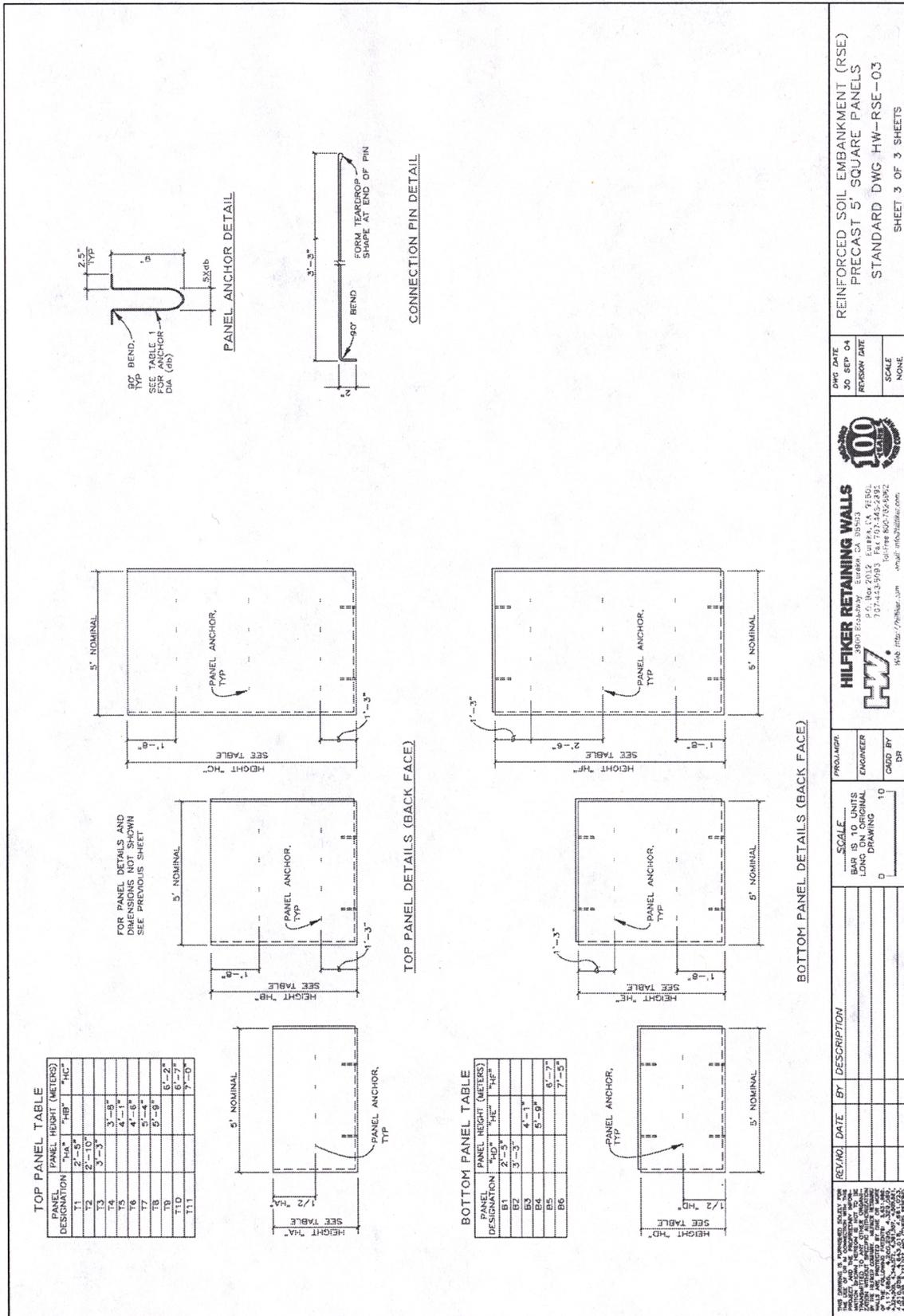
Reinforcement pullout shall be calculated based on the default values for steel grid reinforcement provided in the AASHTO Specifications. If, at some future time product and soil specific pullout data is provided to support use of non-default pullout interaction coefficients, it should be noted that 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.







REINFORCED SOIL EMBANKMENT (RSE)  
 PRECAST 5' SQUARE PANELS  
 STANDARD DWG HW-RSE-03  
 SHEET 3 OF 3 SHEETS

**HILFIKER RETAINING WALLS**  
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 ENGINEER: \_\_\_\_\_  
 CADD BY: \_\_\_\_\_  
 DR: \_\_\_\_\_

SCALE: \_\_\_\_\_  
 BAC TO UNITS: \_\_\_\_\_  
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 DRAWING: \_\_\_\_\_

REV/NO	DATE	BY	DESCRIPTION

THE USER OF THIS DOCUMENT SHALL BE RESPONSIBLE FOR OBTAINING ALL NECESSARY PERMITS AND APPROVALS FROM THE APPROPRIATE AGENCIES. THE USER SHALL BE RESPONSIBLE FOR OBTAINING ALL NECESSARY PERMITS AND APPROVALS FROM THE APPROPRIATE AGENCIES. THE USER SHALL BE RESPONSIBLE FOR OBTAINING ALL NECESSARY PERMITS AND APPROVALS FROM THE APPROPRIATE AGENCIES.

# **Preapproved Wall Appendix: Specific Requirements and Details for Hilfiker Welded Wire Faced Walls**

## **Appendix 15-H**

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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 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.

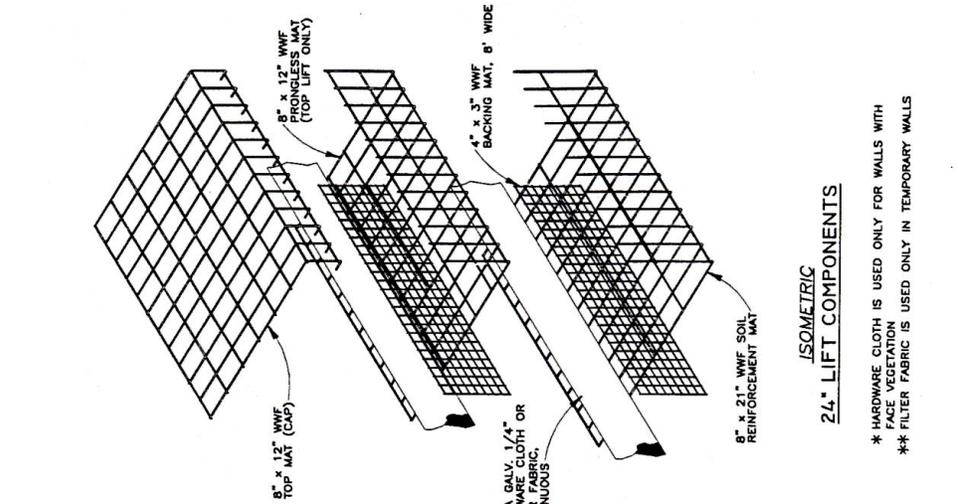
Regarding the backing mats used in the welded wire facing, the minimum clear opening dimension of the backing mat shall not exceed the minimum particle size of the wall facing backfill. The maximum particle size for the wall facing backfill shall be 6 inches.

The maximum vertical spacing of soil reinforcement shall be 24 inches. For wall heights greater than 20 feet, for the portion of the wall more than 20 feet below the wall top at the face, the maximum vertical spacing of reinforcement shall be 18 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.

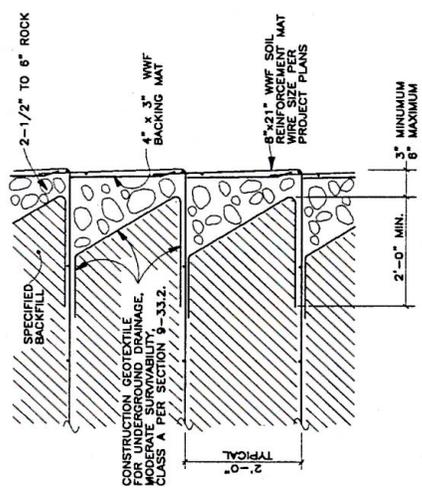
This wall system is preapproved for a welded wire/gravel fill face for vertical to near vertical facing batter and welded wire vegetated face for wall face batters as steep as 6V:1H. This preapproval presumes that the facing tolerances in the WSDOT *Standard Specifications* Section 6-13.3(1) for welded wire faced walls are met.

The following standard details shall be used for the Hilfiker Welded Wire Faced Wall system:

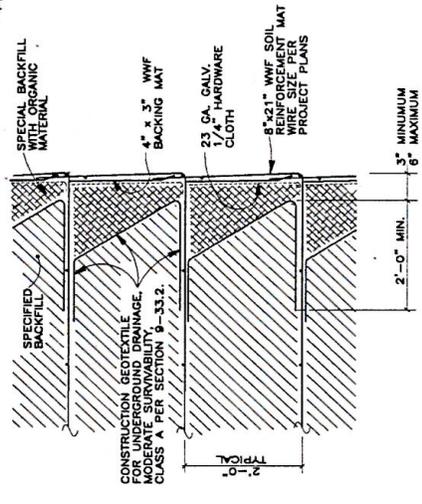


ISOMETRIC  
24" LIFT COMPONENTS

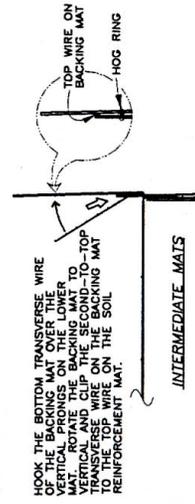
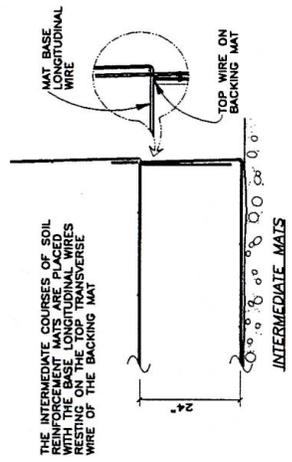
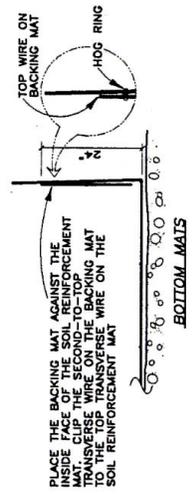
\* HARDWARE CLOTH IS USED ONLY FOR WALLS WITH FACE VEGETATION  
\*\* FILTER FABRIC IS USED ONLY IN TEMPORARY WALLS



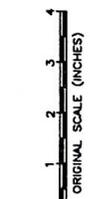
SECTION  
WALL FACE DETAIL  
WITHOUT VEGETATION



SECTION  
WALL FACE DETAIL  
WITH VEGETATION



SECTIONS  
24" MAT CONNECTION DETAILS



DATE: \_\_\_\_\_  
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DESIGNED BY: \_\_\_\_\_  
CHECKED BY: \_\_\_\_\_  
DATE: \_\_\_\_\_

SHEET NO. 10 OF 5 PAGE 15-H-2	WASHINGTON STATE Department of Transportation 	HILFIKER 24" LIFT WIRE FACED MSE DETAILS
HILFIKER RETAINING WALLS HILFIKER 24" LIFT WIRE FACED MSE HILFIKER 24" LIFT WIRE FACED MSE		









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 Keystone for the KeySystem I wall are summarized in the HITEC evaluation report for this wall system (HITEC, 2000, *Evaluation of the KeySystem™ I Retaining Wall*, ASCE, CERF Report 40478). The design methodology, which is based on the Standard Specifications for Highway Bridges (2002) conflicts with the general design requirements in Appendix 15-A regarding the K value for internal stability (for the Simplified Method, Keystone recommends K of 2.0 at the top of the wall rather than 2.5), and the allowable stress for design of the steel grid reinforcement strips (Keystone recommends an allowable stress of  $0.55F_y$  rather than  $0.48F_y$  for design of the steel grid strip reinforcement). WSDOT does not concur with the reduced K value of 2.0. Therefore, the K value at the wall top should be 2.5 to be consistent with the AASHTO design specifications. WSDOT does concur with the use of an allowable stress of  $0.55F_y$ . 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 LRFD Specifications is 2 feet. Regarding horizontal spacing of steel grid reinforcement strips, reinforcement shall be located at a maximum spacing of every other block, as allowed by the AASHTO LRFD Specifications.

Reinforcement pullout shall be calculated based on the default values for steel grid reinforcement provided in the AASHTO LRFD Specifications. If, at some future time product and soil specific pullout data is provided to support use of non-default pullout interaction coefficients (data is provided in the HITEC report for this wall system, but different interaction coefficients were not specifically proposed), it should be noted that 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.

Concrete for dry cast concrete blocks used in the KeySystem I 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 C1262:
  - 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 the HITEC report for this wall system that Keystone allows a dimensional tolerance for the height of the block of 1/8 inch, which is consistent with ASTM C1372, 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.

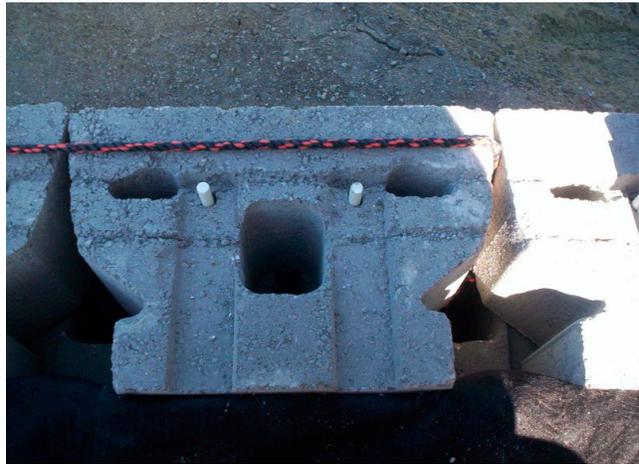
Block connector pins shall conform to AASHTO M 32, and shall be galvanized after fabrication in accordance with AASHTO M 111.

The steel grid ladder strips shall be transported to and handled at the project site in a manner that minimizes bending of the steel. As shipped to the wall site, the steel strips must still meet the tolerance requirements of ASTM A185 (i.e., the permissible variation of the center-to-center distance between longitudinal wires shall not exceed  $\pm 0.5$  inch of the specified distance).

Approved details for the KeySystem I wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

1. Immediately behind the facing blocks, either a strip of Construction Geotextile for Underground Drainage, Moderate Survivability, Class A per *Standard Specifications* Section 9-33 shall be placed vertically against the blocks, with 1 foot horizontal tails placed at each reinforcement level (i.e., the geotextiles strip forms a sideways "U") shall be used (see Figures 15-I-2 and 15-I-3), or a 1 foot wide column of crushed rock shall be placed as shown in Plan Sheets 3 and 5. In both cases, the purpose is to prevent movement of fines in the backfill from washing through the wall facing.
2. Any field bending of the welded wire strip reinforcement required to accommodate obstructions as shown in the attached plan sheets shall be done in accordance with *Standard Specifications* Section 6-02.3(24)A "Field Bending". Any damage to the galvanizing resulting from the bending shall be repaired such that the galvanizing layer effectiveness for resisting corrosion is restored to its original condition.

3. Any adjustments to the facing batter needed during erection of the wall shall be done in a manner that prevents adding additional stress to the reinforcement-facing connection and that also prevents significant stress concentrations between the facing blocks that could cause cracking of the facing blocks as additional blocks are placed. The use of rope as shown in Figure 15-(KeySystem I)-1 below is not acceptable as a method to adjust facing batter. In general, any shims used between blocks to adjust facing batter shall be no more than 0.125 inch thick, shall minimize the creation of local stress concentrations, and shall be made of a material that is durable and not degrade over the life of the wall.
4. 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.



**Keystone KeySystem I Block With Fiberglass Alignment Pins**  
*Figure 15-I-1*



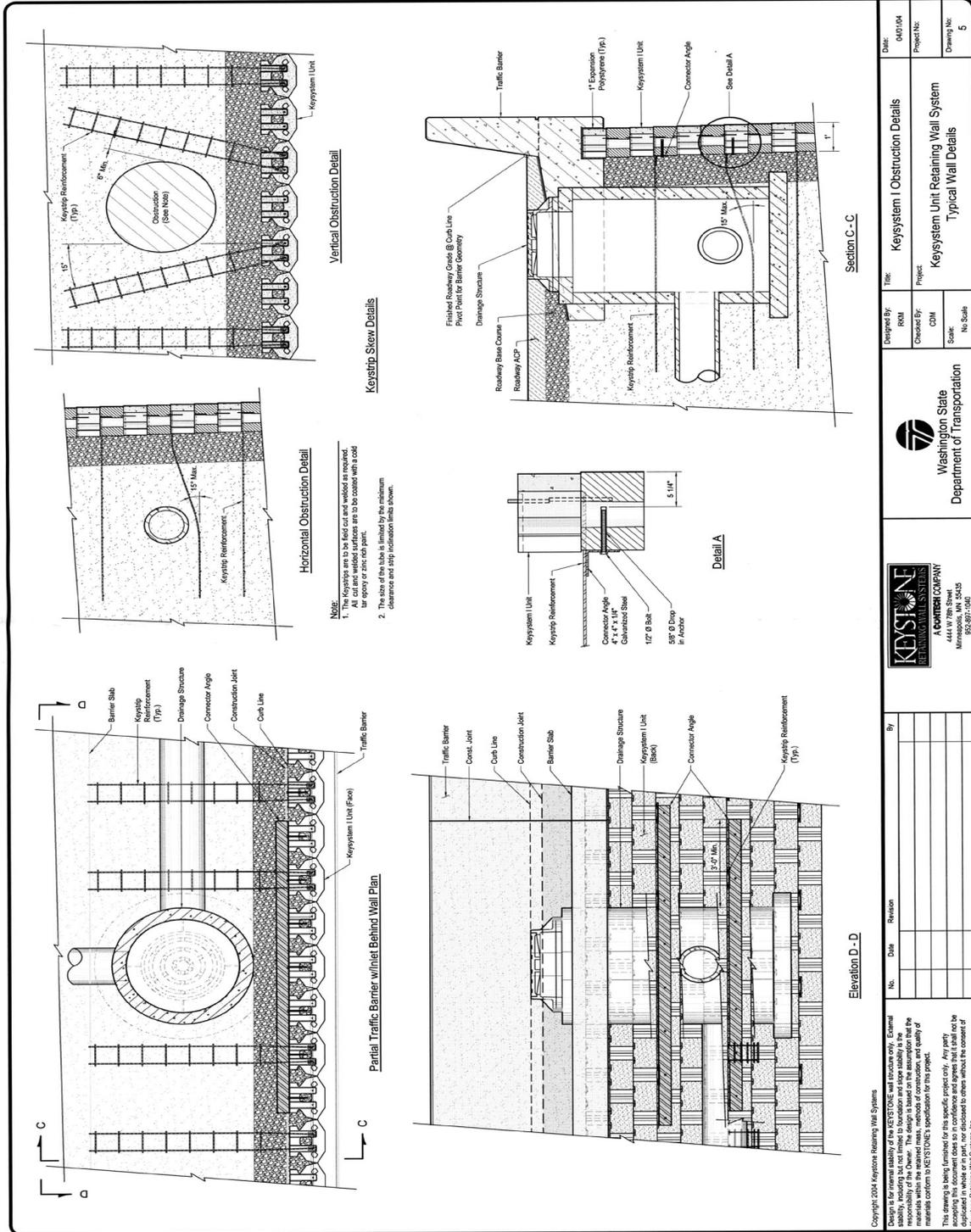
**KeySystem I wall Keysteel Reinforcement Connector  
and Block as Assembled, Showing Both Fiberglass Alignment Pins  
and Galvanized Steel Connector Pins**  
*Figure 15-I-2*



**KeySystem I Wall Keystone Reinforcement Connector and Block as Assemble, with Block Placed on Top**  
*Figure 15-I-3*

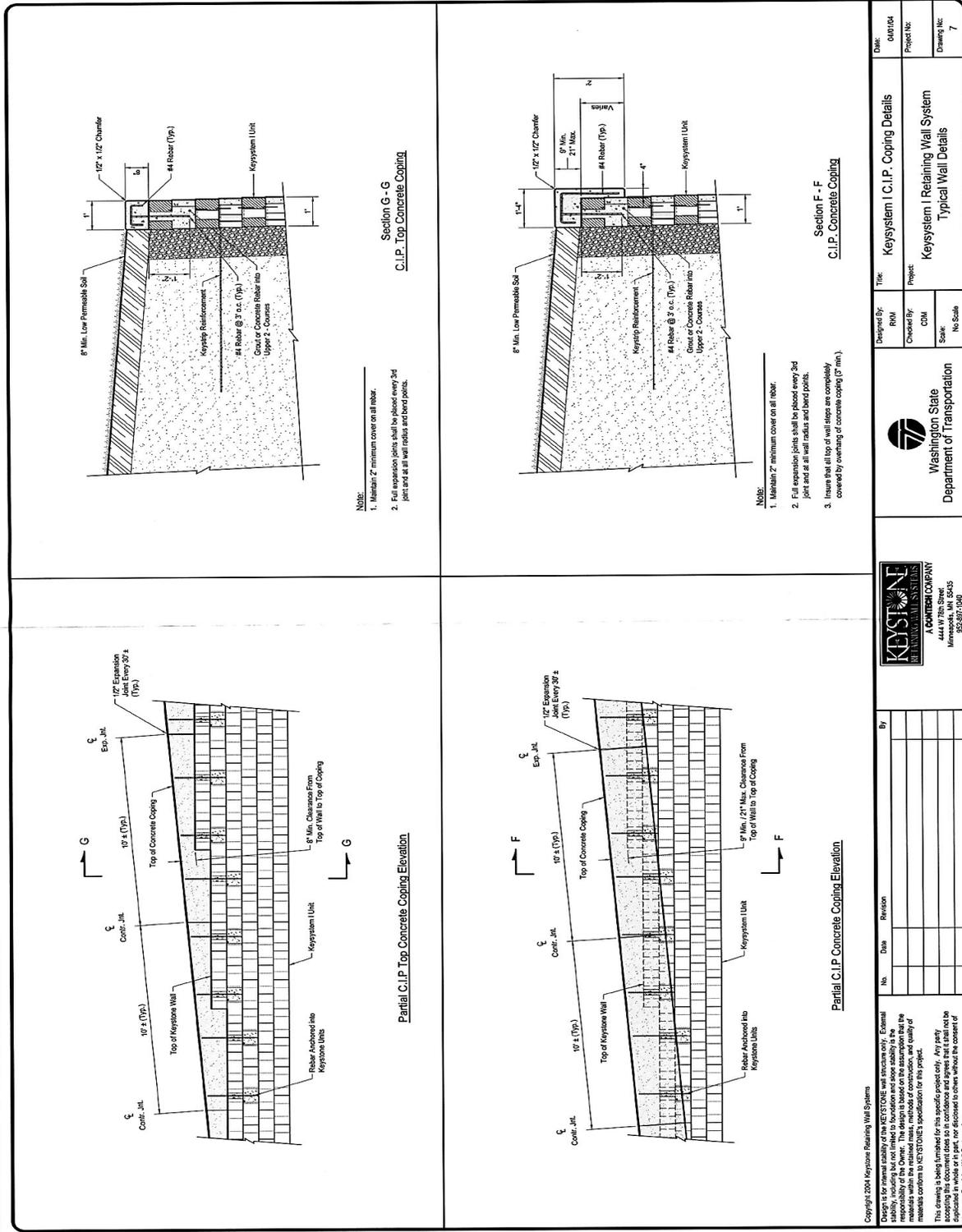






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<p>Designed By: ROOM</p> <p>Checked By: CDM</p> <p>Scale: _____</p> <p>No. Scale: _____</p>	<p>Title: KeySystem I Obstruction Details</p> <p>Project: KeySystem Unit Retaining Wall System Typical Wall Details</p>	<p>Date: 04/01/04</p> <p>Project No: _____</p> <p>Drawing No: 5</p>





Designed By: RMA Checked By: CDM Scale: No Scale		Title: KeySystem I C.I.P. Coping Details Project: KeySystem I Retaining Wall System Typical Wall Details	Date: 04/04/04 Project No: Drawing No: 7
Washington State Department of Transportation		KEYSTONE RETAINING WALL SYSTEMS A CAMBERT COMPANY 444 W. 7th Street Minneapolis, MN 55405 952.877.7040	
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No.	Date	Revision	By



# **Preapproved Wall Appendix: Specific Requirements and Details for Tensar MESA Walls**

## **Appendix 15-J**

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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<sup>3</sup> system and the HP System. Both systems provide partial connection coverage, with the DOT<sup>3</sup> system only providing 14 teeth per 21 openings, and the HP System providing 17 teeth per 21 openings. The DOT<sup>3</sup> 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.

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 $T_{ult}$ (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  $T_{ultconn}$  value as in HITEC report.

### Approved Connection Strength Design Values for Tensar MESA Walls

Table 15-J-1

$T_{ac}$ , the long-term connection strength, shall be calculated as follows:

$$T_{ac} = \frac{T_{MARV} \cdot CR_u}{RF_{CR} \cdot RF_D} \quad (15-J-1)$$

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<sup>3</sup> 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 1/8 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.



**MESA DOT System Connector and Block**  
*Figure 15-J-1*



**MESA DOT System Connector and Block as Assembled**  
*Figure 15-J-2*



**MESA DOT System Connector and Block as Assembled, With Block Placed on Top**  
**Figure 15-J-3**

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.

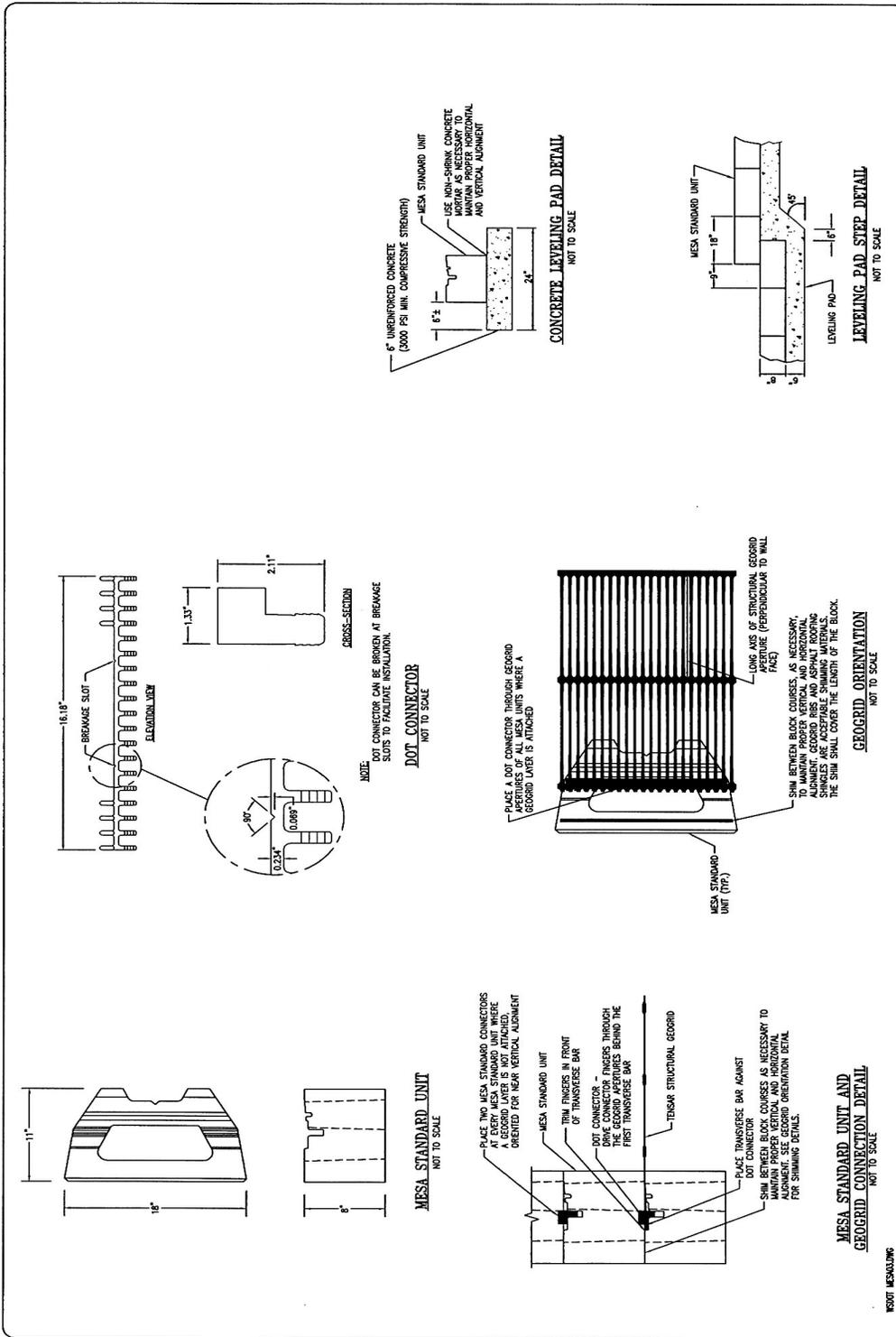
Block connectors for block courses without geogrid reinforcement shall be glass fiber reinforced high-density polyethylene (HDPE) conforming to the following minimum material specifications:

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 be 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.





**STANDARD MESA DETAILS & CONSTRUCTION NOTES**

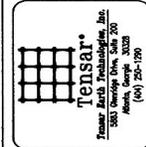
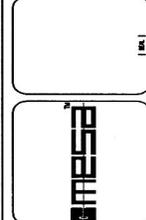
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**TYPICAL DETAILS**

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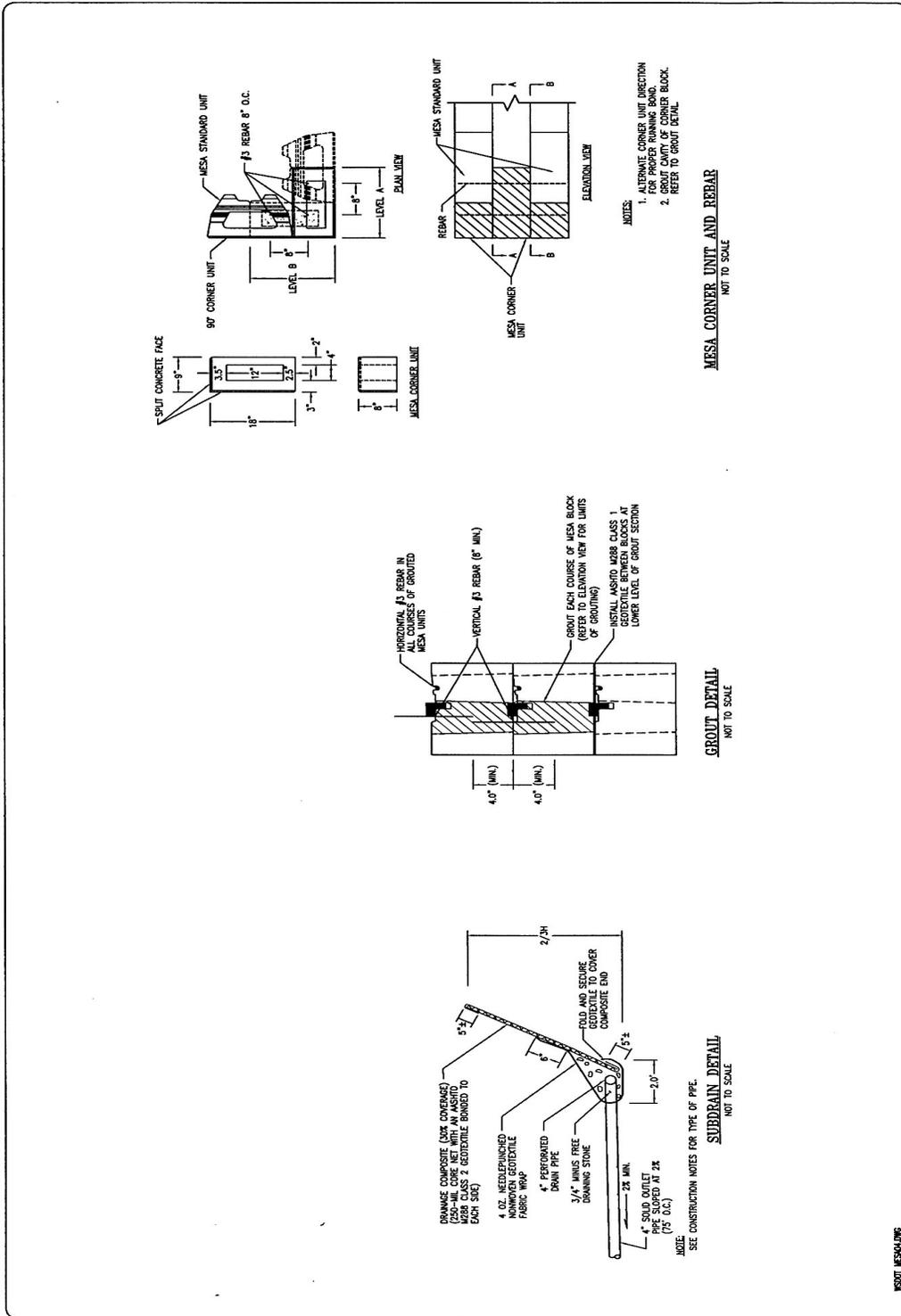
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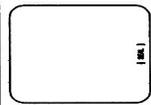


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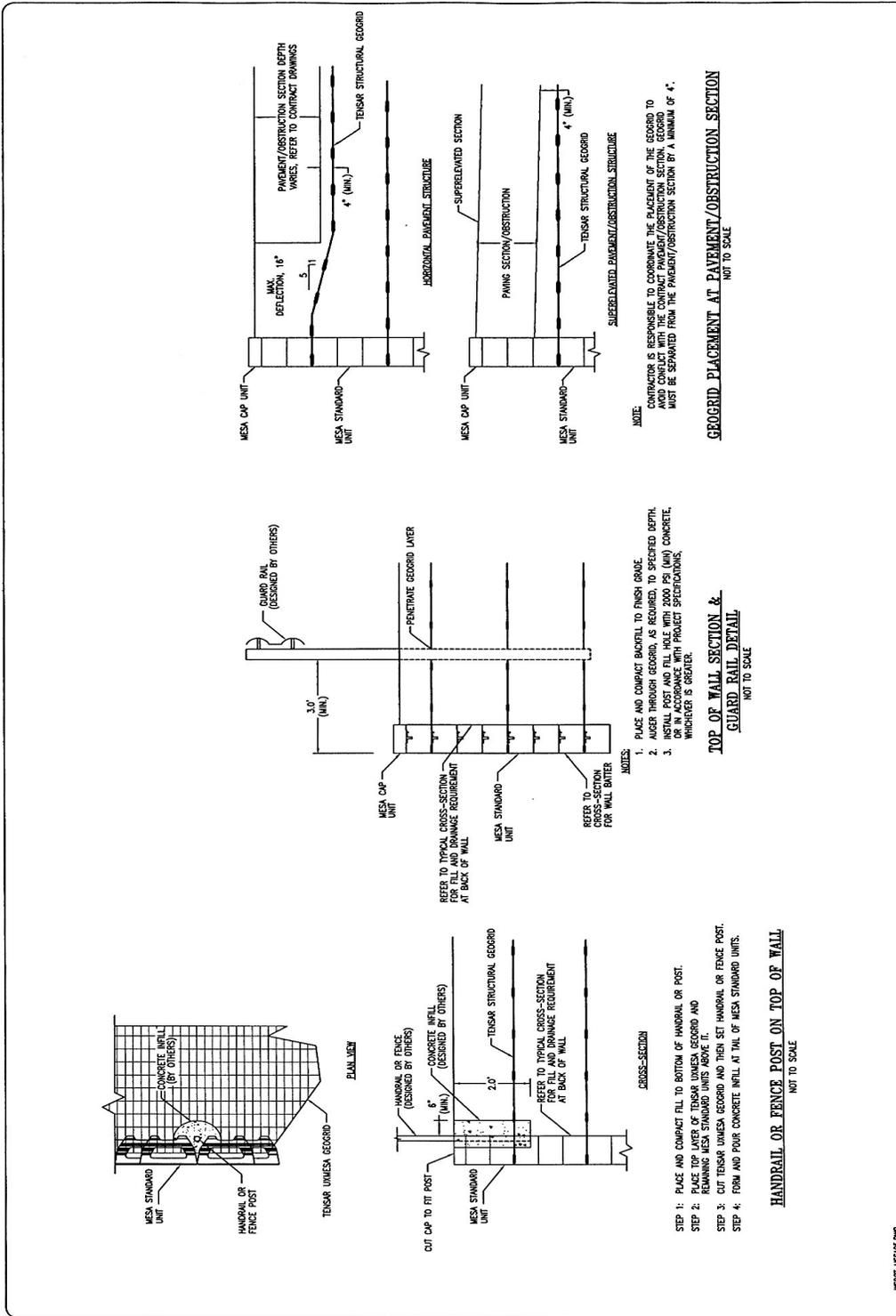
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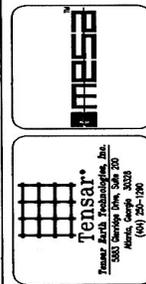


STANDARD MESA DETAILS & CONSTRUCTION NOTES

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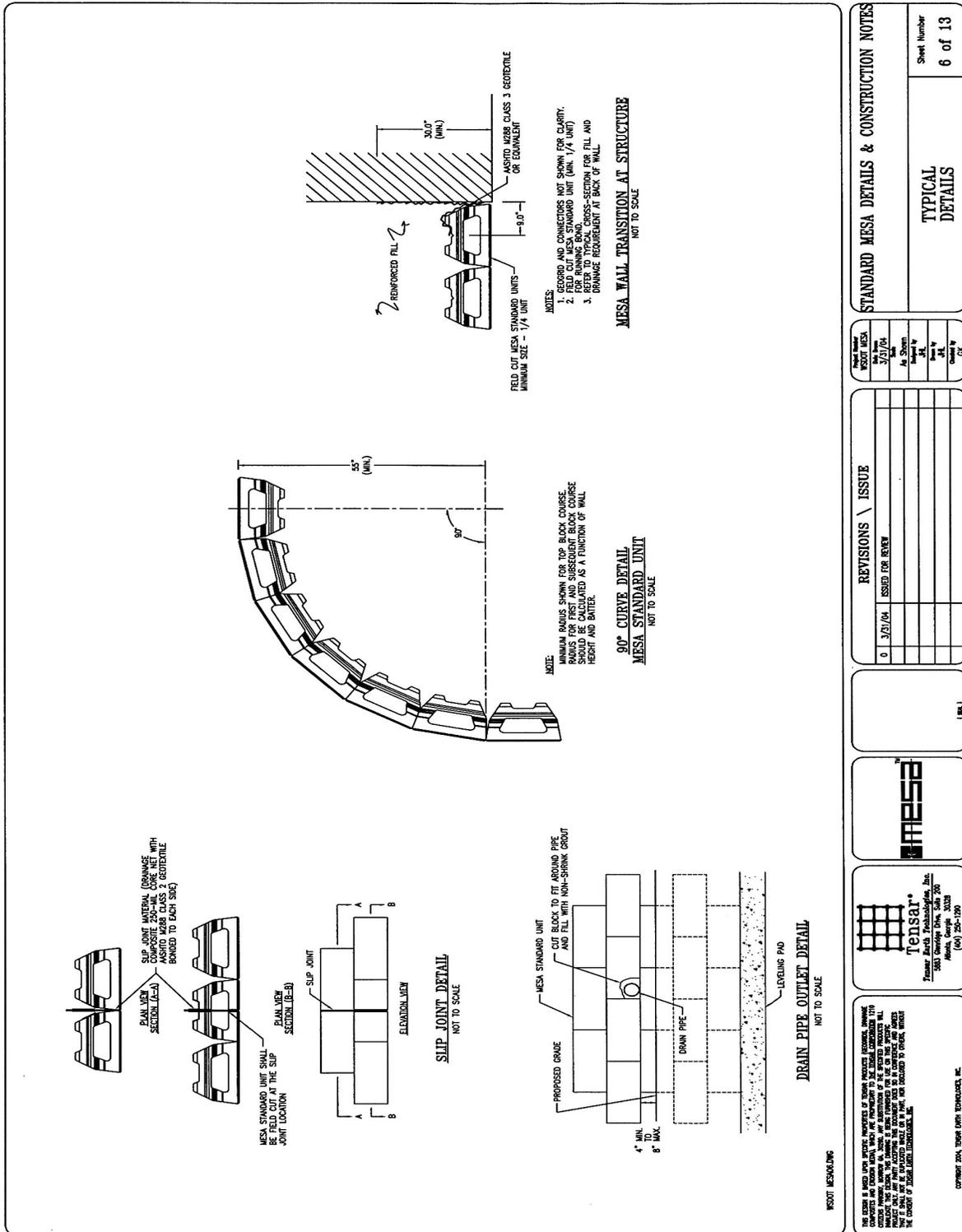
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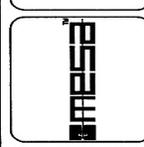
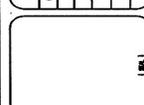
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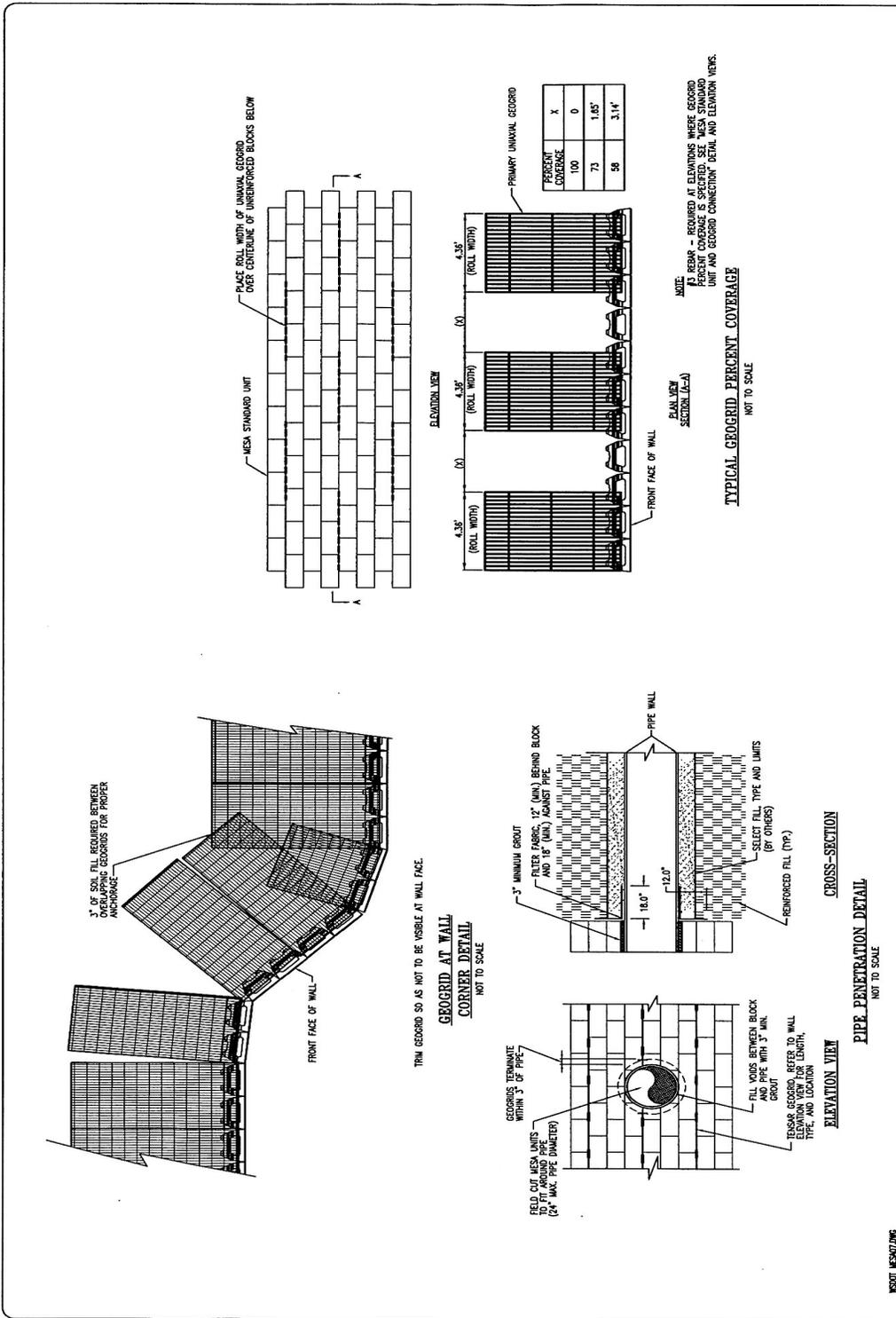


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 (704) 252-1700

**MESA**

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**TYPICAL DETAILS**

Sheet Number  
 7 of 13

**FABRIC DRAINAGE SPECIFICATION**

Geotextile may be used in lieu of aggregate for the drainage medium behind the Meso facing units if the select fill used in the reinforced zone, as a minimum, complies with the ASHTO Guidelines for Mechanically Stabilized Earth Retaining Walls, Section 5.8, Mechanically Stabilized Earth (MSE) Wall Design of the 2002 ASHTO guidelines defines the soil's physical properties.

An eight (8) oz per square yard nonwoven needle punched geotextile, ASHTO 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.

**STAGES OF GEOTEXTILE LAYOUT**  
MESA WALL  
GEOTEXTILE SEPARATOR  
NOT TO SCALE

**GEOTEXTILE ON TOP MESA UNIT**  
DETAIL S2

**GEOTEXTILE ON SECOND ROW MESA UNIT**  
DETAIL S1

**TYPICAL SECTION**  
NOT TO SCALE

**CAP ADHESIVE**  
DETAIL S3

**MESA CAP**  
DETAIL S4

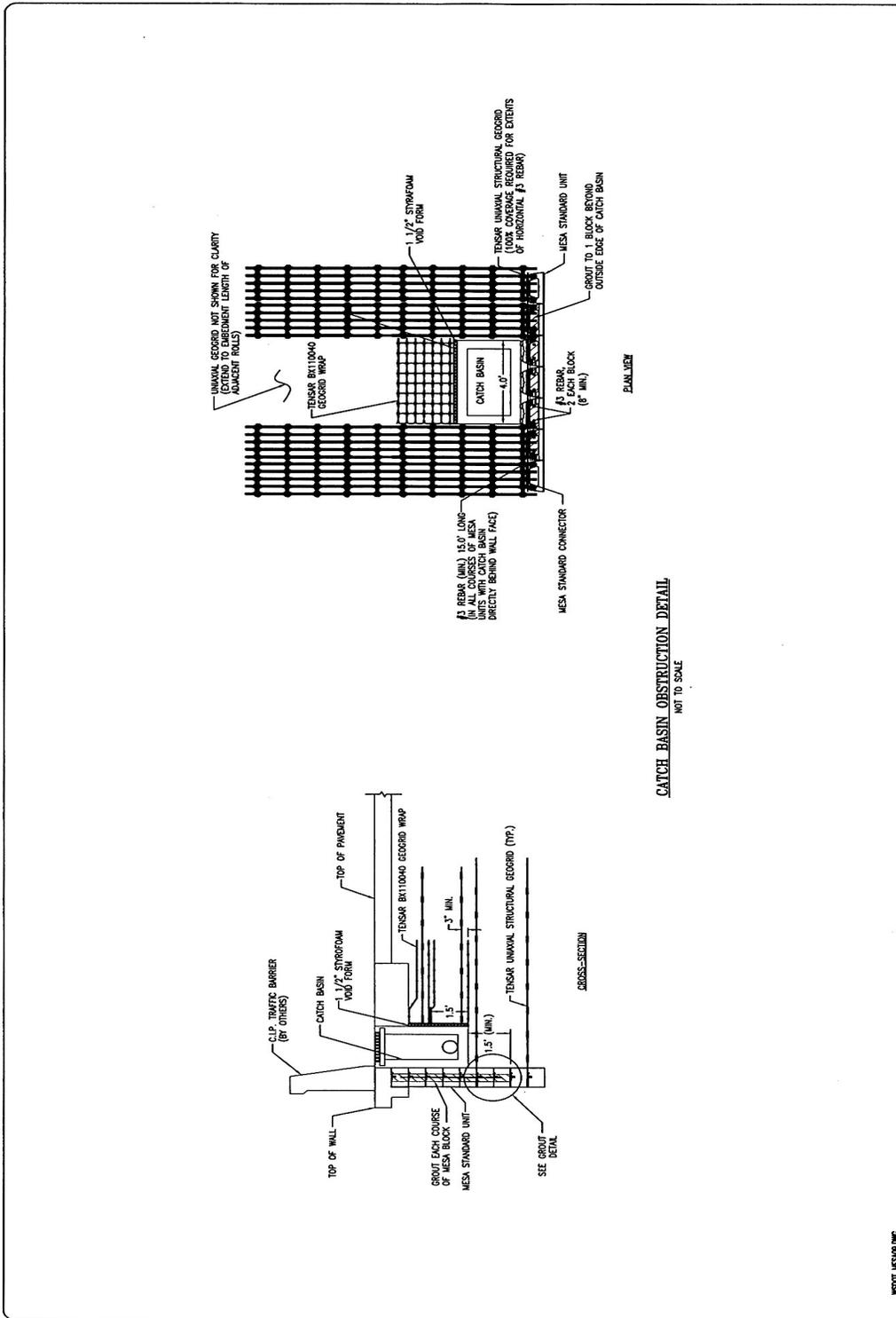
**REVISIONS / ISSUE**

NO.	DATE	ISSUED FOR REVIEW	REVISIONS / ISSUE
0	3/21/04		

**STANDARD MESA DETAILS & CONSTRUCTION NOTES**

**FABRIC SEPARATOR DETAIL**

Sheet Number  
**8 of 13**



CATCH BASIN OBSTRUCTION DETAIL

NOT TO SCALE

STANDARD MESA DETAILS & CONSTRUCTION NOTES

Project Name	Sheet Number
DESIGN MESA	9 of 13
Date	
As Shown	
By	
Checked	
OK	

0	3/21/04	ISSUED FOR REVIEW
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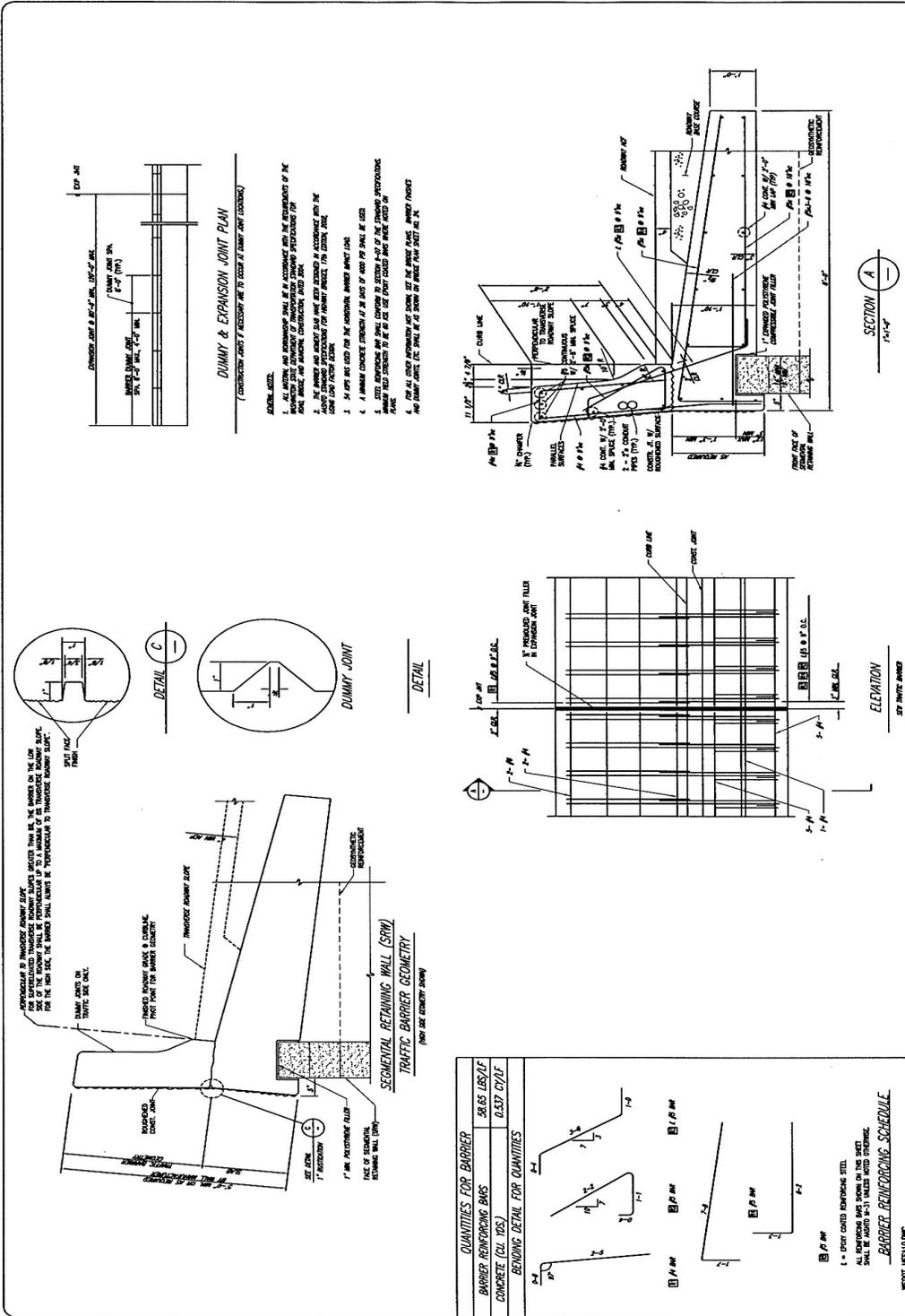
REVISIONS / ISSUE		



**Tensar**  
 Polymer Geotextiles, Geogrids, Geomembranes, and Geocomposites  
 5850 Chesapeake Drive, Suite 200  
 Atlanta, Georgia 30339  
 (404) 250-1200

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**STANDARD MESA DETAILS & CONSTRUCTION NOTES**

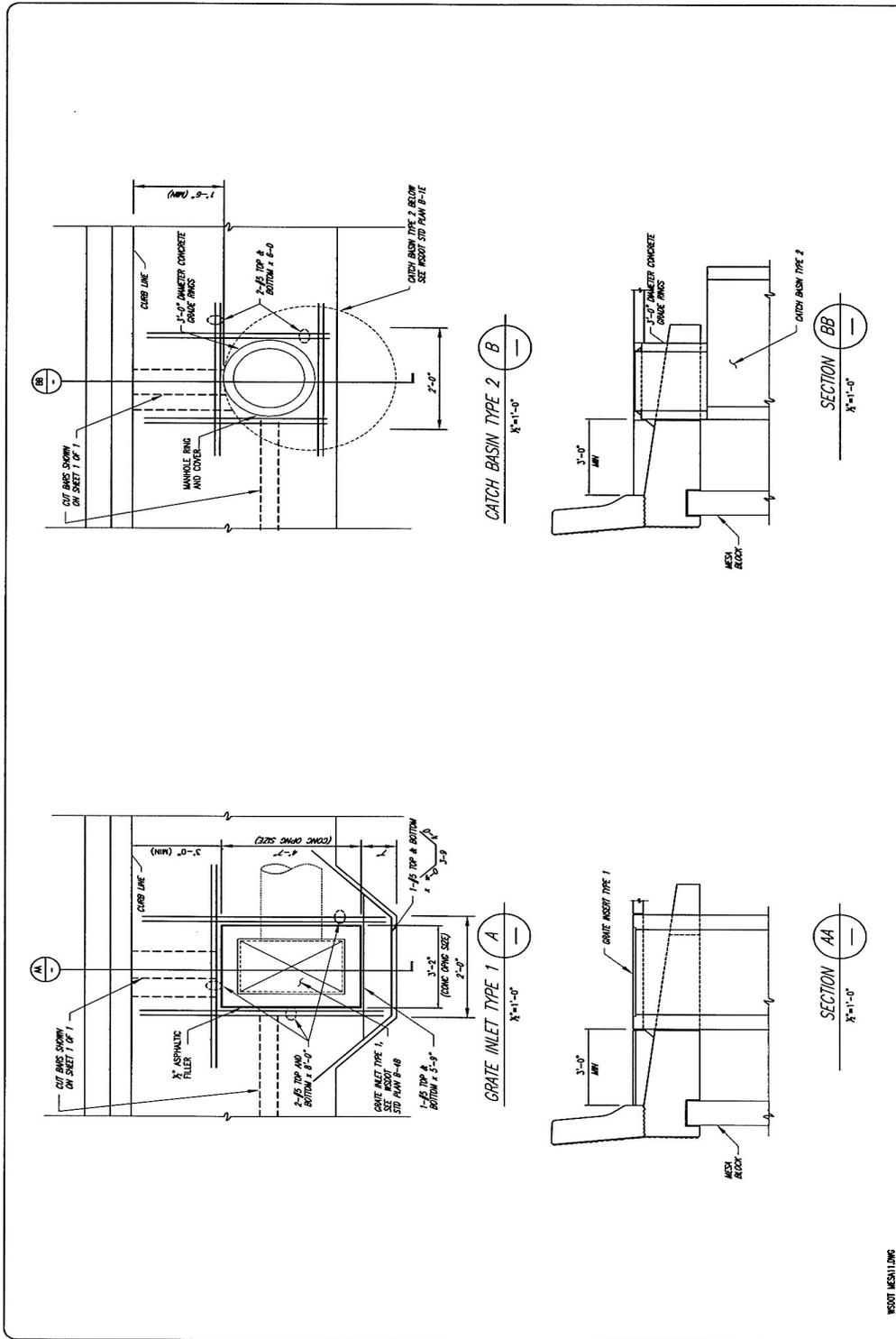
**32 INCH TYPE "F" TRAFFIC BARRIER STANDARD**

Sheet Number: 10 of 13

**Tensar**  
 Tensar Geotechnical, Inc.  
 1580 S. State Street, Suite 200  
 Phoenix, Arizona 85042  
 (602) 954-1200

**MESA**

**COMPANY: TENSAR GEOTECHNICAL, INC.**



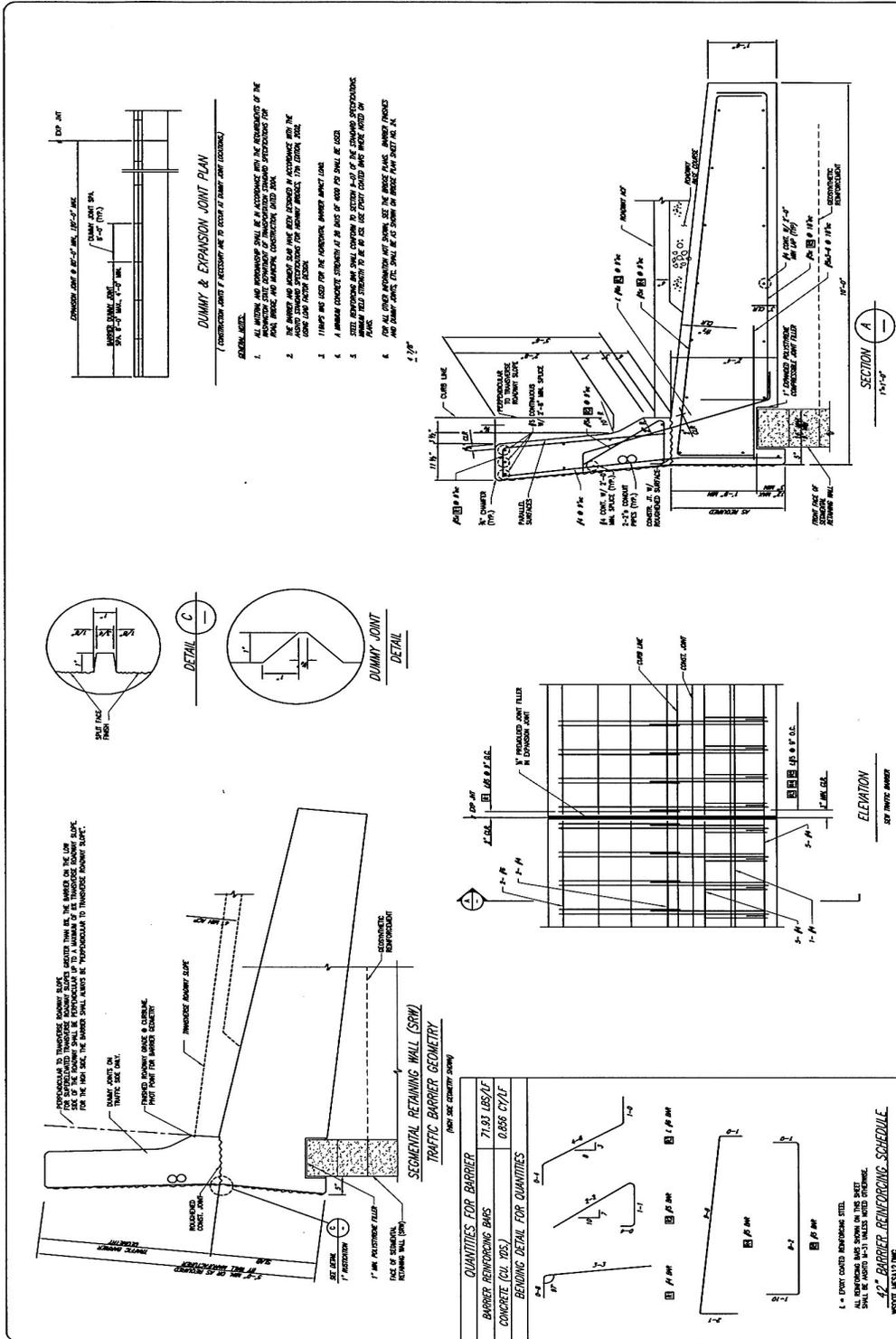
WSDOT MESA11.DWG

<p><b>REVISIONS / ISSUE</b></p> <table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 10%;">0</td> <td style="width: 40%;">3/21/04</td> <td style="width: 50%;">ISSUED FOR REVIEW</td> </tr> <tr> <td> </td> <td> </td> <td> </td> </tr> <tr> <td> </td> <td> </td> <td> </td> </tr> <tr> <td> </td> <td> </td> <td> </td> </tr> </table>	0	3/21/04	ISSUED FOR REVIEW										<p><b>STANDARD MESA DETAILS &amp; CONSTRUCTION NOTES</b></p> <p>32 INCH TYPE "F" TRAFFIC BARRIER STANDARD</p> <p style="text-align: right;">Sheet Number 11 of 13</p>
0	3/21/04	ISSUED FOR REVIEW											

**Tensar**  
Tensar Geotextiles, Inc.  
5850 Cleveland Blvd., Suite 200  
Meriden, Georgia 30259  
(404) 252-1200

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**STANDARD MESA DETAILS & CONSTRUCTION NOTES**

42 INCH TYPE "T" TRAFFIC BARRIER STANDARD

Sheet Number 12 of 13

**REVISIONS / ISSUE**

NO.	DATE	ISSUED FOR REVIEW
0	3/23/04	



**Tensar**  
 Tensar Research, Inc.  
 5500 University Drive, Suite 205  
 Morris, Pennsylvania, 15460  
 (717) 232-1100

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**TRAFFIC BARRIER WALL (SBW)**

QUANTITIES FOR BARRIER

BARRETT REINFORCING BARS	71.83 LBS/LF
CONCRETE (CU YDS)	0.858 CU YD/LF

**BENDING DETAIL FOR QUANTITIES**

**42" BARRIER REINFORCING SCHEDULE**

WSDOT MESA12/03

1" = 1" EXCEPT REINFORCING STEEL. ALL REINFORCING BARS SHOWN ON THIS SHEET SHALL BE NUMBER 4-3, UNLESS NOTED OTHERWISE.

- DUMMY & EXPANSION JOINT PLAN**
- CONSTRUCTION DETAILS AS NECESSARY ARE TO BE SHOWN AT DUMMY JOINT (DETAIL C)
- GENERAL NOTES:**
1. THE JOINT AND EXPANSION JOINT SHALL BE AS SHOWN WITH THE DIMENSIONS OF THE REINFORCING STEEL CONFORMING TO THE DIMENSIONS OF THE STANDARD SPECIFICATIONS FOR REINFORCING STEEL.
  2. THE JOINT AND EXPANSION JOINT SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE DIMENSIONS AND MATERIALS SHOWN ON THIS SHEET. THE JOINT SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE DIMENSIONS AND MATERIALS SHOWN ON THIS SHEET.
  3. THE JOINT AND EXPANSION JOINT SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE DIMENSIONS AND MATERIALS SHOWN ON THIS SHEET.
  4. THE JOINT AND EXPANSION JOINT SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE DIMENSIONS AND MATERIALS SHOWN ON THIS SHEET.
  5. THE JOINT AND EXPANSION JOINT SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE DIMENSIONS AND MATERIALS SHOWN ON THIS SHEET.
  6. THE JOINT AND EXPANSION JOINT SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE DIMENSIONS AND MATERIALS SHOWN ON THIS SHEET.





## ***Preapproved Wall Appendix: Specific Requirements and Details for T-WALL® (The Neel Company)***

### ***Appendix 15-K***

---

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply to the design of the T-WALL®:

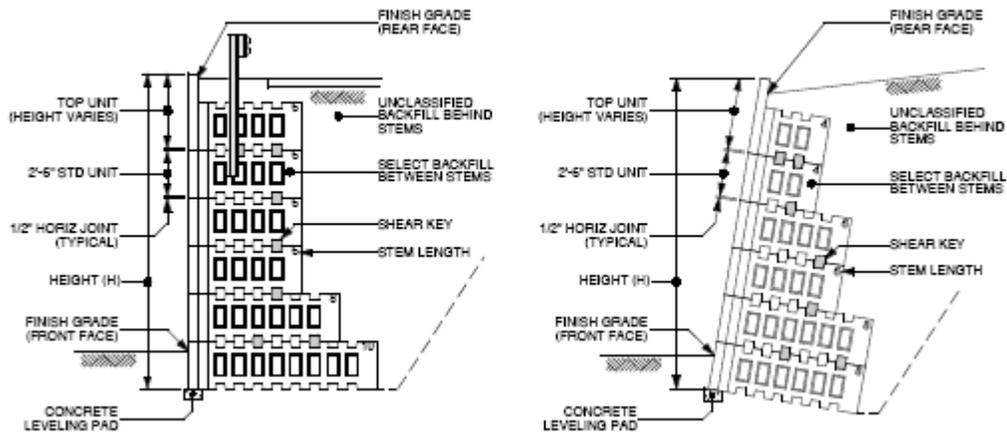
No HITEC evaluation report is currently available for this wall system. The design procedures used for T-WALL® are based on the AASHTO Standard Specifications for Highway Bridges (2002). This wall system is considered to be a hybrid wall, having characteristics of both MSE walls and Modular walls. Interim approval is given for the continued use of the AASHTO Standard Specifications as the basis for design.

The design procedures provided in the AASHTO Standard Specifications, Articles 5.8 and 5.9, are most applicable to this wall system and shall in general be used for design of the T-WALL® system. For internal geotechnical stability, each panel level shall be internally stable against local pullout stresses with a minimum safety factor of 1.5 for static forces and 1.5 for the seismic loading case. Each panel level shall also be stable against overturning (minimum FS of 2.0 for static forces and 1.5 for seismic forces) and sliding (minimum FS of 1.5 for static forces and 1.1 for seismic forces) per the AASHTO Standard Specifications. Only 80 percent of the backfill weight shall be considered effective for resisting lateral earth pressure behind the wall for overturning stability as required in Article 5.9 of the AASHTO Standard Specifications for Highway Bridges (2002). For pullout analysis, a maximum friction factor of 0.5 shall be used for soil against concrete, and  $\tan \phi$  for soil against soil. At rest lateral earth pressure ( $K_0$ ) shall be used to calculate pullout resistance of the stems in the backfill soil, and active earth pressure ( $K_a$ ) shall be considered to act on the back of the facing panel. Furthermore, this criterion is applicable to a center-to-center horizontal spacing of the stems of 5 feet or less. Larger center-to-center spacings of the stems may require that even less of the backfill weight be considered effective – the specific percentage of backfill weight that is considered effective in the case of a stem spacing greater than 5 feet shall be approved by the State Geotechnical Engineer.

The preapproved height for this wall system (25 feet) is less than the standard preapproved height of 33 feet for proprietary wall systems. Use of this wall system for heights greater than 25 feet requires approval by the State Bridge Design Engineer and the State Geotechnical Engineer. Furthermore, the first two T-WALLs constructed on WSDOT projects greater than 20 feet in height shall be instrumented to specifically assess the stability of the wall and the percentage of backfill that is effective in resisting overturning and sliding instability.

Approved details for the T-WALL® system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- In the plan sheet that shows the typical guard rail arrangement, the minimum spacing between the back of the wall and the edge of the guardrail post shall be a minimum of 2 feet, and the select backfill requirements shown shall meet the *Standard Specifications* for Gravel Borrow, or other backfill shown in the contract documents, rather than the specifications shown on the plan sheet.
- Where filter cloth 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 the plan sheets.
- 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.



Typical Cross Sections and Definitions

**STANDARD UNIT DIMENSIONS**

T-WALL® UNIT WEIGHTS		
STEM LENGTH (ft)	SINGLE UNIT WEIGHT (#)	DOUBLE UNIT WEIGHT (#)
4	1600	3200
6	1850	3700
8	2100	4200
10	2350	4700
12	2600	5200
14	2900	5800
16	3150	6300
18	3400	6800
20	3650	7300

STEM LENGTH VARIES

6"

2'-8"

5'-0"

STEM LENGTH VARIES

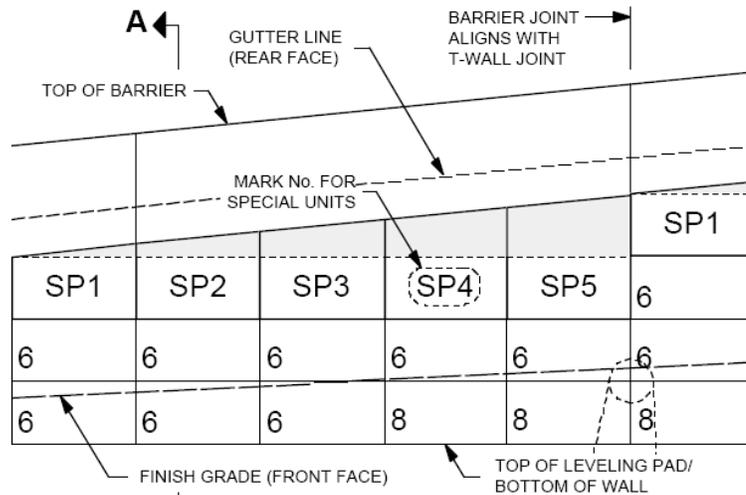
6"

5'-0 1/2"

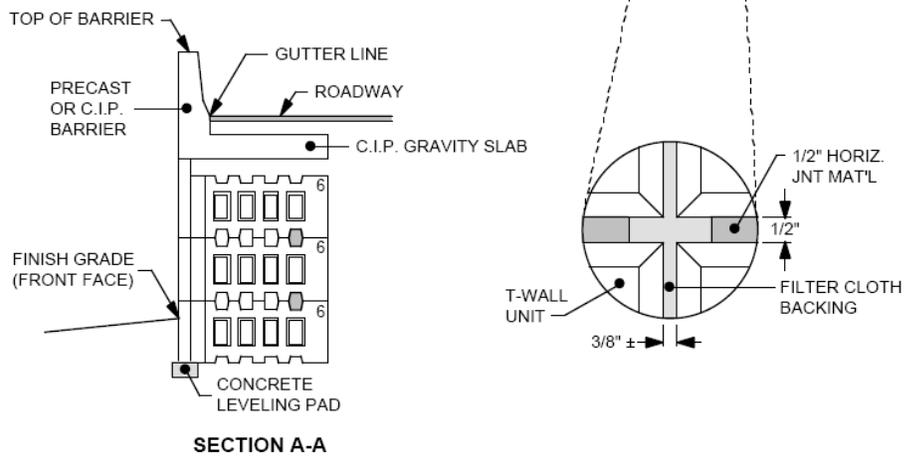
5'-0"

•The size of the T-WALL unit refers to the stem length.  
 •The unit reinforcing steel meets AASHTO and ACI codes.

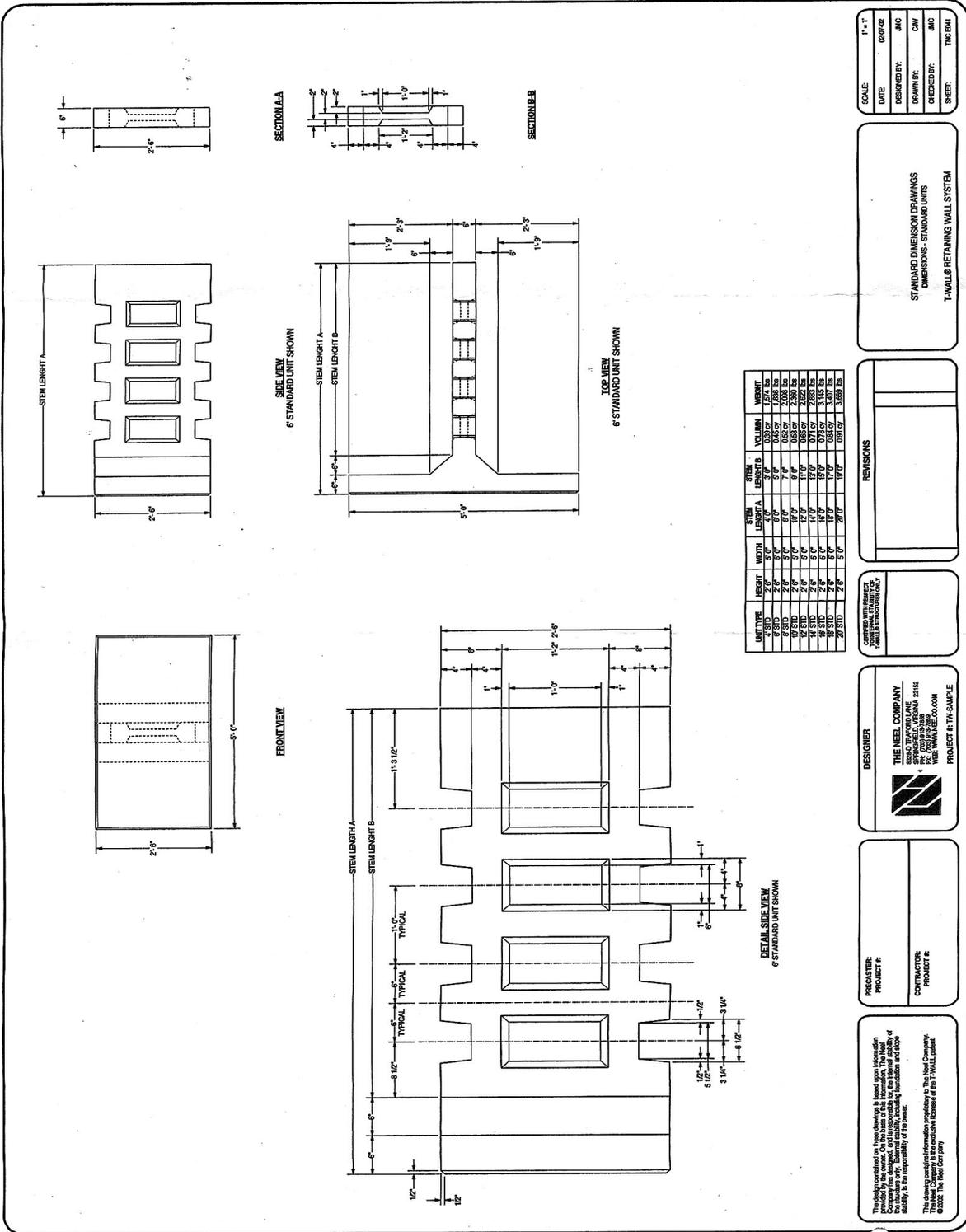
**DETAILS**



**PART ELEVATION - WALL WITH TRAFFIC BARRIER**

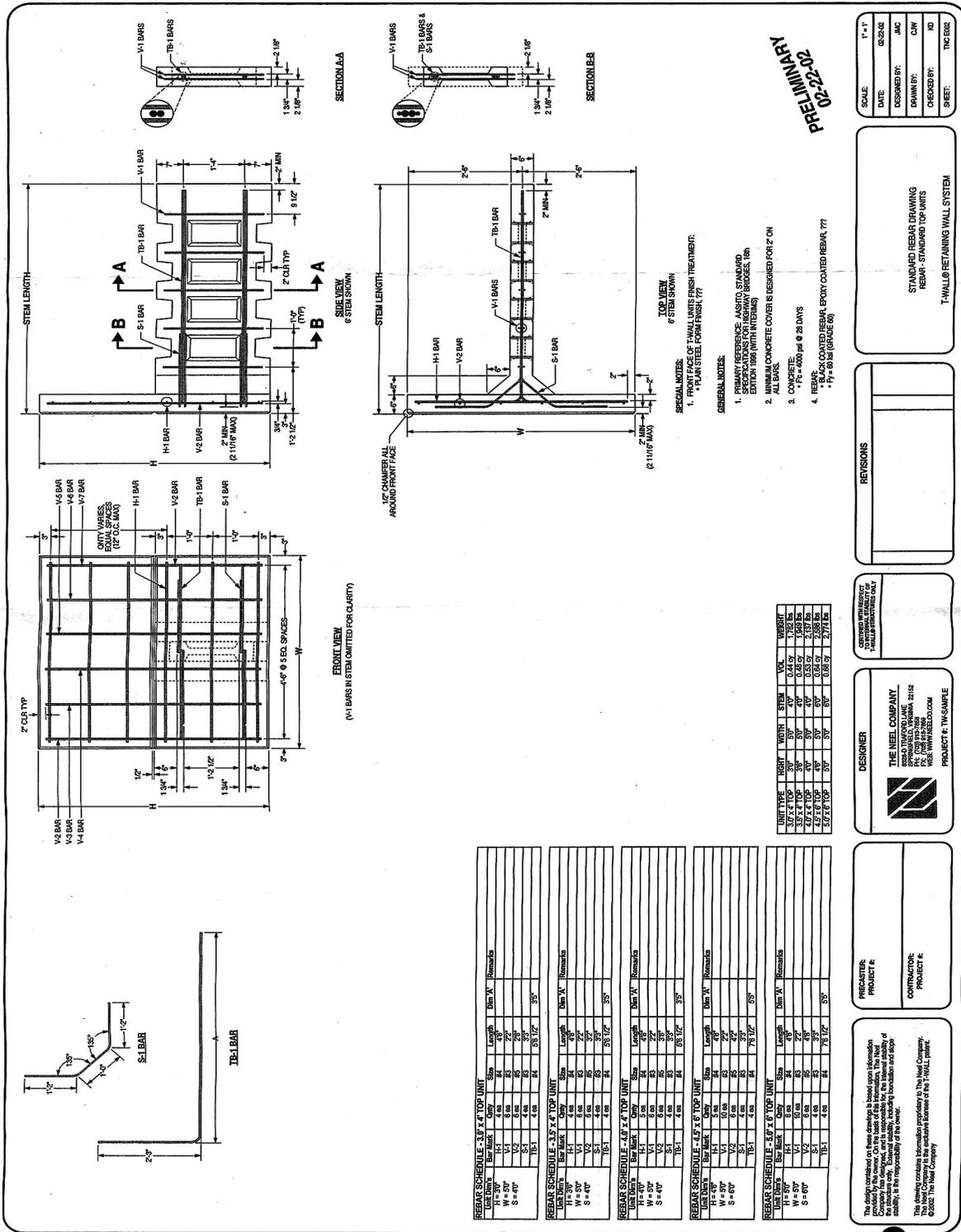


**SECTION A-A**









PRELIMINARY  
2012-2012

SCALE	1" = 1'
DATE	08/26/02
DESIGNED BY	JAC
DRAWN BY	CAF
CHECKED BY	KD
SHEET	TWC-202

STANDARD REBAR DRAWING  
REBAR - STANDARD TOP UNITS  
T-WALL® RETAINING WALL SYSTEM

REVISIONS

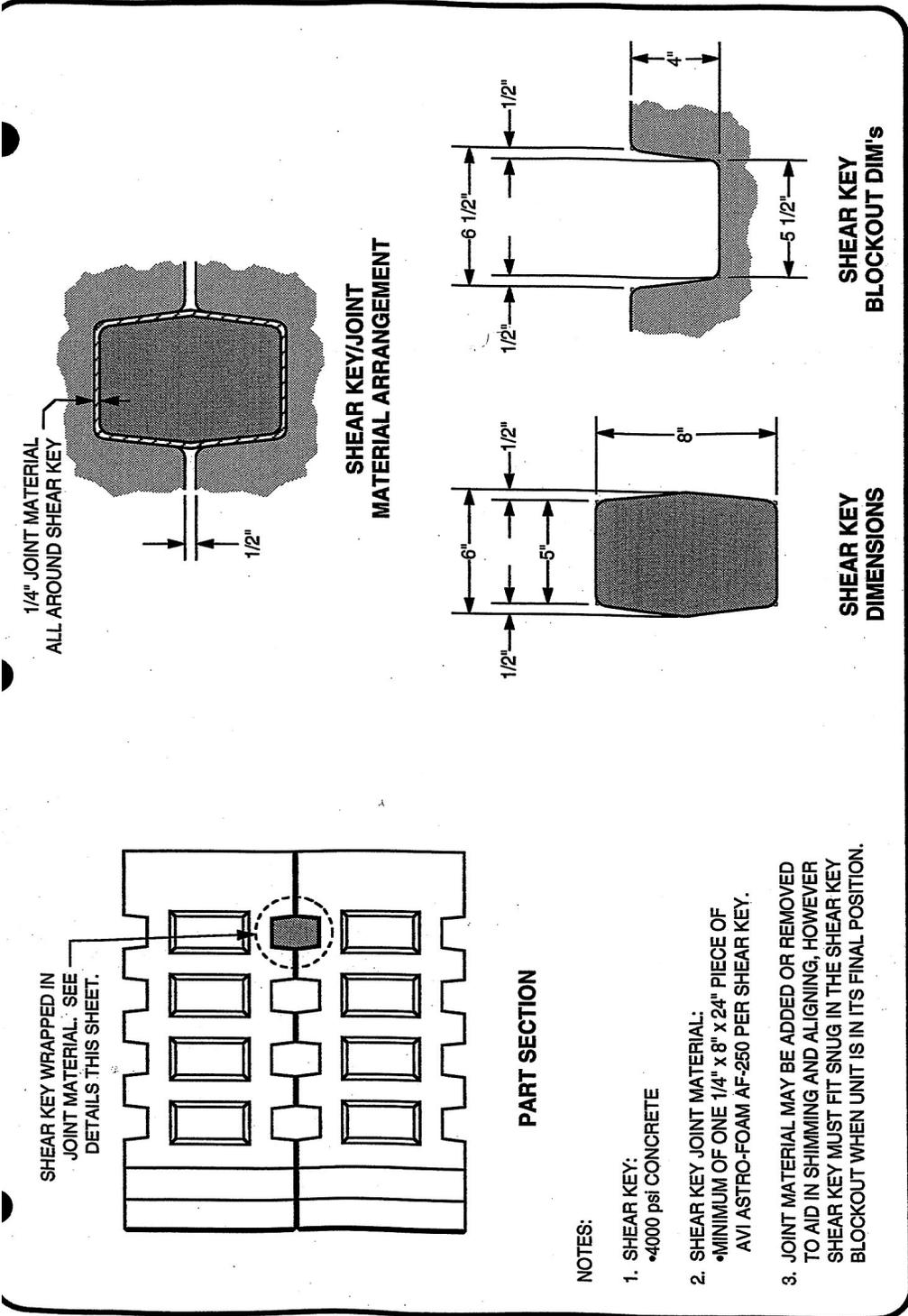
DESIGNER  
**THE NEEL COMPANY**  
REBAR TRACKING UNIT  
FOR PRECAST CONCRETE WALLS  
P.O. BOX 1000, WILSONVILLE, OR 97150  
TEL: 503.535.2200  
WWW.NEELCORP.COM

PROJECT #: TWS-SAMPLE

PRECASTER:  
PROJECT #:

CONTRACTOR:  
PROJECT #:

The Neel Company, Inc. warrants that the rebar and rebar tracking units shown on this drawing were manufactured in accordance with the specifications and standards of the American Iron and Steel Institute (AISI) and the American Concrete Institute (ACI). The Neel Company, Inc. is not responsible for the design or construction of the retaining wall system. The Neel Company, Inc. is not responsible for the design or construction of the retaining wall system. The Neel Company, Inc. is not responsible for the design or construction of the retaining wall system.

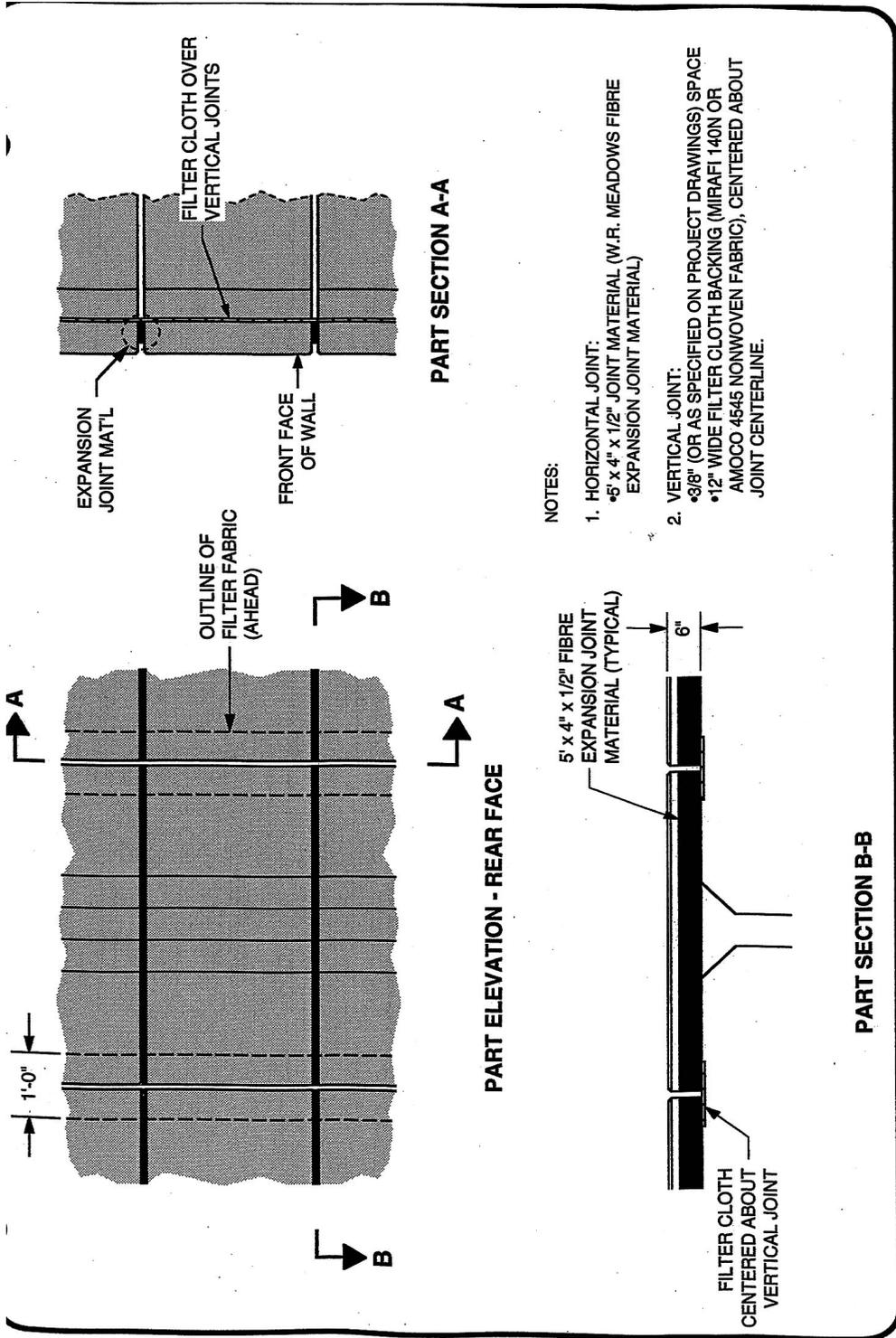


**JOINT MATERIAL DETAILS  
SHEAR KEYS**  
T-WALL® RETAINING WALL SYSTEM

DATE: 20MAY/96  
DRAWN: CAA  
SHEET: 1 OF 1

**THE NEEL COMPANY**  
8328 TRAFORD LANE  
SPRINGFIELD, VA 22152  
PH: (703) 913-7858  
FX: (703) 913-7689

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**JOINT MATERIAL DETAILS  
NORMAL CONDITIONS**

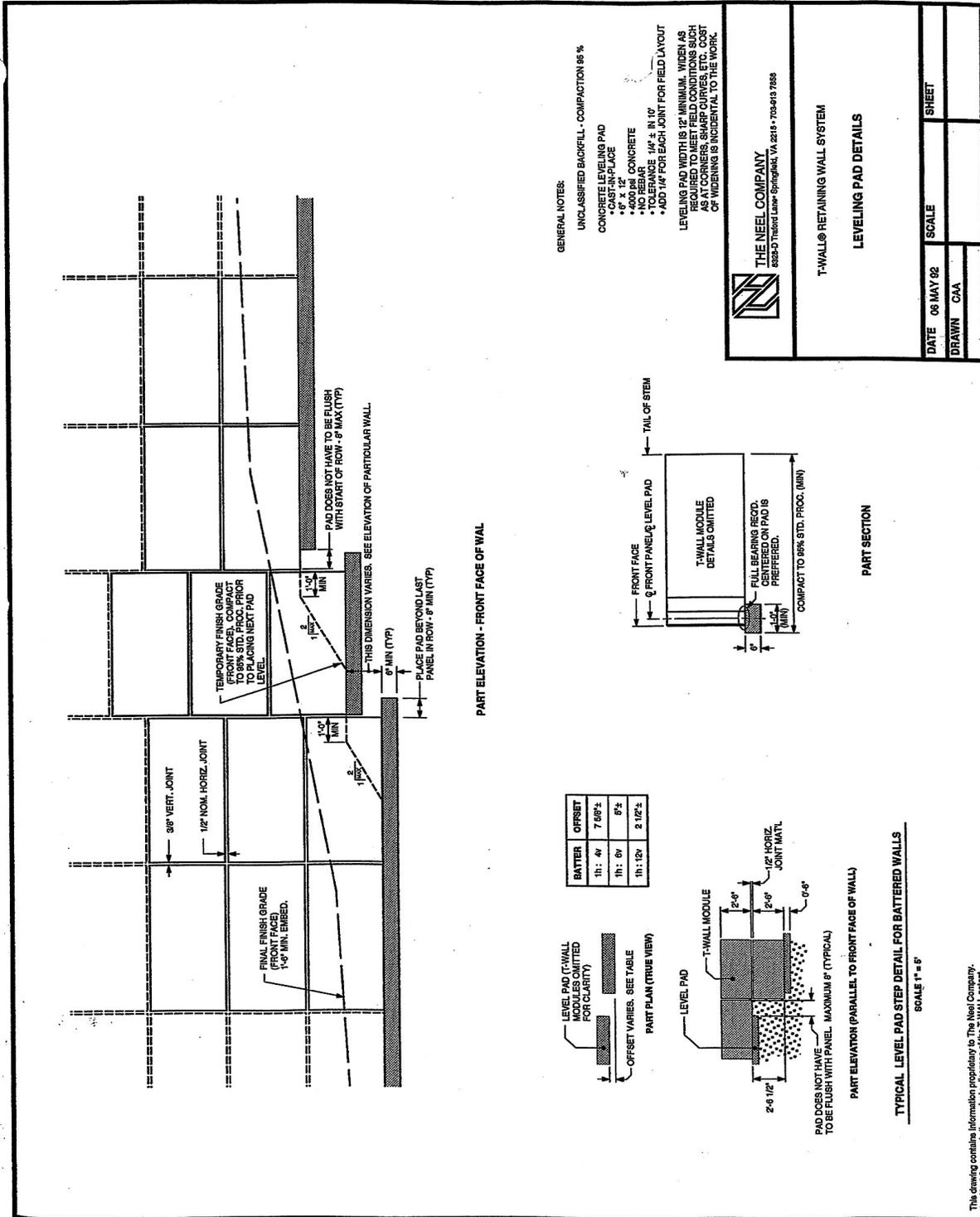
T-WALL® RETAINING WALL SYSTEM

DATE: 09MAY96  
DRAWN: CAA  
SHEET: 1 OF 1

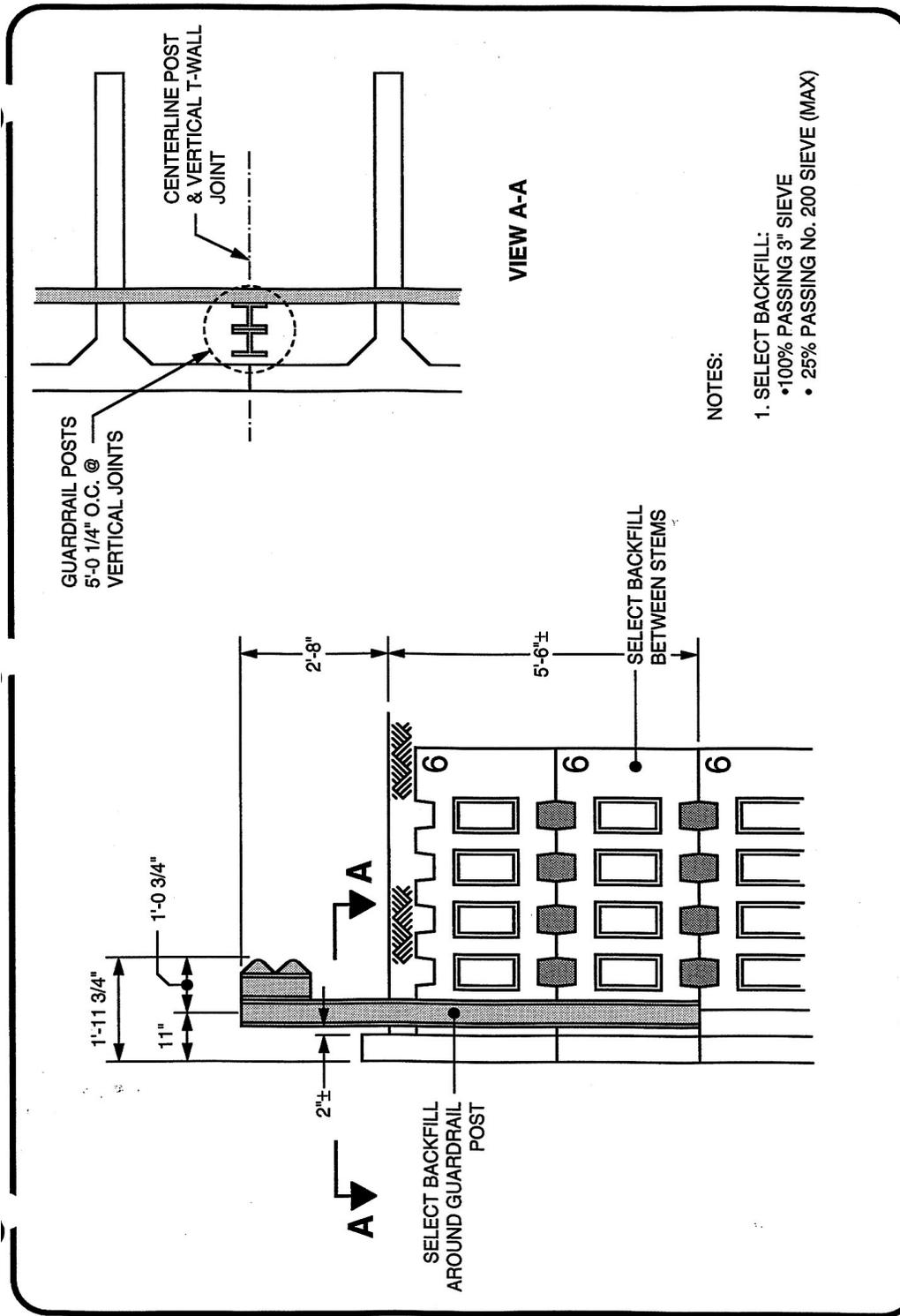
**THE NEEL COMPANY**  
8328-D TRAFORD LANE  
SPRINGFIELD, VA 22152  
PH: (703) 913-7858  
FX: (703) 913-7859



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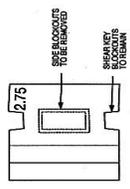
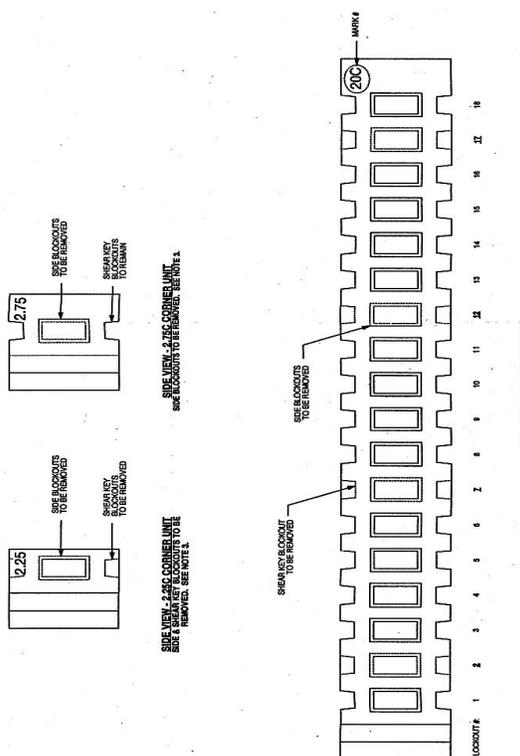


**TYPICAL POST & BEAM  
GUARDRAIL ARRANGEMENT**  
T-WALL® RETAINING WALL SYSTEM

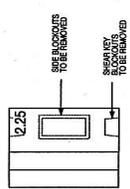
DATE: 24JUN96  
DRAWN: CAA  
SHEET: 1 OF 1

**THE NEEL COMPANY**  
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SPRINGFIELD, VA 22152  
PH: (703) 913-7858  
FX: (703) 913-7859

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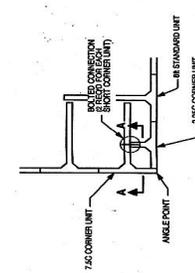
SIDE VIEW - 275C CORNER UNIT  
SIDE BLOCKOUTS TO BE REMOVED, SEE NOTE A.



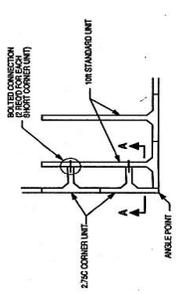
SIDE VIEW - 275C CORNER UNIT  
SIDE & SHEAR KEY BLOCKOUTS TO BE REMOVED, SEE NOTE A.

ANGLE POINT		FIRST ROW		SECOND ROW	
6	8C	2.75C	6	2.25C	8
2.75C	2.75C	6C	6	2.25C	8
7.5	7.5C	10C	10	10C	10
2.75C	2.75C				

PART ELEVATION



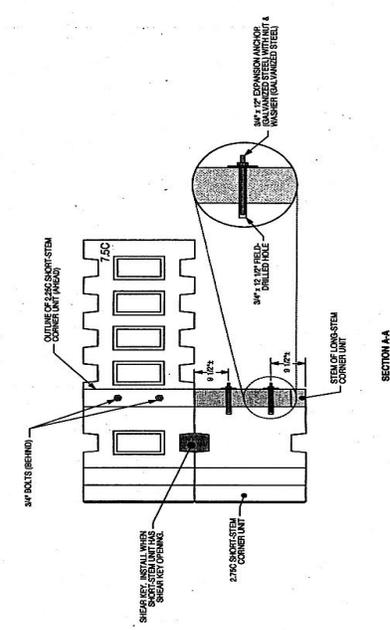
PART PLAN - SECOND ROW



PART PLAN - FIRST ROW

TYPICAL CORNER UNIT ARRANGEMENT  
STEM ELEVATIONS MAY VARY. SEE SPECIFIC  
ELEVATIONS FOR PROPER UNITS  
NO SCALE

SIDE VIEW - LONG STEM CORNER UNIT  
SIDE BLOCKOUTS TO BE REMOVED, SEE NOTE A.  
SHEAR KEY BLOCKOUTS TO REMAIN.  
BASE LOCATIONS AS INDICATED, SEE NOTE B.



SECTION A-A

- NOTES:
1. WALLING UNITS ARE SUBSTITUTES BY A CONTRACTOR IN MATERIALS, UNLESS INDICATED OTHERWISE.
  2. SIDE BLOCKOUTS TO BE REMOVED.
  3. SIDE BLOCKOUTS TO BE REMOVED.
  4. SIDE BLOCKOUTS TO BE REMOVED.
  5. SIDE BLOCKOUTS TO BE REMOVED.
  6. SIDE BLOCKOUTS TO BE REMOVED.
  7. SIDE BLOCKOUTS TO BE REMOVED.
  8. SIDE BLOCKOUTS TO BE REMOVED.
  9. SIDE BLOCKOUTS TO BE REMOVED.
  10. SIDE BLOCKOUTS TO BE REMOVED.
  11. SIDE BLOCKOUTS TO BE REMOVED.
  12. SIDE BLOCKOUTS TO BE REMOVED.
  13. SIDE BLOCKOUTS TO BE REMOVED.
  14. SIDE BLOCKOUTS TO BE REMOVED.
  15. SIDE BLOCKOUTS TO BE REMOVED.
  16. SIDE BLOCKOUTS TO BE REMOVED.
  17. SIDE BLOCKOUTS TO BE REMOVED.
  18. SIDE BLOCKOUTS TO BE REMOVED.
  19. SIDE BLOCKOUTS TO BE REMOVED.
  20. SIDE BLOCKOUTS TO BE REMOVED.

PREPARED BY

SCALE:	DATE:	DESIGNED BY:	DRAWN BY:	CHECKED BY:	IN CHARGE:

STANDARD DETAILS  
RETAINING WALL  
STANDARD DETAILS - 90° OUTSIDE CORNER  
T-WALL® RETAINING WALL SYSTEM

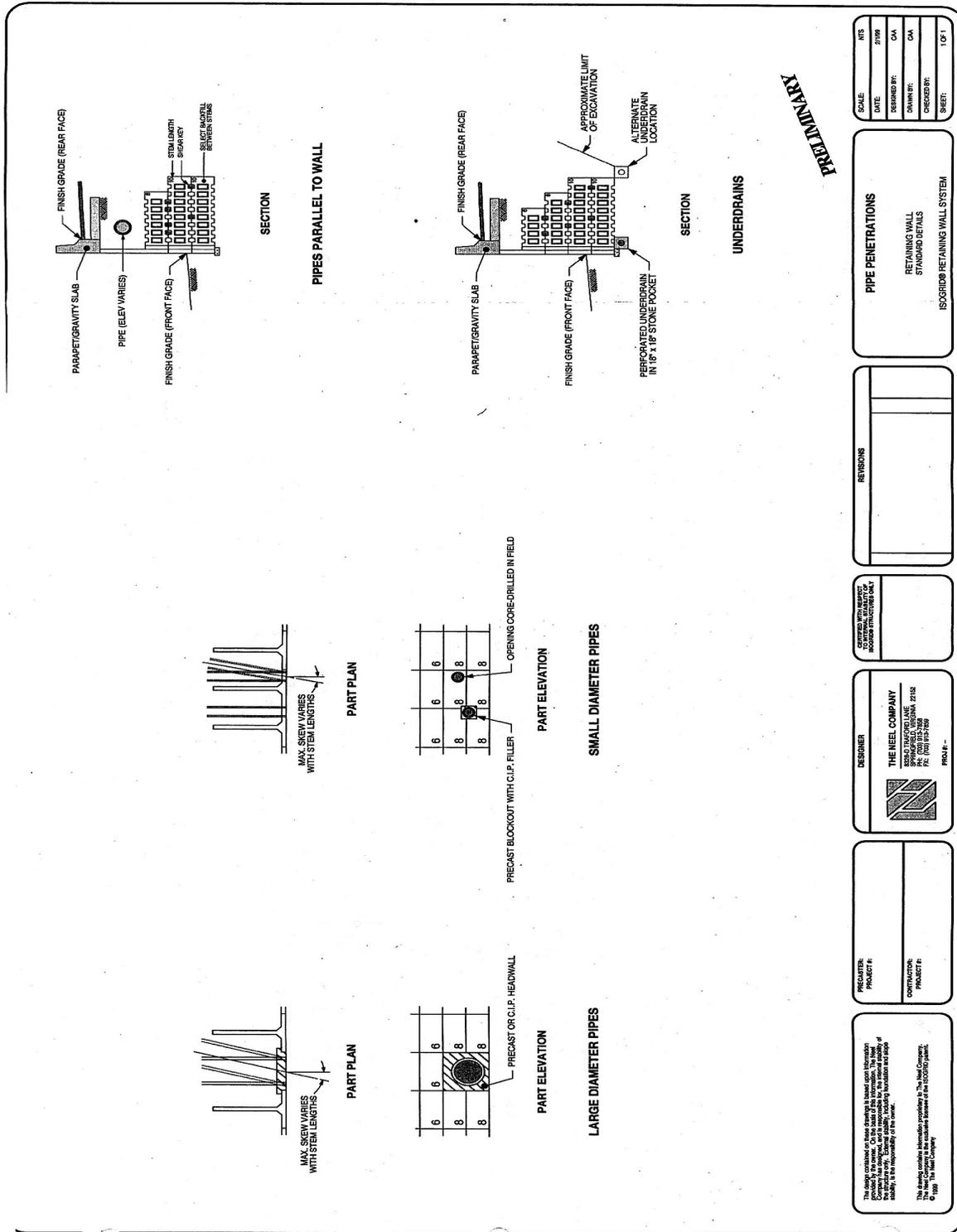
REVISIONS

DESIGNER  
THE NEEL COMPANY  
SPRINGFIELD, VIRGINIA 22152  
PHONE: (703) 831-7288  
FAX: (703) 831-7289

PREPARED BY  
PROJECT #

CONTRACTOR  
PROJECT #

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SCALE	R/S
DATE	2/1998
DESIGNED BY:	CA
DRAWN BY:	CA
CHECKED BY:	
SHEET:	1 OF 1

**PIPE PENETRATIONS**  
RETAINING WALL  
STANDARD DETAILS  
ISORRID® RETAINING WALL SYSTEM

**REVISIONS**

**GENERAL NOTES**  
1. SEE DRAWING FOR ALL DIMENSIONS AND TOLERANCES.  
2. ALL DIMENSIONS ARE UNLESS OTHERWISE SPECIFIED.

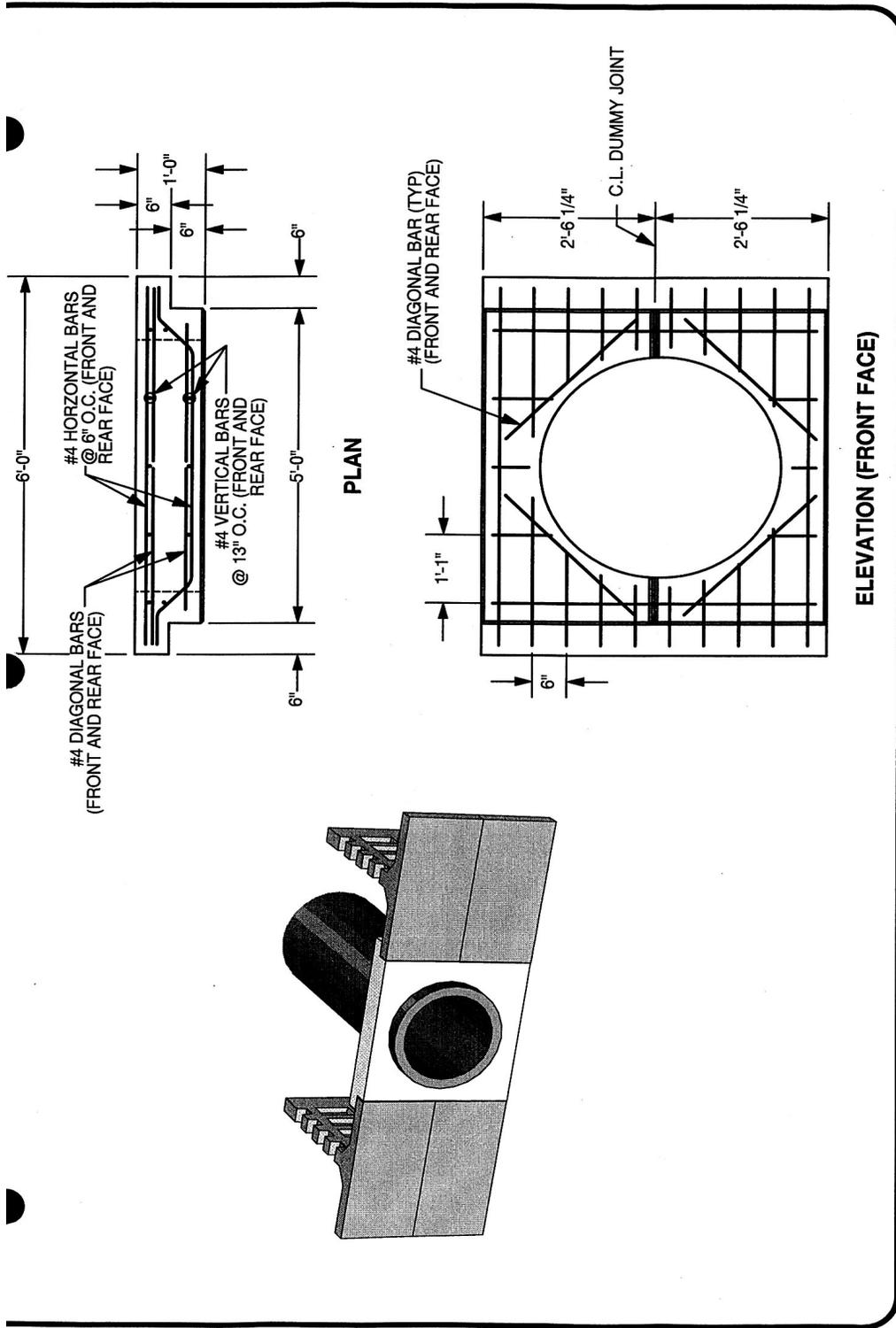
**DESIGNER**  
**THE NEEL COMPANY**  
8334 TRANSOM LANE  
MILWAUKEE, WI 53218  
TEL: (414) 312-2400  
FAX: (414) 312-2404  
PROJECT # \_\_\_\_\_

**PRECASTER**  
PROJECT # \_\_\_\_\_

**CONTRACTOR**  
PROJECT # \_\_\_\_\_

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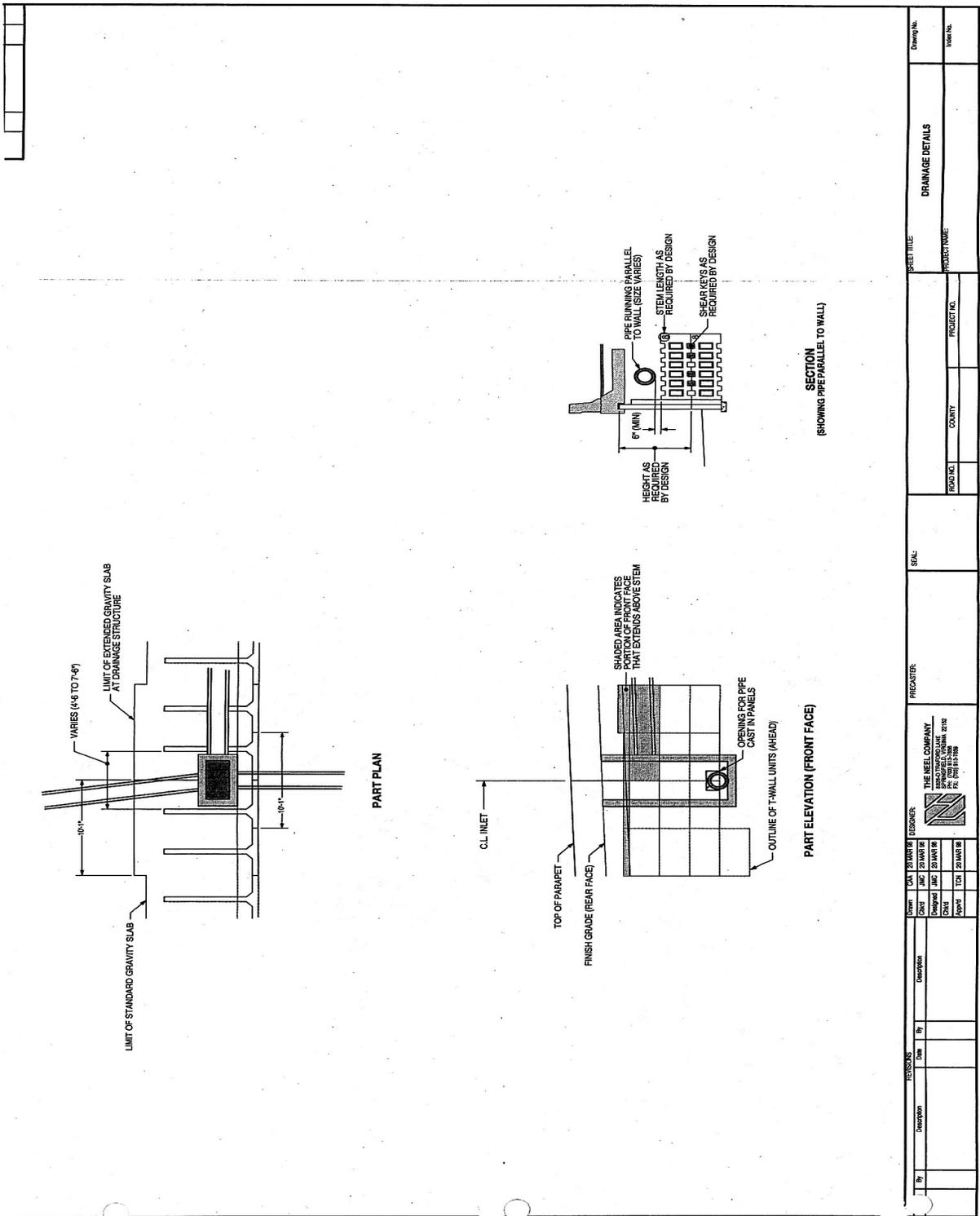


DATE: 21MAY99  
 DRAWN: CAA  
 SHEET: 1 OF 1

**STANDARD DETAILS**  
**PRECAST HEADWALL**  
 T-WALL® RETAINING WALL SYSTEM

**THE NEEL COMPANY**  
 8328-D TRAFORD LANE  
 SPRINGFIELD, VA 22152  
 PH: (703) 913-7858 TNC#  
 FX: (703) 913-7859

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DATE		BY	DESCRIPTION	REVISIONS		THE NEEL COMPANY SPECIALTY CONCRETE PIPE PO BOX 101 P.O. BOX 101-1000		PROJECT NO.		PROJECT NAME		COUNTY		PARISH		DRAINAGE DISTRICT		DRAINAGE NO.		DRAWING NO.	
Checked	DATE	BY	DESCRIPTION	1	DATE	BY	DESCRIPTION	PROJECT NO.		PROJECT NAME		COUNTY		PARISH		DRAINAGE DISTRICT		DRAINAGE NO.		DRAWING NO.	
Drawn	20 MAR 08	JAC		2	20 MAR 08	JAC		PROJECT NO.		PROJECT NAME		COUNTY		PARISH		DRAINAGE DISTRICT		DRAINAGE NO.		DRAWING NO.	
Checked	20 MAR 08	JAC		3	20 MAR 08	JAC		PROJECT NO.		PROJECT NAME		COUNTY		PARISH		DRAINAGE DISTRICT		DRAINAGE NO.		DRAWING NO.	
Designed	20 MAR 08	JAC		4	20 MAR 08	JAC		PROJECT NO.		PROJECT NAME		COUNTY		PARISH		DRAINAGE DISTRICT		DRAINAGE NO.		DRAWING NO.	
Checked	20 MAR 08	JAC		5	20 MAR 08	JAC		PROJECT NO.		PROJECT NAME		COUNTY		PARISH		DRAINAGE DISTRICT		DRAINAGE NO.		DRAWING NO.	
Approved	20 MAR 08	TOK		6	20 MAR 08	TOK		PROJECT NO.		PROJECT NAME		COUNTY		PARISH		DRAINAGE DISTRICT		DRAINAGE NO.		DRAWING NO.	



# **Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls**

## **Appendix 15-L**

---

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply to the design of the Reinforced Earth<sup>TM</sup> concrete 5 feet × 5 feet panel faced retaining wall:

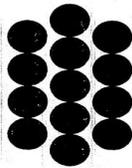
No HITEC evaluation report is currently available for this wall system. Design procedures for specific elements of the wall system have been provided to WSDOT in a binder dated March 29, 2004. The design procedures used by RECO are based on the AASHTO Standard Specifications for Highway Bridges (2002). 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<sup>TM</sup> 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.

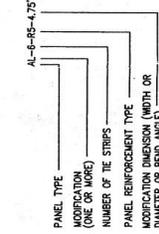


**The Reinforced Earth Company**  
 8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175

# STANDARD PRECAST PANEL SHOP DRAWINGS - SQUARE PANELS

**PANEL GENERAL NOTES**

1. REINFORCEMENT SHALL COMPLY TO THE ASTM A618, GRADE 60 (AS SHOWN) AND SHALL BE PROVIDED 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 1 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 STRIPS. IF MESH REINFORCEMENT IS USED, THE 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. THE 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 PANELS 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, ALL FABRICATION DETAILS SHALL BE SHOWN WITH THE REBAR REPRESENTATION. ALL FABRICATION DETAILS SHALL BE PROVIDED ALONG WITH THE REBAR REPRESENTATION.
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.

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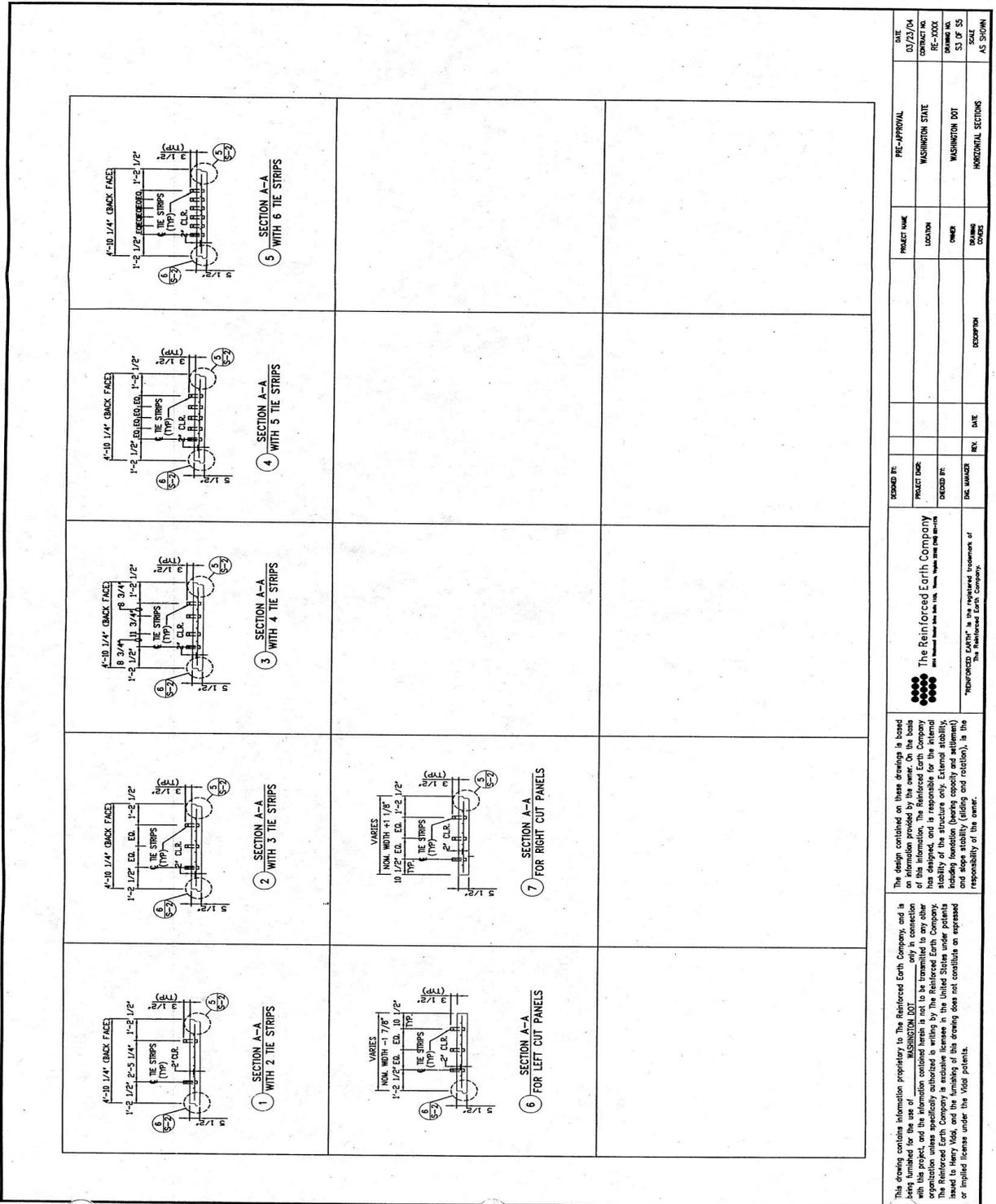
**The Reinforced Earth Company**  
 8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1175

DESIGNED BY: [Blank]  
 PROJECT DATE: [Blank]  
 CHECKED BY: [Blank]  
 DATE: [Blank]

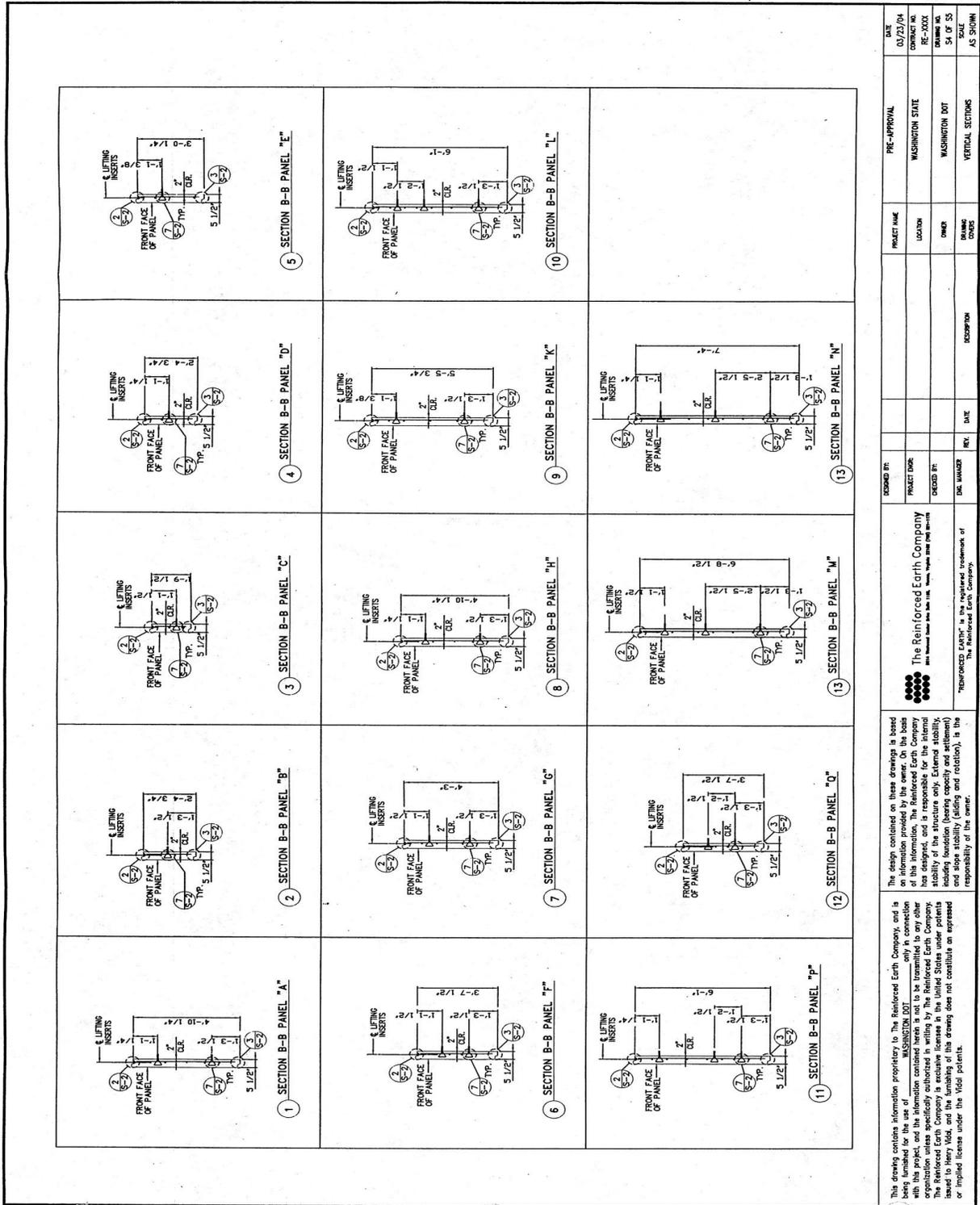
DATE: [Blank]  
 REVISION: [Blank]

DATE	DESCRIPTION	BY	DATE	DESCRIPTION	BY
03/23/04	PRE-APPROVAL				
RE-XXXX	WASHINGTON STATE				
SI OF SS	WASHINGTON DOT				
SCALE	GENERAL NOTES				
AS SHOWN					





<p>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, bearing capacity, sliding and rotation, in the and slope stability (sliding and rotation), in the responsibility of the owner.</p>		<p>The drawing contains information proprietary to The Reinforced Earth Company, and is being furnished for the use of _____ only in connection with this project, and the information contained herein is not to be transmitted to any other organization unless specifically authorized in writing by The Reinforced Earth Company. The Reinforced Earth Company's exclusive license in the United States under patents and/or other intellectual property rights in this drawing does not constitute an expressed or implied license under the Void patents.</p>	
<p>DESIGNED BY:                    The Reinforced Earth Company                  1000 North 10th Street, Suite 1000, Tacoma, WA 98402-1000                  Phone: 253.733.8100                  Fax: 253.733.8101                  Email: info@reco.com</p>	<p>PROJECT NAME:                  WASHINGTON STATE                  WASHINGTON DOT                  DRAWING NO.                  RE-000X                  DRAWING NO.                  S3 OF S5</p>	<p>DATE:                  03/23/04</p>	<p>SCALE:                  AS SHOWN</p>
<p>PROJECT DATE:                  03/23/04</p>	<p>PROJECT LOCATION:                  WASHINGTON STATE</p>	<p>PRE-APPROVAL:</p>	<p>CONTRACT NO.                  RE-000X</p>
<p>DESIGNED BY:                  [Signature]</p>	<p>DATE:                  03/23/04</p>	<p>PROJECT NAME:                  WASHINGTON STATE</p>	<p>CONTRACT NO.                  RE-000X</p>
<p>DATE:                  03/23/04</p>	<p>REVISIONS:</p>	<p>PROJECT NAME:                  WASHINGTON STATE</p>	<p>CONTRACT NO.                  RE-000X</p>
<p>DATE:                  03/23/04</p>	<p>REVISIONS:</p>	<p>PROJECT NAME:                  WASHINGTON STATE</p>	<p>CONTRACT NO.                  RE-000X</p>



DATE	PRE-APPROVAL	PROJECT NAME	DESCRIPTION
03/23/04	WASHINGTON STATE	LOCATION	
RE-XXX	WASHINGTON DOT	OWNER	
SCALE	VERTICAL SECTIONS	DRAWING NUMBER	
5/4 OF 55		DATE	
AS SHOWN		REVISION	

DESIGNED BY	CHECKED BY	DATE

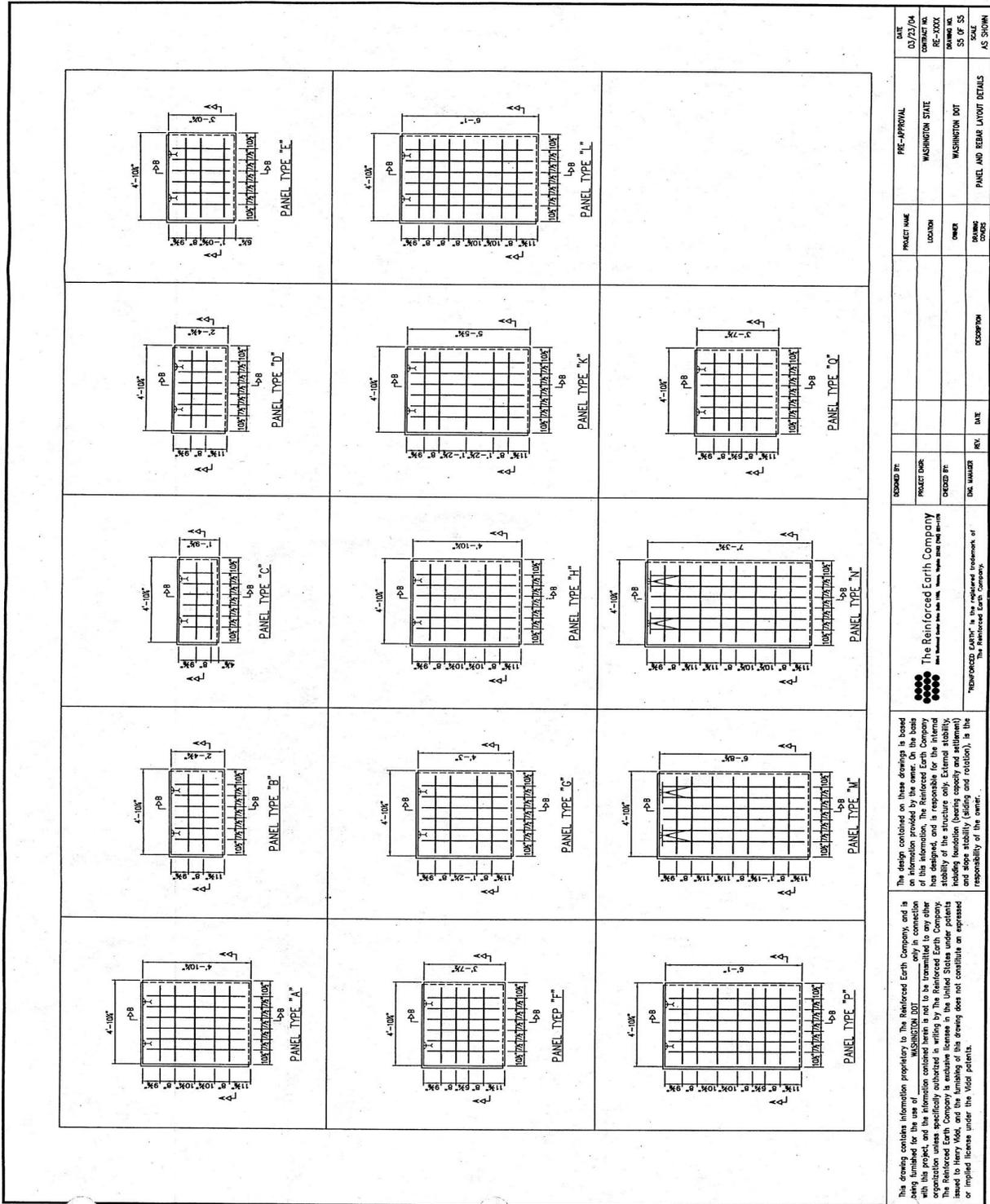
PROJECT NAME	PROJECT NO.	DATE

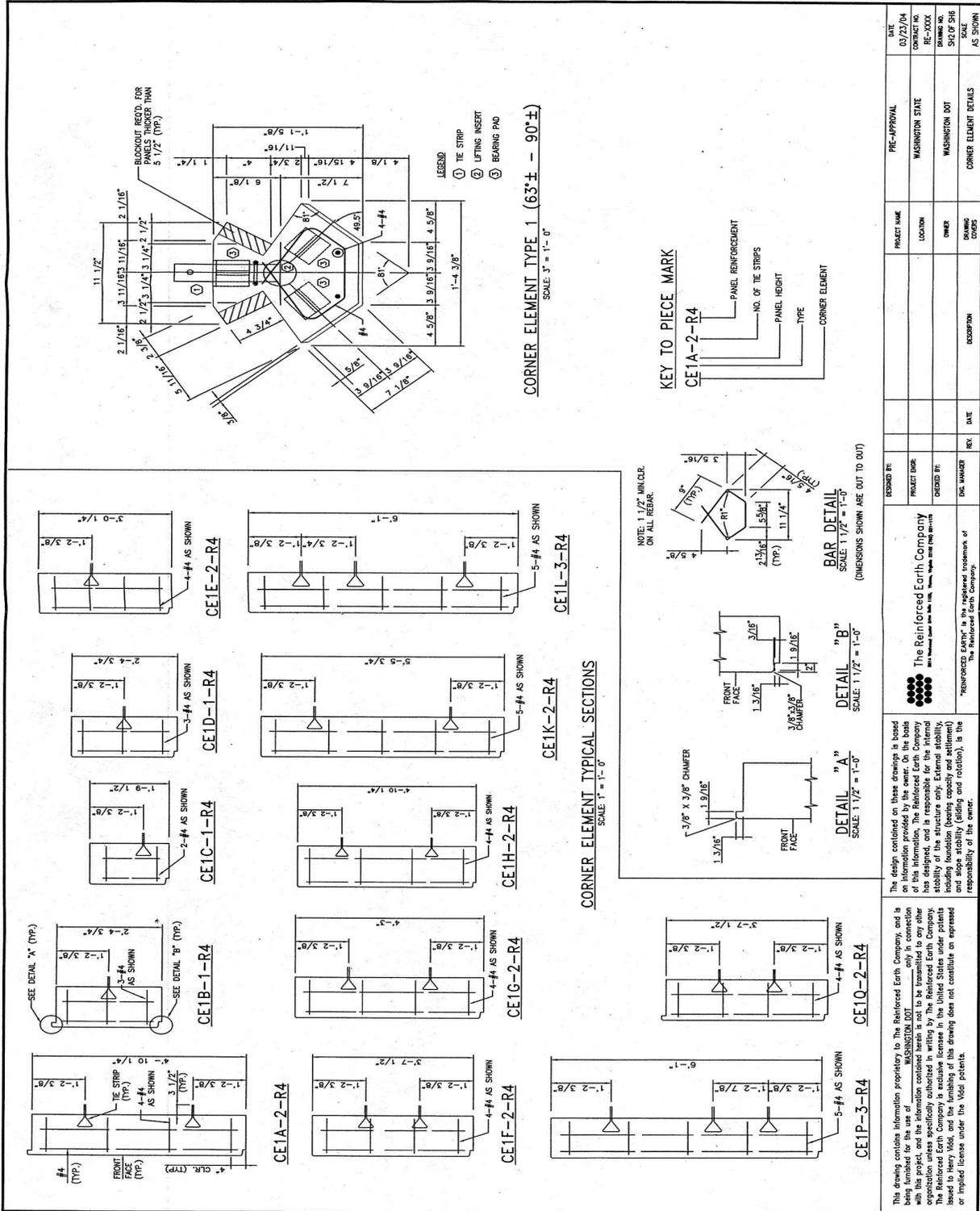
The Reinforced Earth Company  
 11111 1st Avenue, Seattle, WA 98101  
 "REINFORCED EARTH" is the registered trademark of The Reinforced Earth Company.

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 the structure only. Extensive stability, bearing capacity and settlement, and slope stability (sliding and rotation), is the responsibility of the owner.

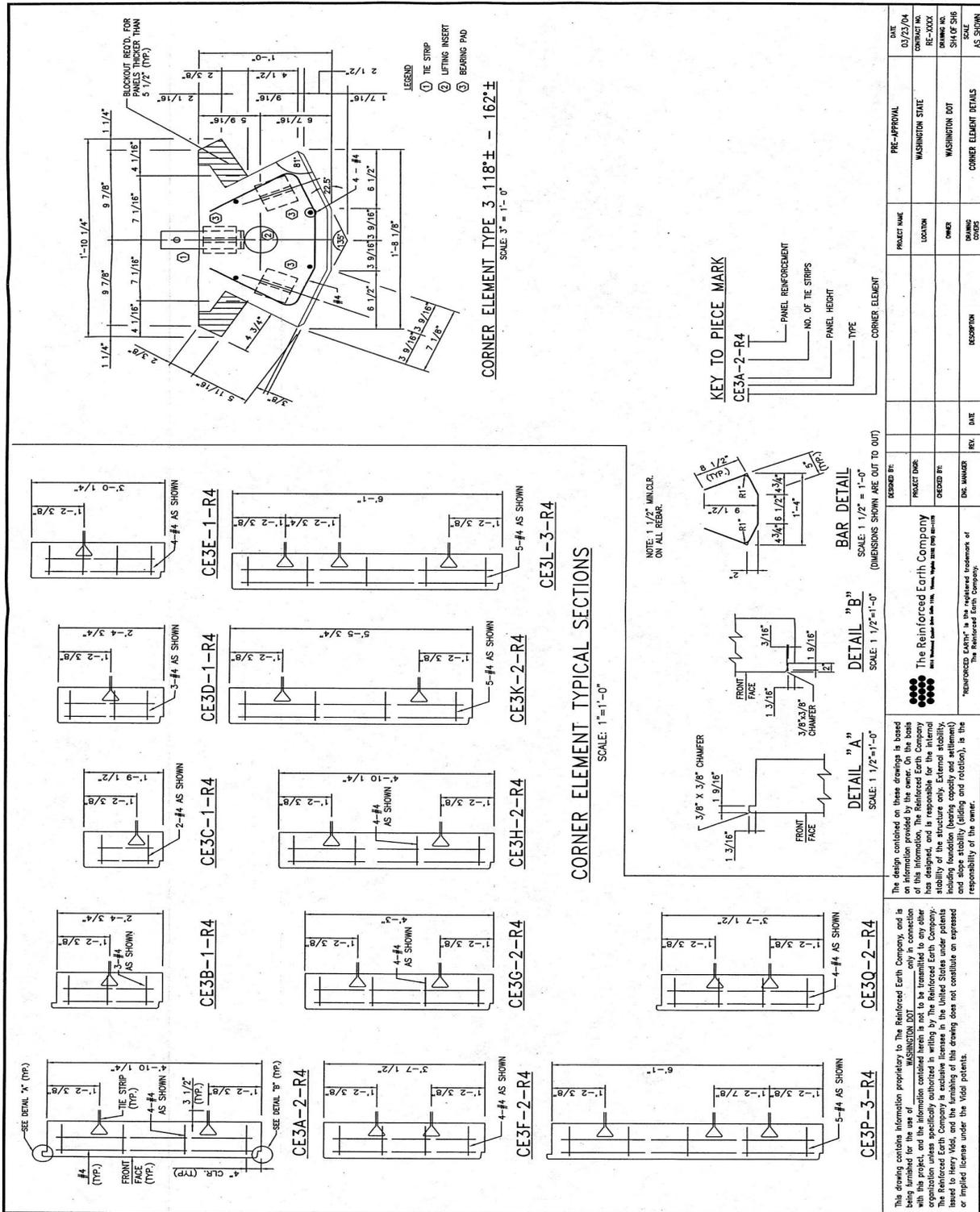
This drawing contains information proprietary to The Reinforced Earth Company, and is being furnished for the use of WASHINGTON DOT only in connection with the project, and the information contained herein is not to be transmitted to any other person or organization without the written consent of The Reinforced Earth Company. The design and construction of the structure are subject to the terms and conditions set forth in the contract between the owner and The Reinforced Earth Company, and the furnishing of this drawing does not constitute an expressed or implied license under the Vidal patents.



<p>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 bearing capacity, settlement, and slope stability (sliding and rotation), is the responsibility of the owner.</p>		<p>The drawing contains information proprietary to The Reinforced Earth Company, and is being furnished for the use of WASHINGTON DOT only for the project with this project, and the information contained herein is not to be transmitted to any other organization unless specifically authorized in writing by The Reinforced Earth Company. The Reinforced Earth Company is exclusive licensee in the United States under patents issued to Henry Vides, and the turning of this drawing does not constitute an expressed or implied license under the Vides patents.</p>	
<p><b>The Reinforced Earth Company</b>                  10000 10th Avenue, Suite 1000, Seattle, WA 98144                  Phone: 206.276.7979                  Fax: 206.276.7978</p>	<p>DATE: 03/23/04                  CONTRACT NO.: RE-XXXX                  DRAWING NO.: SS OF SS                  SCALE: AS SHOWN</p>	<p>PROJECT NAME: WASHINGTON STATE                  LOCATION: CHERRY                  DRAWING NO.: SS OF SS</p>	<p>PRE-APPROVAL: WASHINGTON STATE                  DATE: 03/23/04</p>
<p>FORMED BY: PROJECT NO.: CHECKED BY: DATE: 03/23/04</p>	<p>PROJECT NO.: LOCATION: CHERRY</p>	<p>DESCRIPTION: PANEL AND BEAM UPOUT DETAILS</p>	<p>DATE: 03/23/04</p>





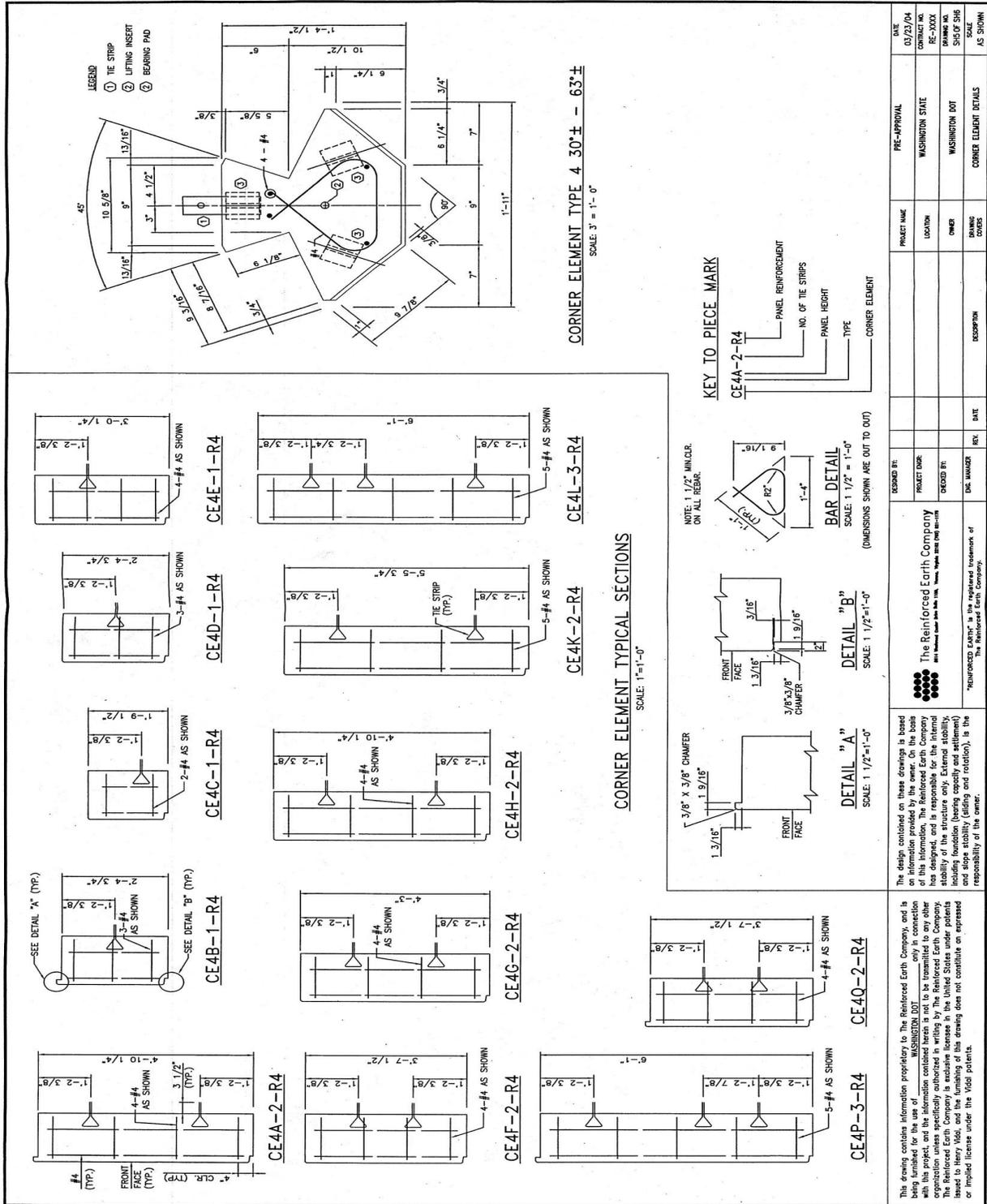


DATE	03/22/04
CONTRACT NO.	WASHINGTON STATE
REV. NO.	RE-100X
DRAWING NO.	WASHINGTON DOT
SHEET NO.	CORNER ELEMENT DETAILS
SCALE	AS SHOWN

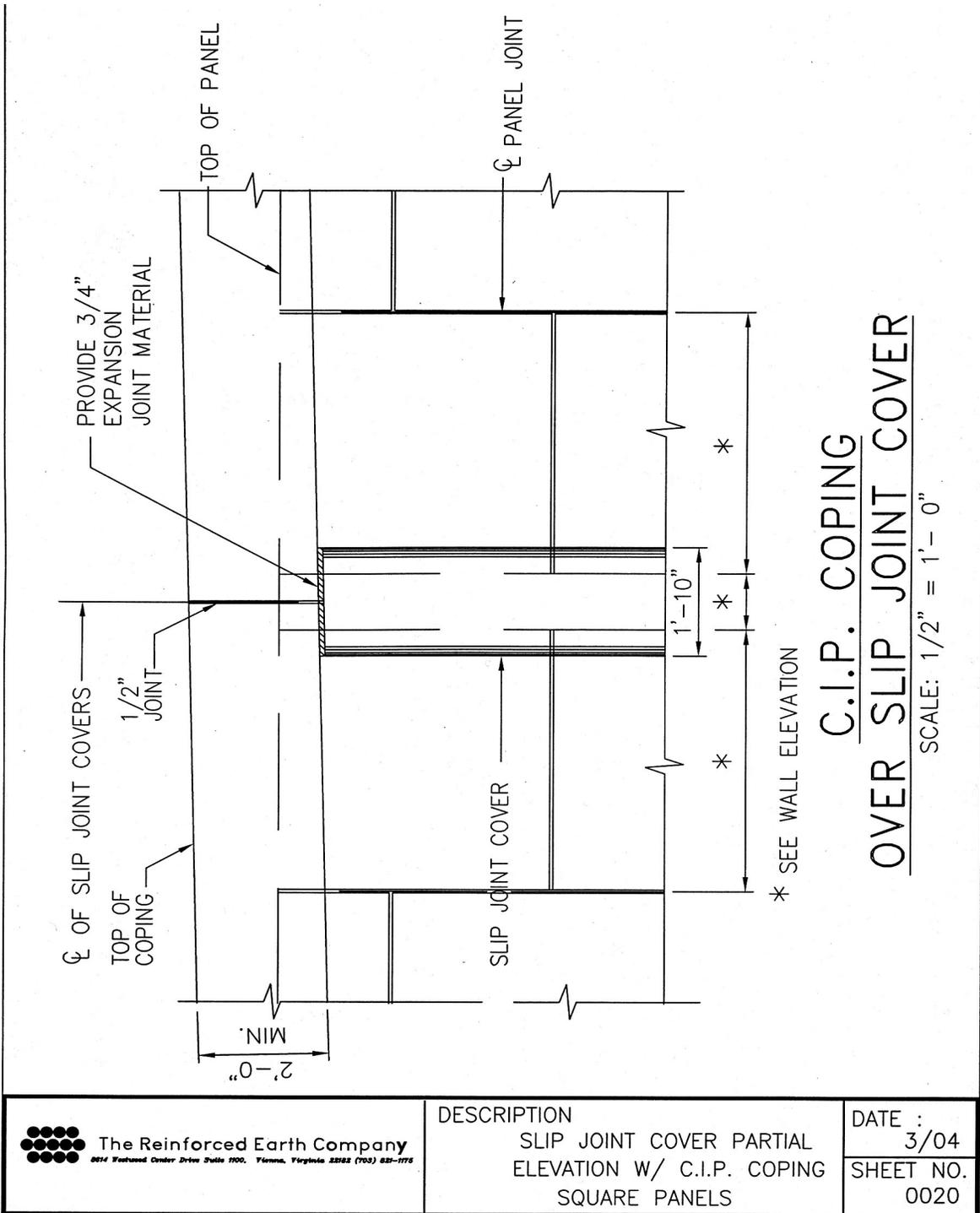
PROJECT NAME	WASHINGTON STATE
LOCATION	WASHINGTON DOT
OWNER	WASHINGTON DOT
DRAWING NO.	CORNER ELEMENT DETAILS
SCALE	AS SHOWN

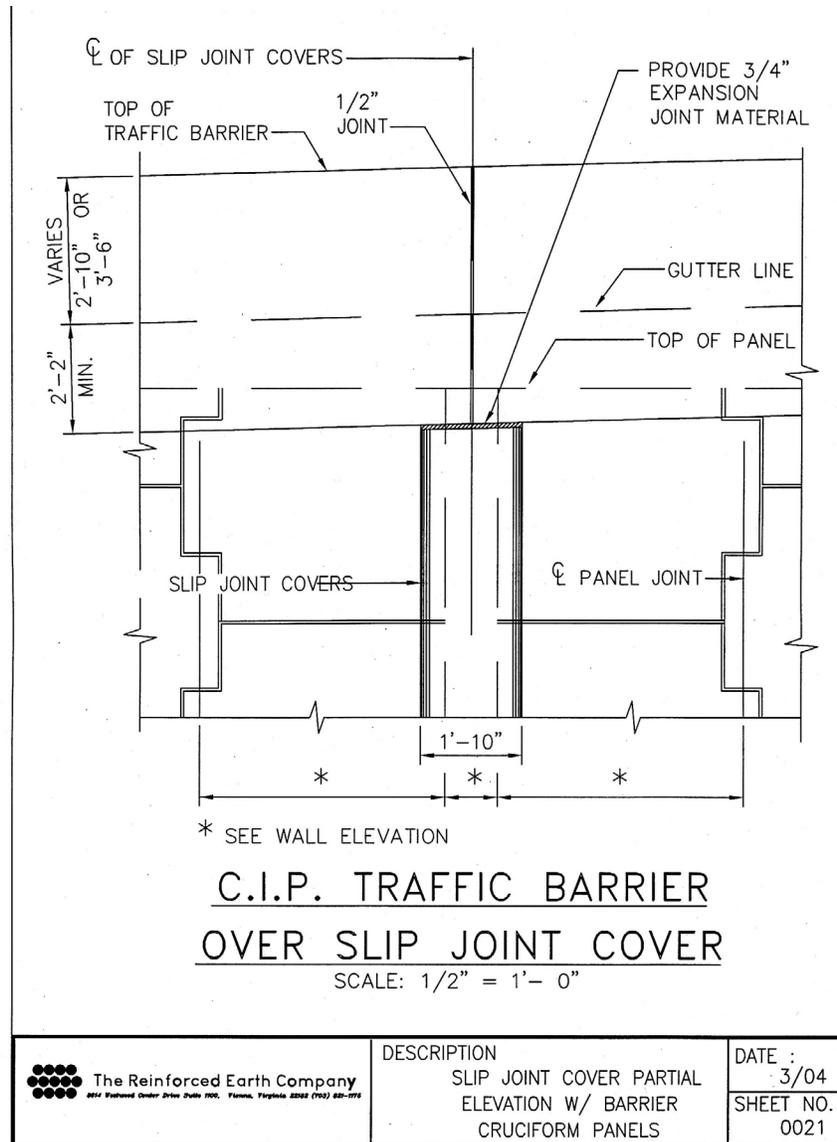
The design contained on these drawings is based on information provided by the owner. On the basis of this information, The Reinforced Earth Company has designed, and is responsible for the internal stability of the structure only. External stability, including global stability (sliding and rotation), is the responsibility of the owner.

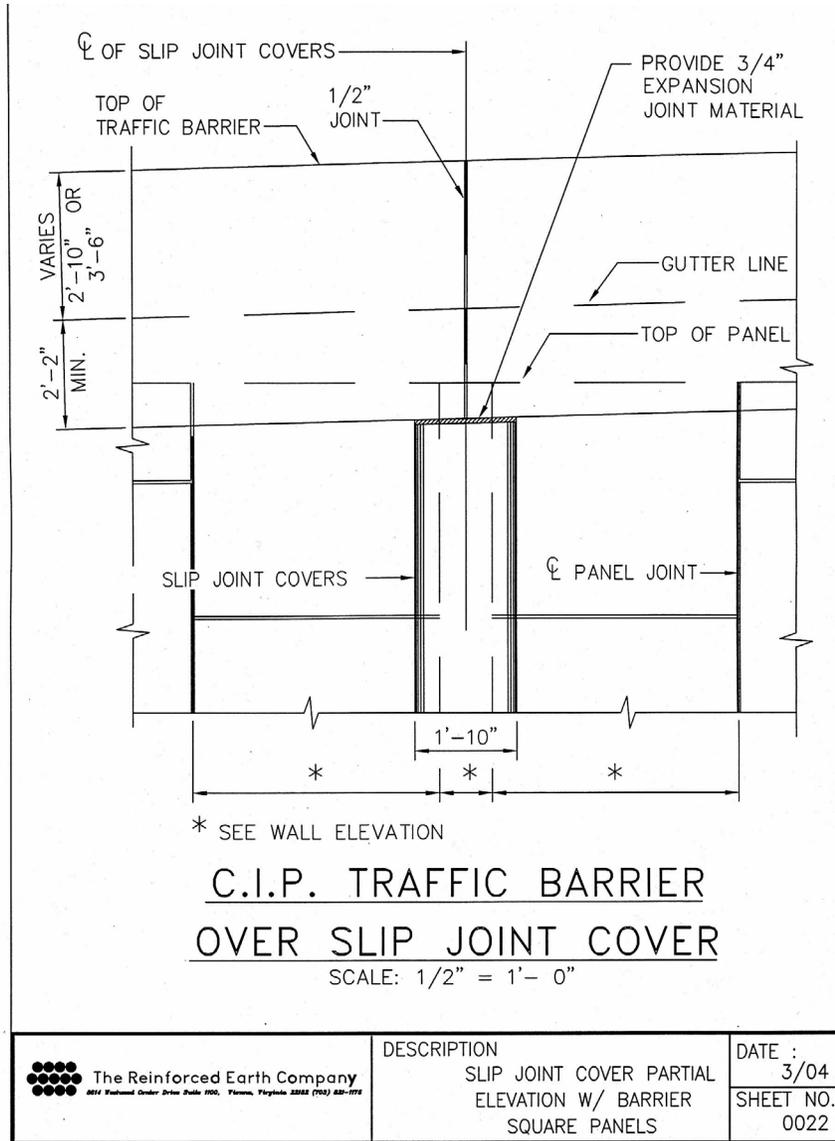
The drawing contains information proprietary to The Reinforced Earth Company, and is being furnished for the use of WASHINGTON DOT only in connection with this project, and the information contained herein is not to be transmitted to any other organization unless specifically authorized in writing by The Reinforced Earth Company. The Reinforced Earth Company is an exclusive licensee in the United States under patents issued to Henry Meiss, and the issuing of this drawing does not constitute an expressed or implied license under the said patents.

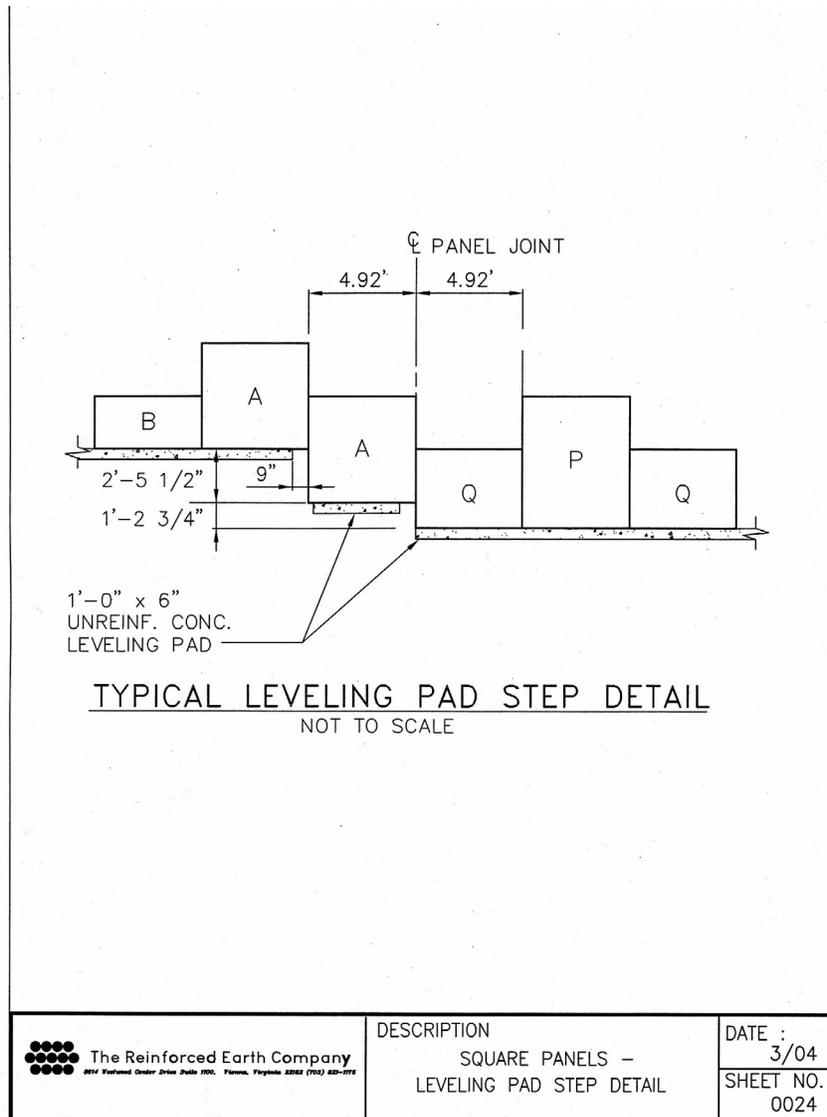


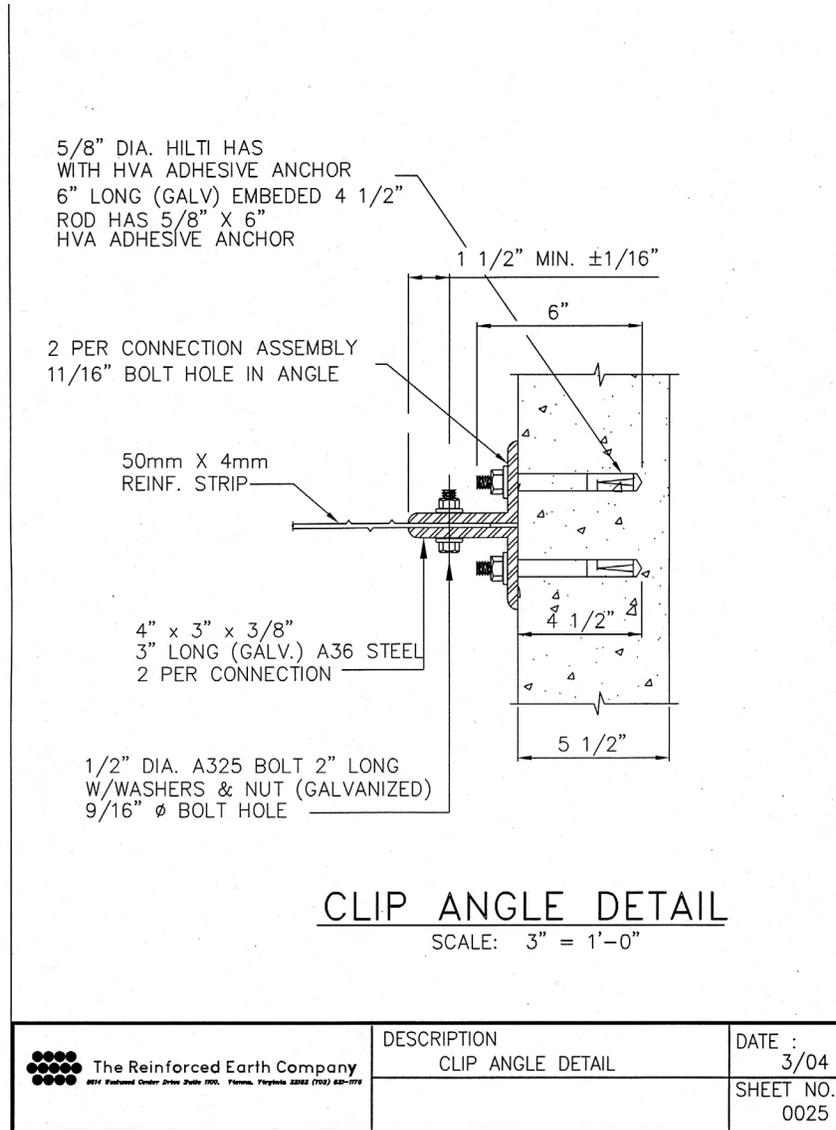


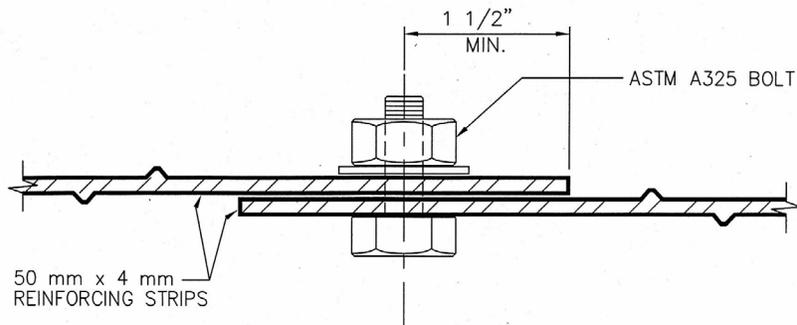












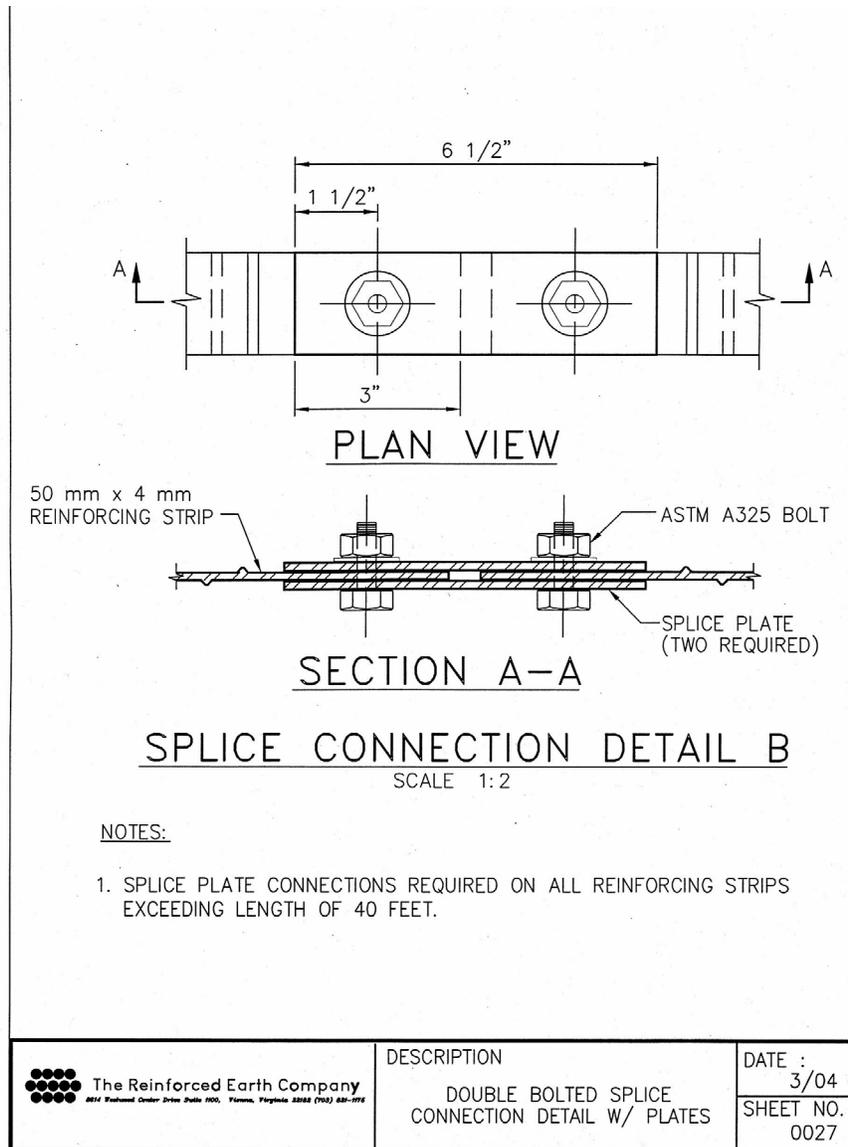
## SPLICE CONNECTION DETAIL A

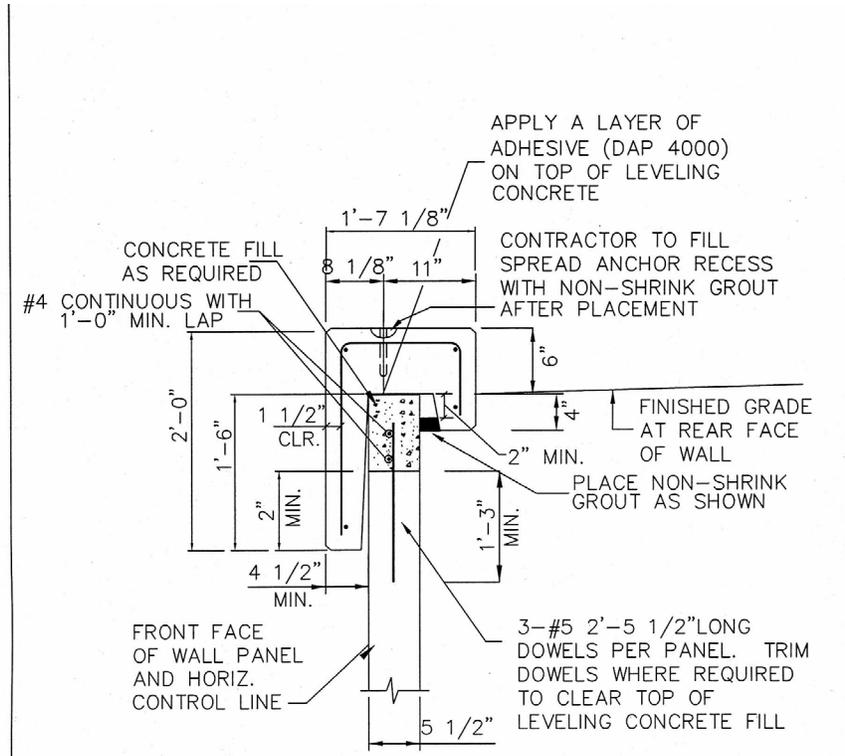
SCALE NTS

NOTES:

1. SPLICE PLATE CONNECTIONS REQUIRED ON ALL REINFORCING STRIPS BETWEEN LENGTH OF 32 FEET AND 40 FEET .

 <p>The Reinforced Earth Company <small>2514 Richmond Center Drive Suite 100, Fairfax, Virginia 22031 (703) 621-2778</small></p>	<p>DESCRIPTION</p> <p>SINGLE BOLT SPLICE CONNECTION DETAIL</p>	<p>DATE :</p> <p style="text-align: center;">3/04</p>
		<p>SHEET NO.</p> <p style="text-align: center;">0026</p>



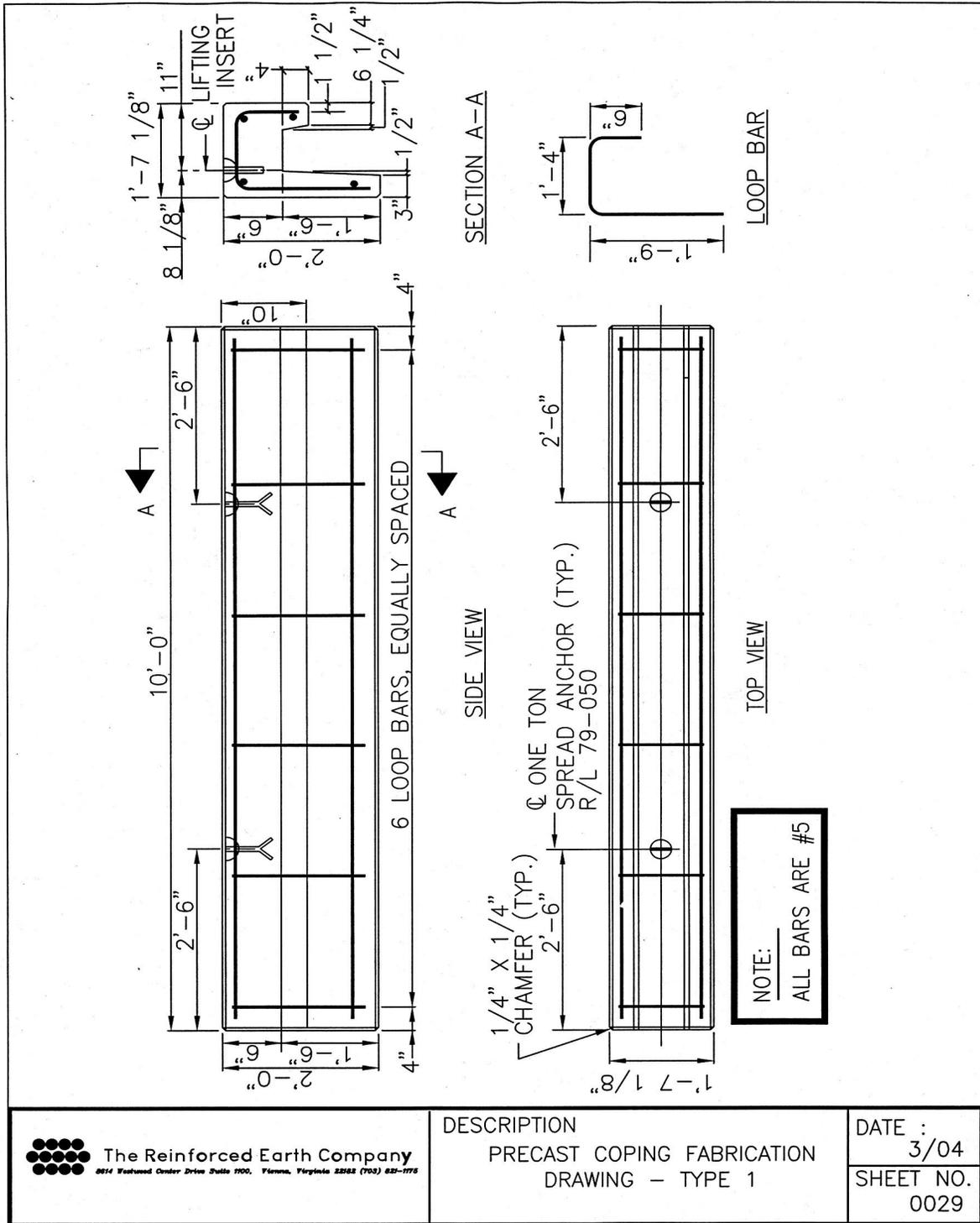


**PRECAST COPING SECTION TYPE 1**

SCALE: NTS

NOTE:  
STANDARD COPING UNIT IS 10-0' LONG.

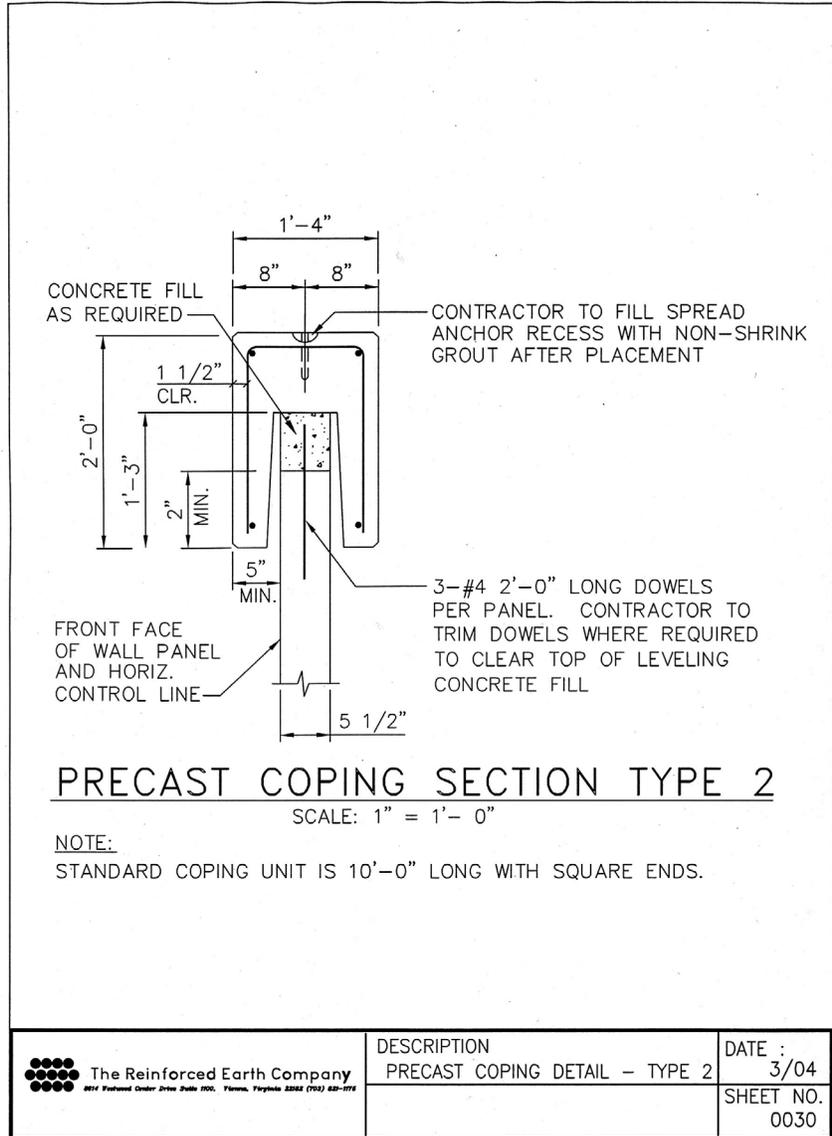
 The Reinforced Earth Company <small>2014 Piedmont Center Drive Suite 100, Yarnon, Virginia 22652 (703) 831-1116</small>	DESCRIPTION	DATE :
	PRECAST COPING DETAIL - TYPE 1	3/04
		SHEET NO.
		0028

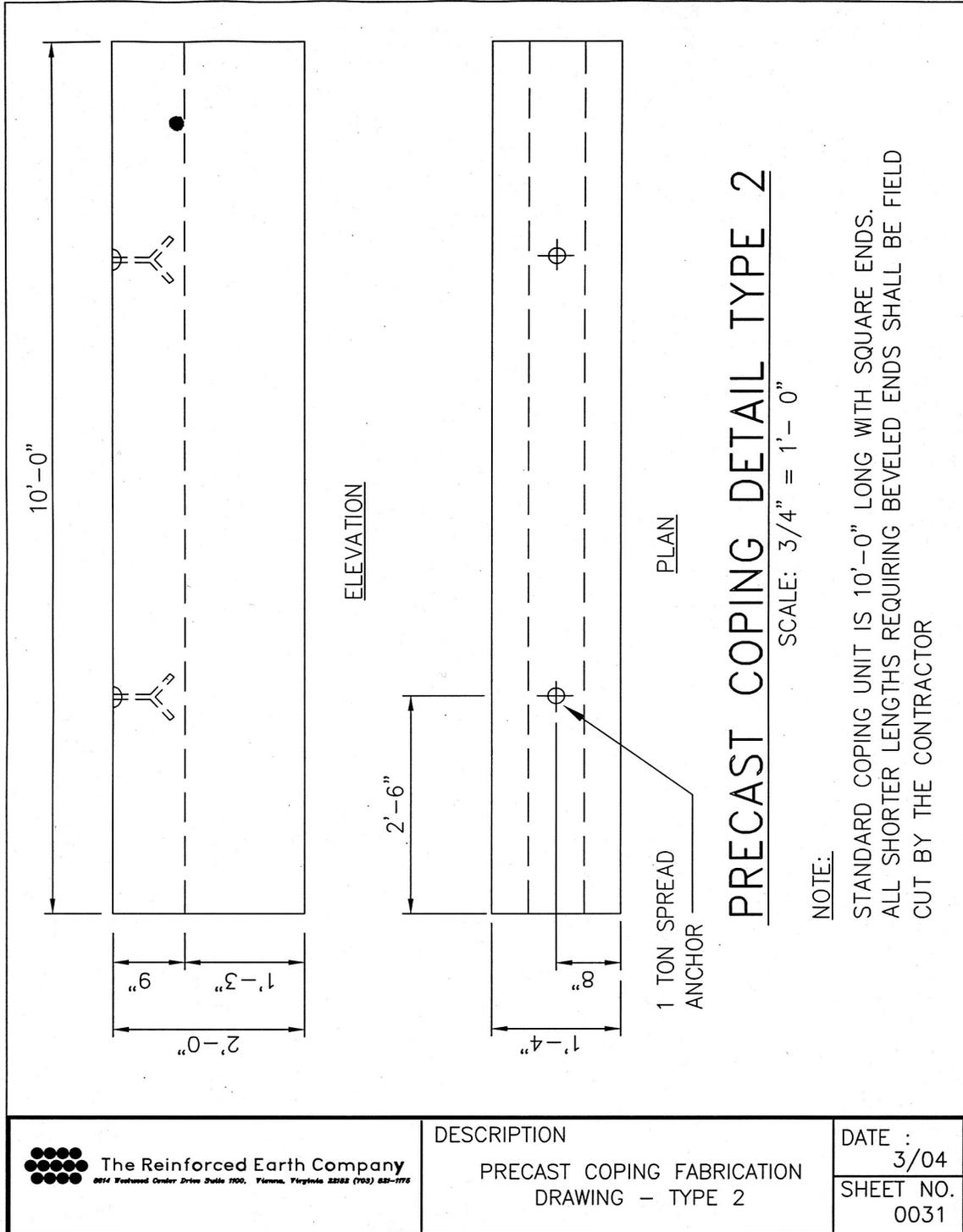


 The Reinforced Earth Company  
 8514 Parkwood Center Drive Suite 1700, Vienna, Virginia 22182 (703) 825-1775

DESCRIPTION  
 PRECAST COPING FABRICATION  
 DRAWING - TYPE 1

DATE : 3/04  
 SHEET NO. 0029

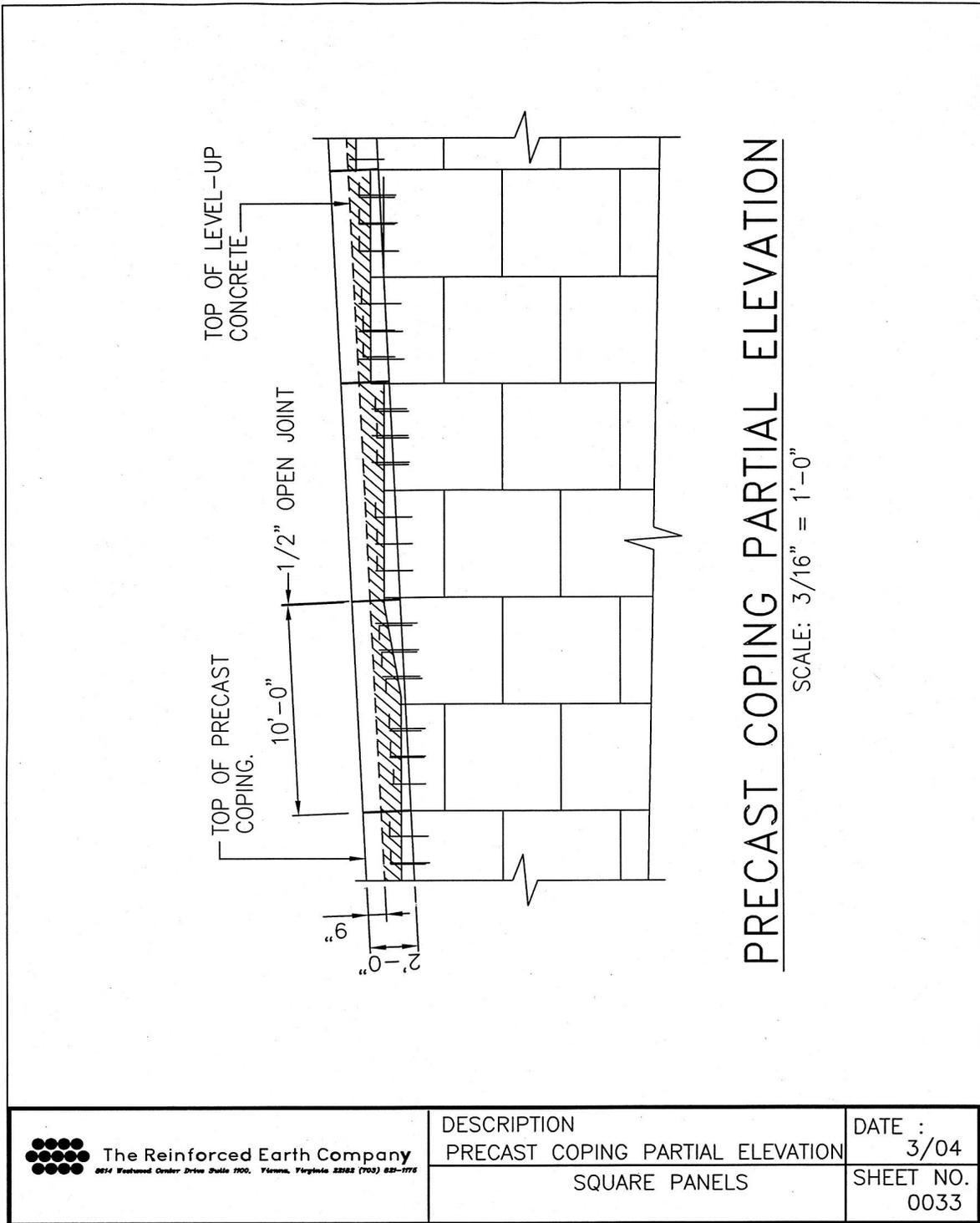


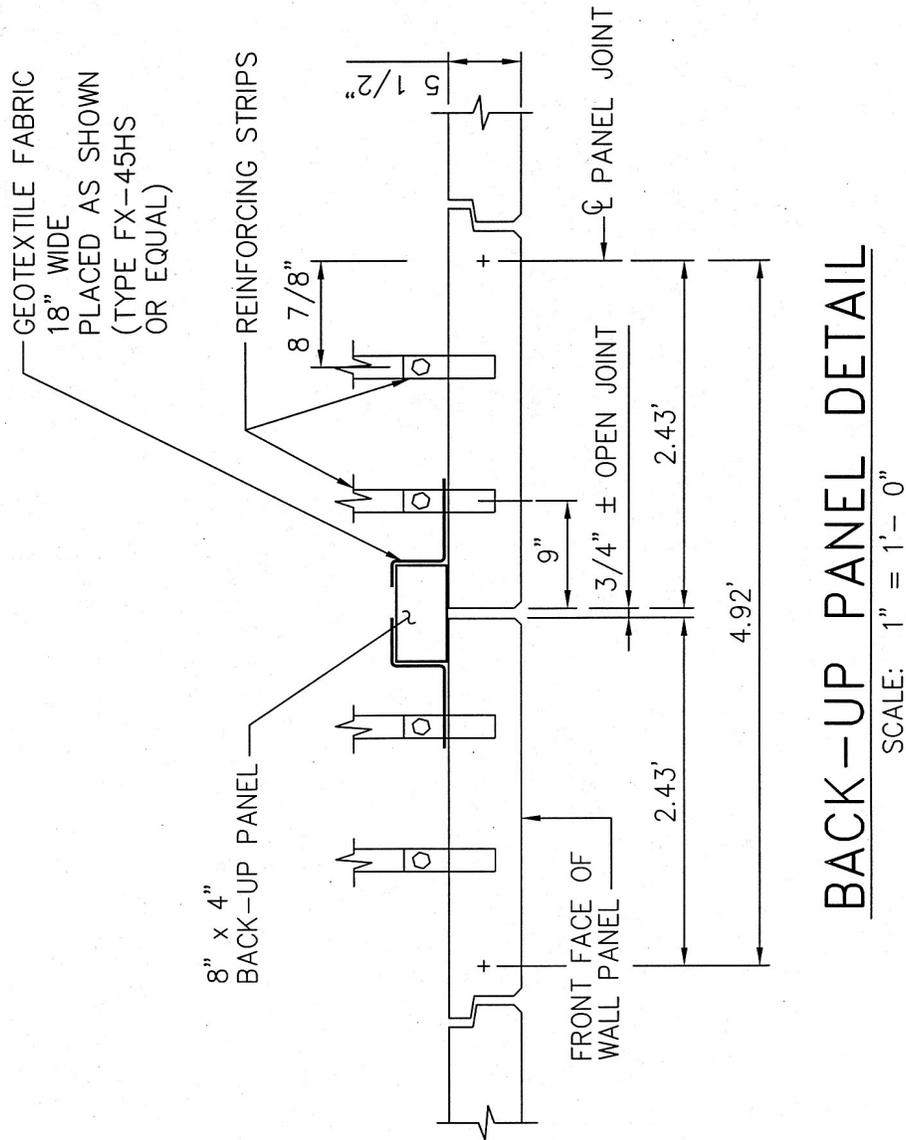


 The Reinforced Earth Company  
 6014 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 823-1776

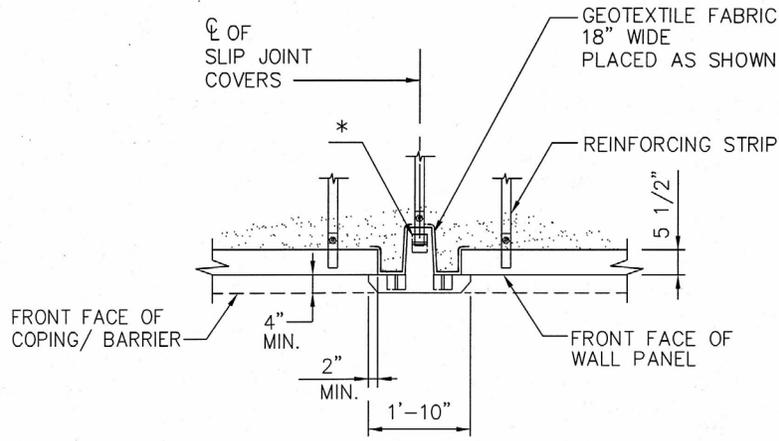
DESCRIPTION  
 PRECAST COPING FABRICATION  
 DRAWING - TYPE 2

DATE : 3/04  
 SHEET NO. 0031





 <b>The Reinforced Earth Company</b> <small>8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22188 (703) 881-1776</small>	DESCRIPTION SLIP JOINT DETAIL W/ BACK-UP PANELS	DATE : 3/04
		SHEET NO. 0017

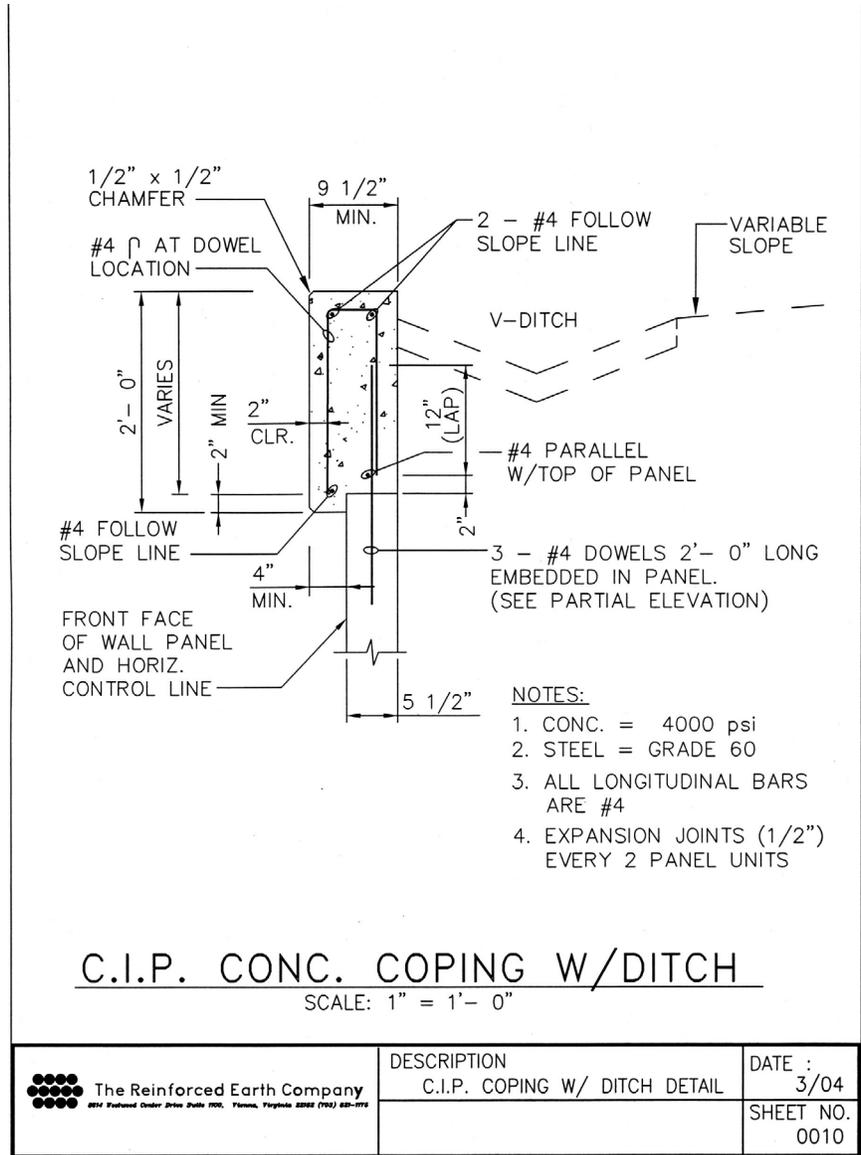


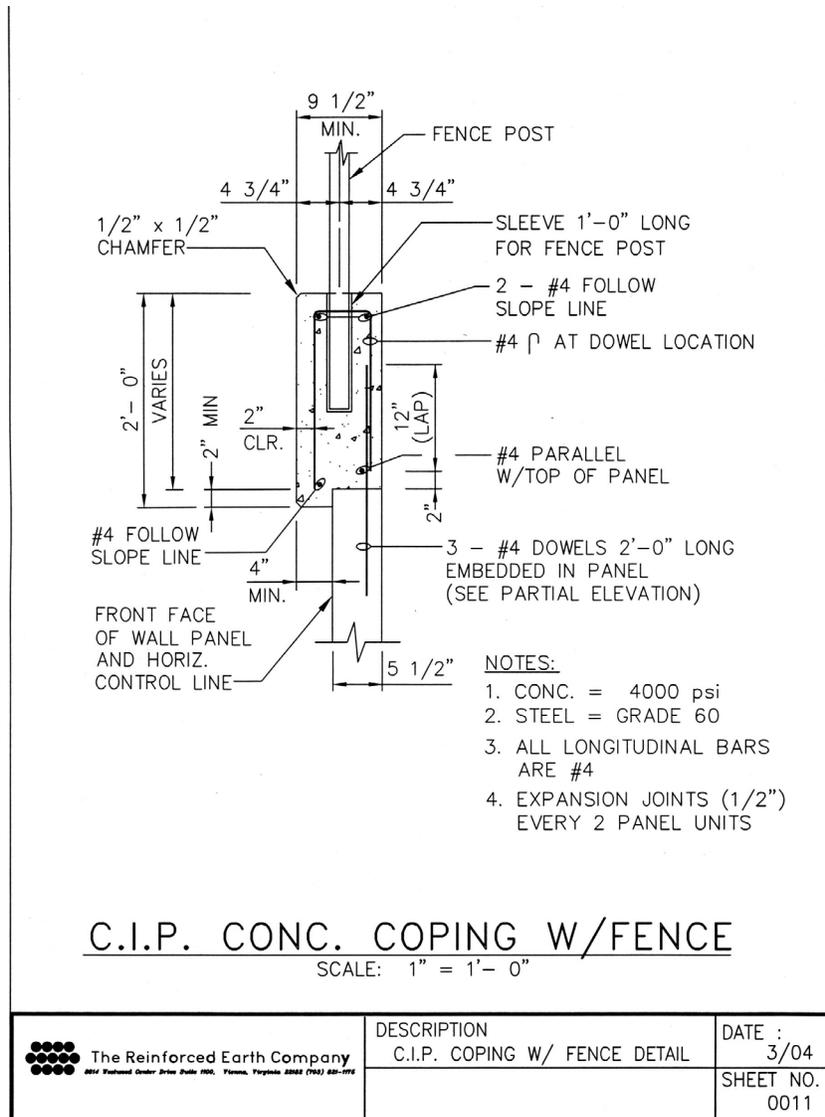
### SLIP JOINT COVER DETAIL

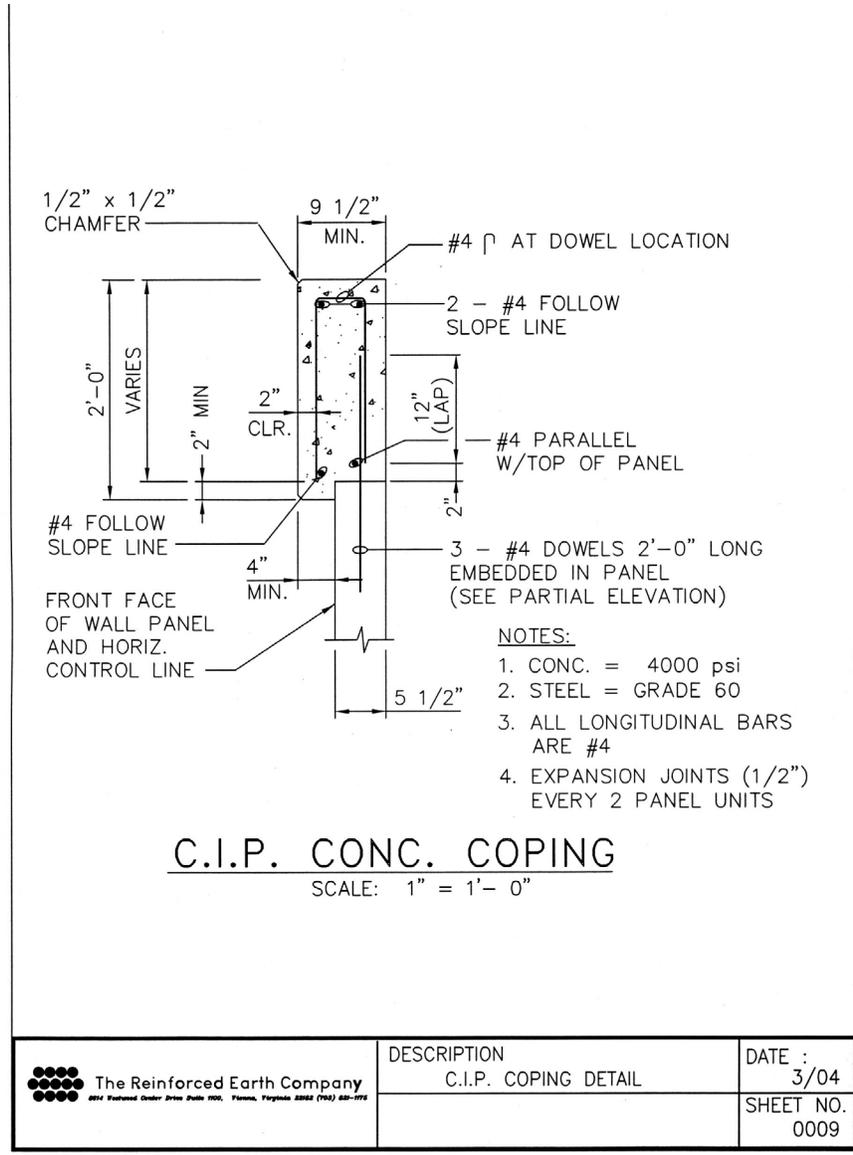
SCALE: 1/2" = 1'- 0"

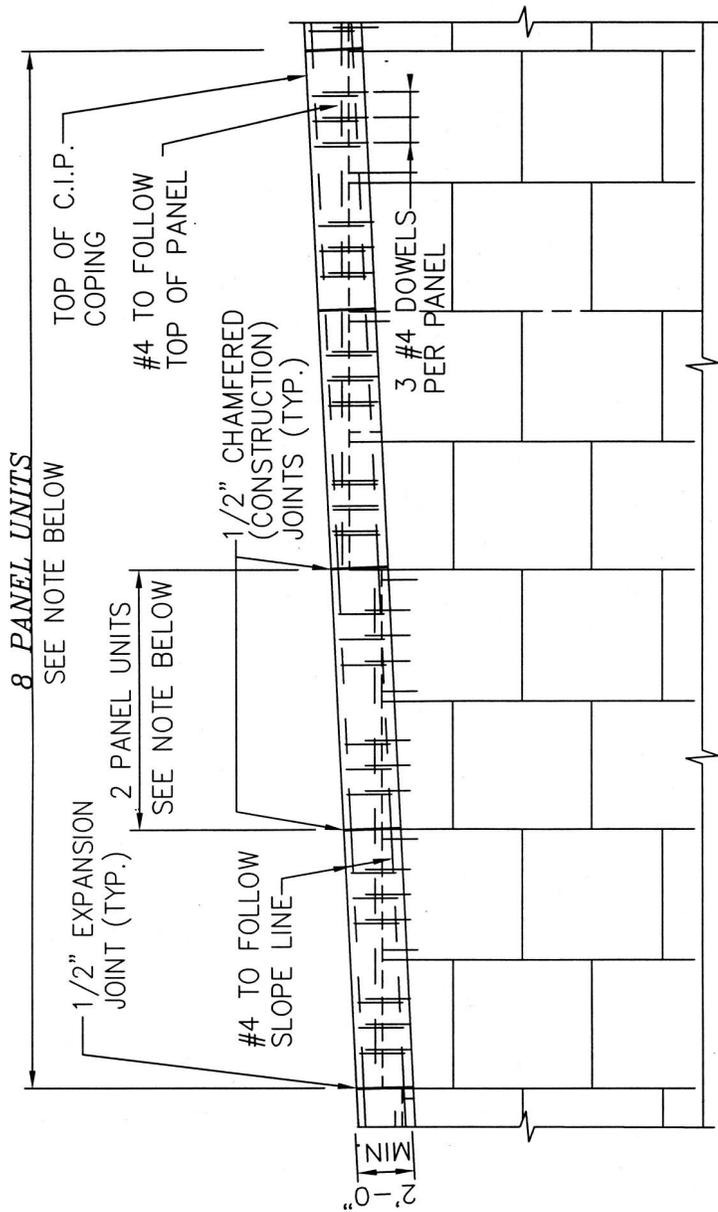
\* THREE BEARING PADS PER UNIT, BASE STEM OF BEARING PAD SHALL BE FIELD CUT TO FIT FLAT ON TOP OF CORNER ELEMENT. FRONT PADS SHALL BE PLACED ON INSIDE EDGE OF LIP.

 The Reinforced Earth Company <small>2614 Fairwood Center Drive Suite 1000, Vienna, Virginia 22182 (703) 826-1772</small>	DESCRIPTION	DATE :
	SLIP JOINT COVER DETAIL	3/04
		SHEET NO.
		0018









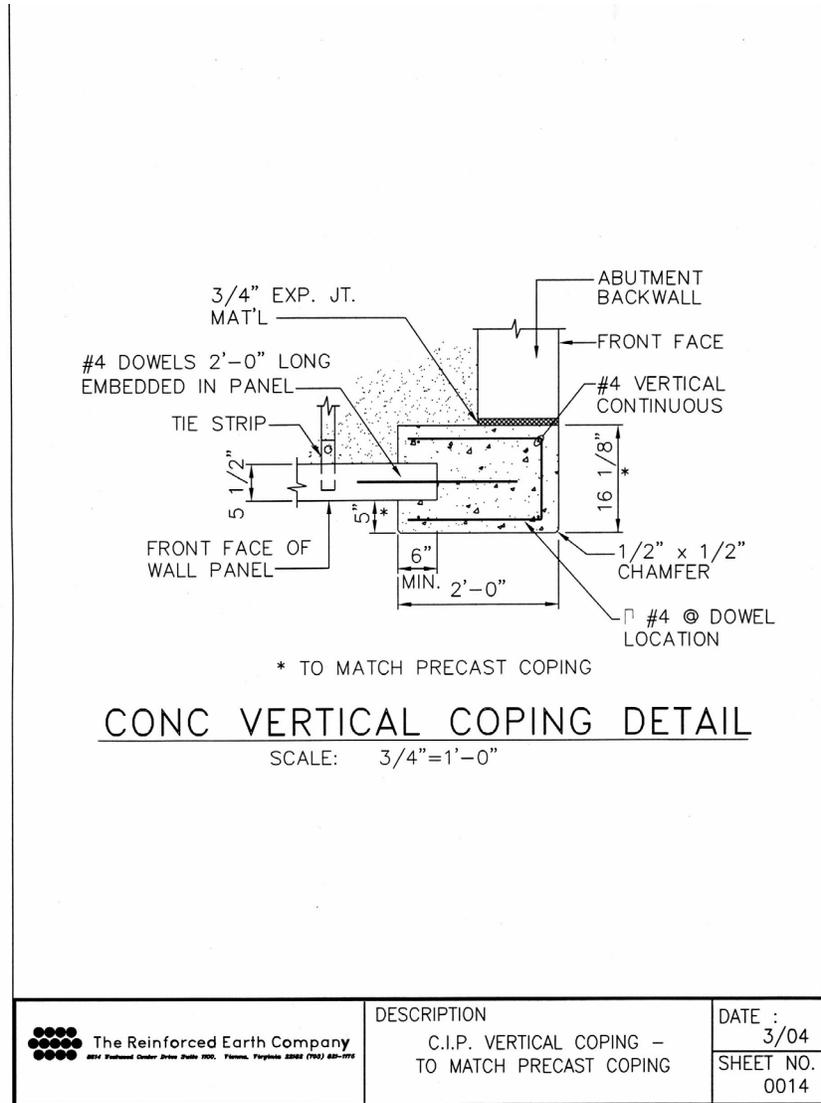
### C.I.P. COPING - PARTIAL ELEVATION

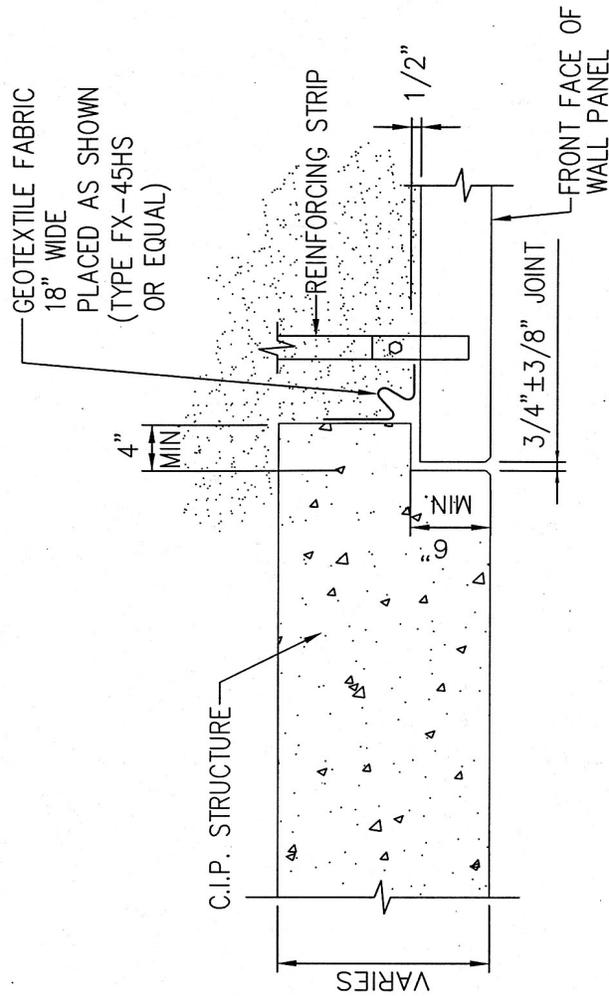
SCALE: 3/16" = 1' - 0"

**NOTE:**

ONE-HALF INCH CHAMFERED (CONSTRUCTION) JOINTS SHOULD BE PLACED AT EVERY TWO-PANEL INTERVAL COINCIDING WITH EVERY OTHER  $\phi$  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.

 <p>The Reinforced Earth Company  <small>8614 Westwood Center Drive Suite 1100, Fairfax, Virginia 22032 (703) 627-1775</small></p>	DESCRIPTION	DATE :
	C.I.P. COPING PARTIAL ELEVATION	3/04
	SQUARE PANELS	SHEET NO.
		0013

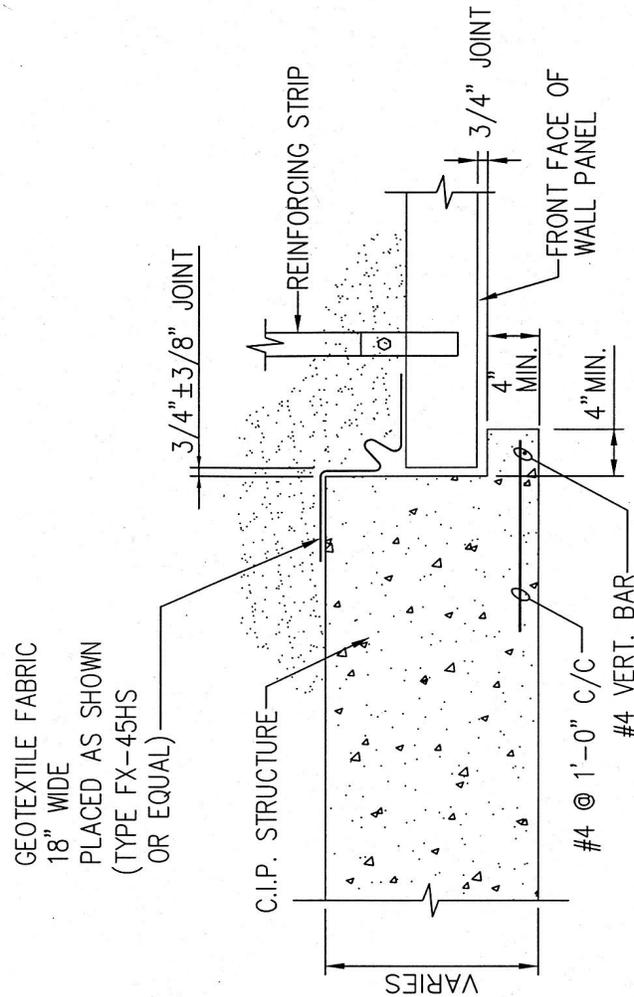




CONNECTION DETAIL @ C.I.P. STRUCTURE

SCALE: 1" = 1'-0"

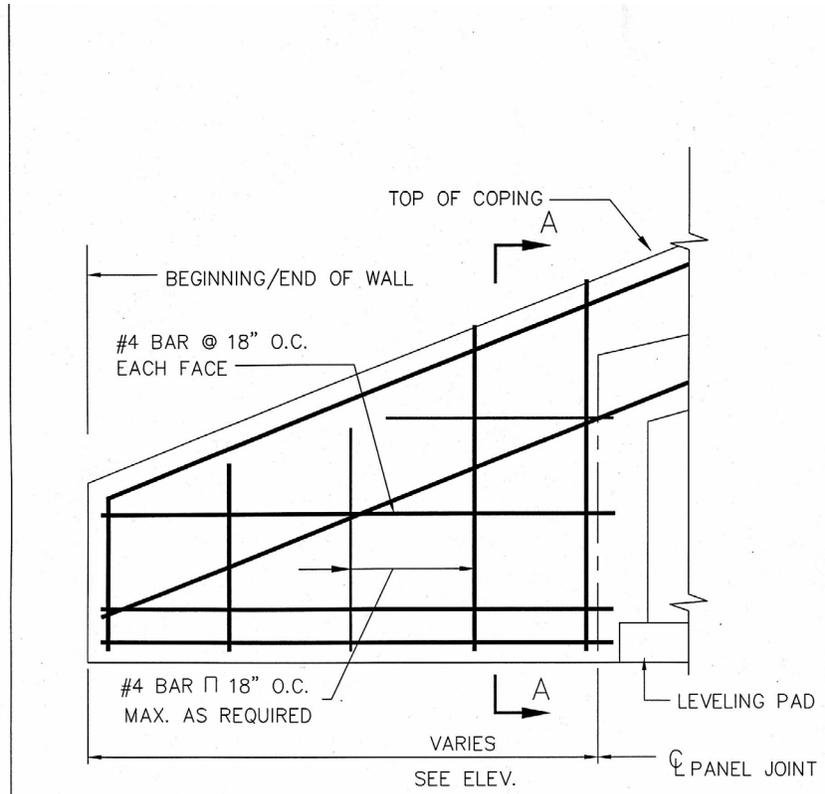
 <p>The Reinforced Earth Company  <small>2614 Westwood Center Drive Suite 1100, Farmington, Virginia 22628 (703) 621-1774</small></p>	DESCRIPTION	DATE :
	CONN. DETAIL @ C.I.P. STRUCTURE TYPE 1 - 4" LIP BEHIND PANEL	3/04



**CONNECTION DETAIL @ C.I.P. STRUCTURE**

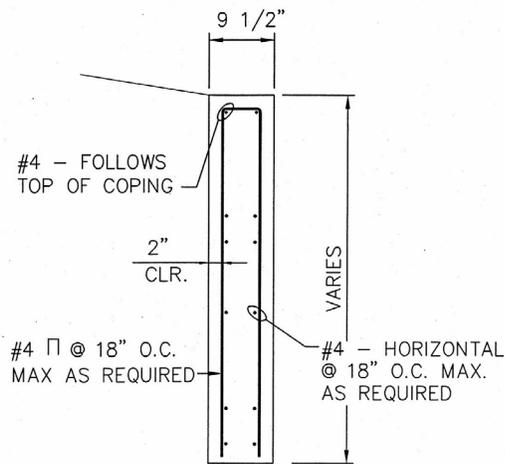
SCALE: 1" = 1'-0"

 <p>The Reinforced Earth Company  <small>2014 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1176</small></p>	DESCRIPTION	DATE :
	CONN. DETAIL @ C.I.P. STRUCTURE TYPE 2 - 4" LIP IN FRONT OF PANEL	3/04



**COPING ENCLOSURE DETAIL**  
 SCALE: 3/4" = 1'- 0"

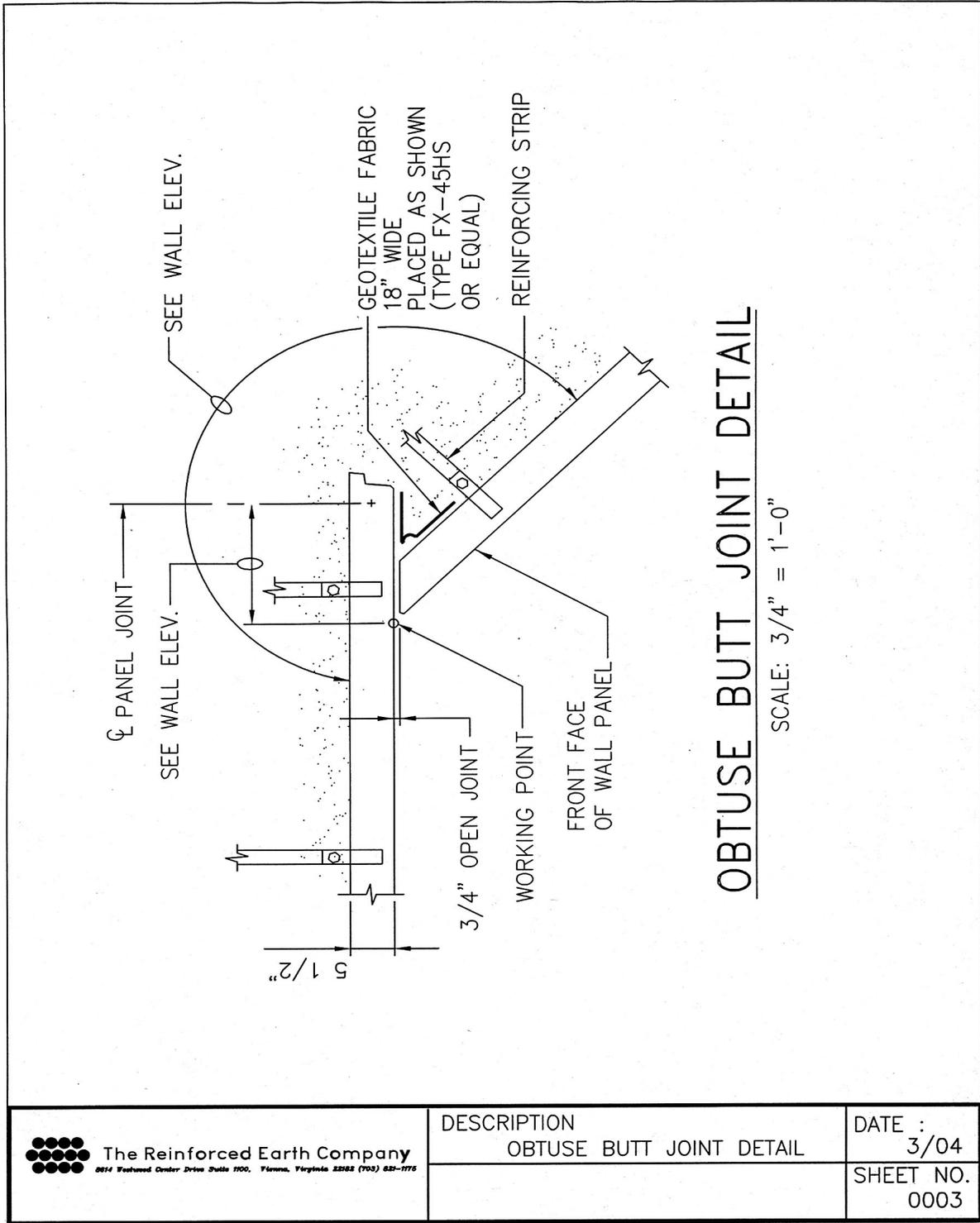
 The Reinforced Earth Company <small>RECO Portland Cement Concrete Block PCC, Tampa, Florida 33622 (703) 882-8772</small>	DESCRIPTION	DATE :
	COPING ENCLOSURE DETAIL	3/04
	PARTIAL ELEVATION	SHEET NO.
		0016A



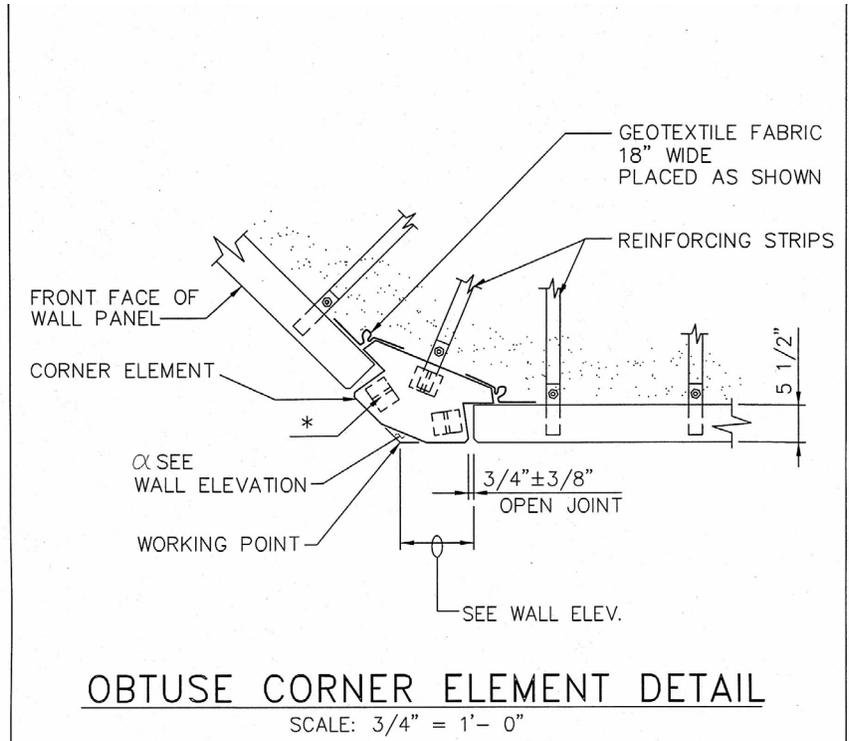
**SECTION A-A**

SCALE: 3/4" = 1'- 0"

 The Reinforced Earth Company <small>RECO Reinforced Concrete Dryer State PCC, Tampa, Florida 33612 (813) 821-2711</small>	DESCRIPTION	DATE :
	COPING ENCLOSURE DETAIL	3/04
	SECTION A-A	SHEET NO.
		0016B

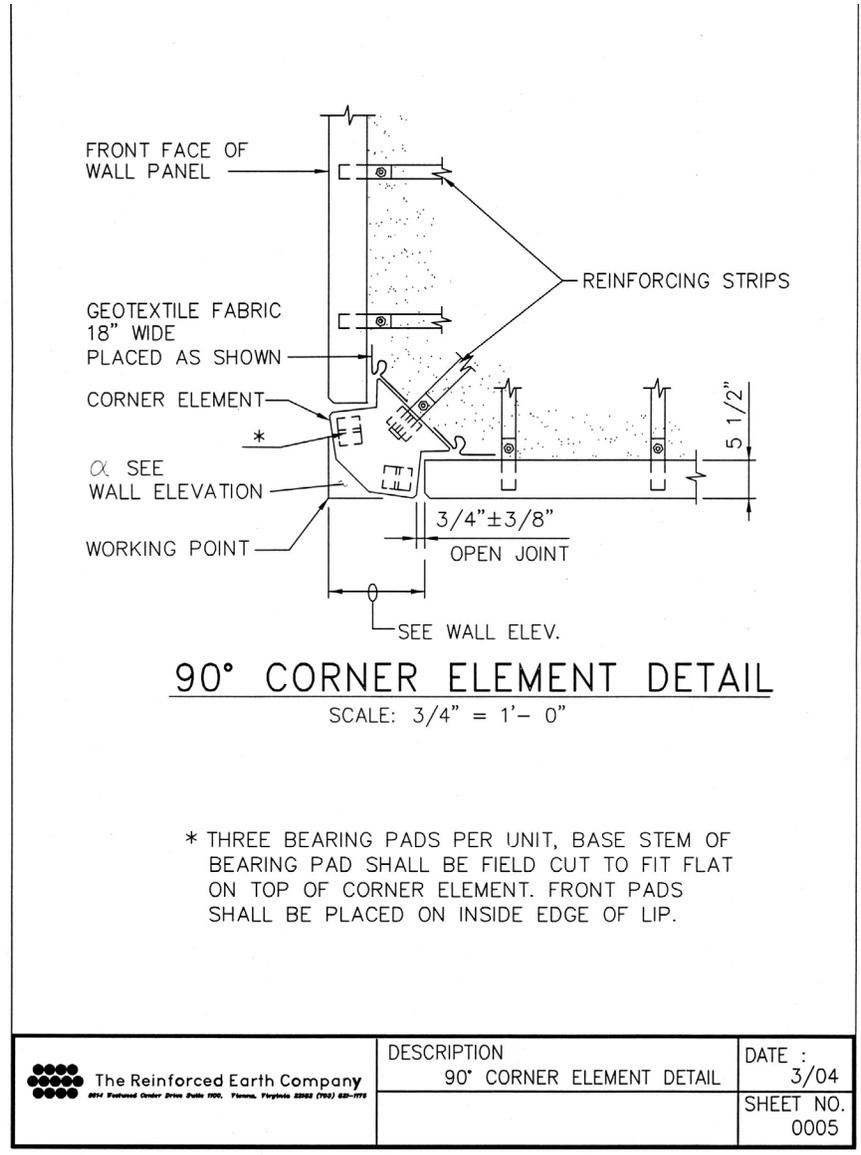


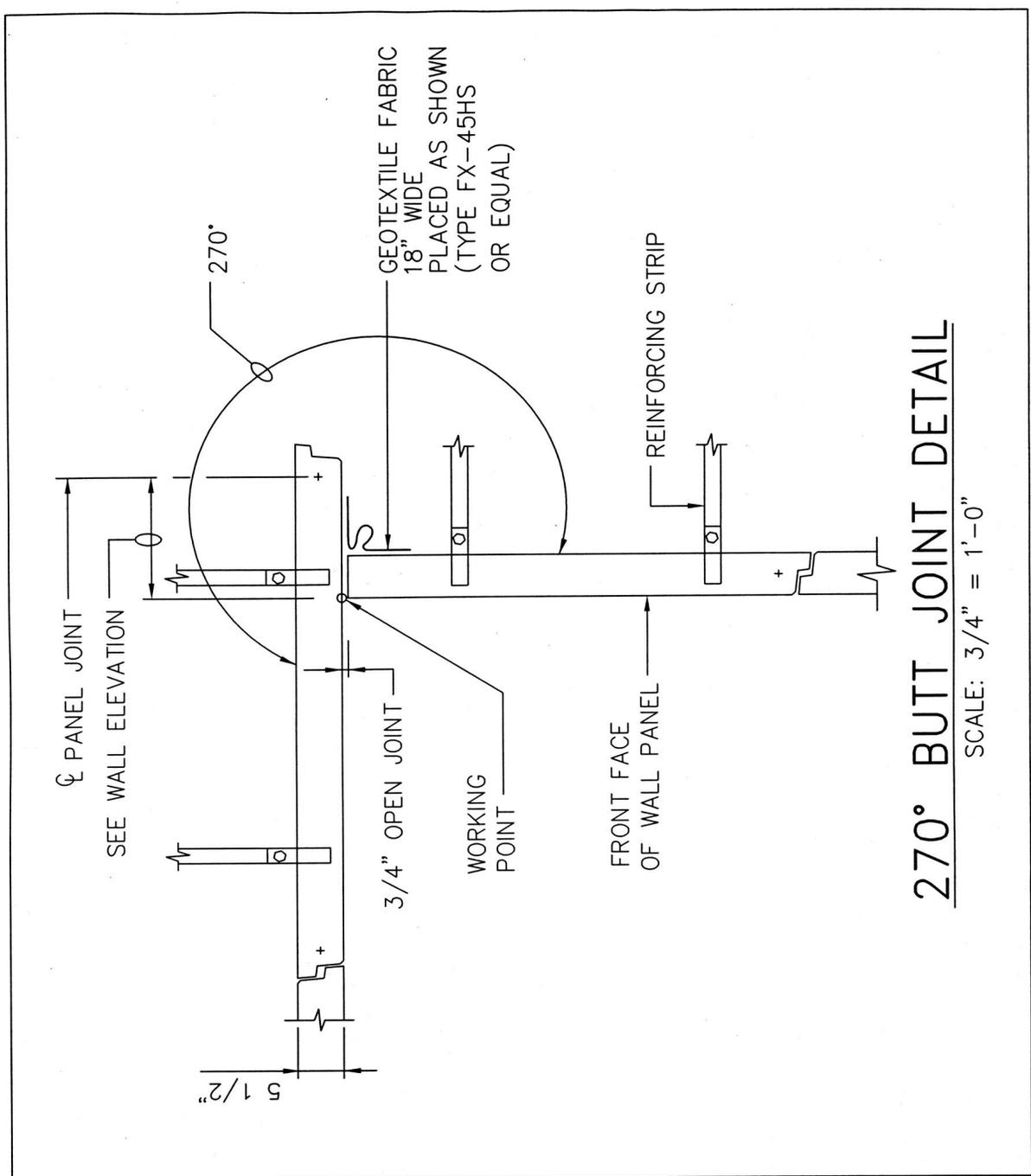
 <p>The Reinforced Earth Company  <small>2014 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 842-1776</small></p>	DESCRIPTION	DATE :
	OBTUSE BUTT JOINT DETAIL	3/04
		SHEET NO.
		0003



\* 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.

 <b>The Reinforced Earth Company</b> <small>8514 Redwood Center Drive Suite 700, Yuma, Virginia 22688 (703) 821-7776</small>	DESCRIPTION	DATE :
	OBTUSE CORNER ELEMENT DETAIL	3/04
		SHEET NO.
		0006

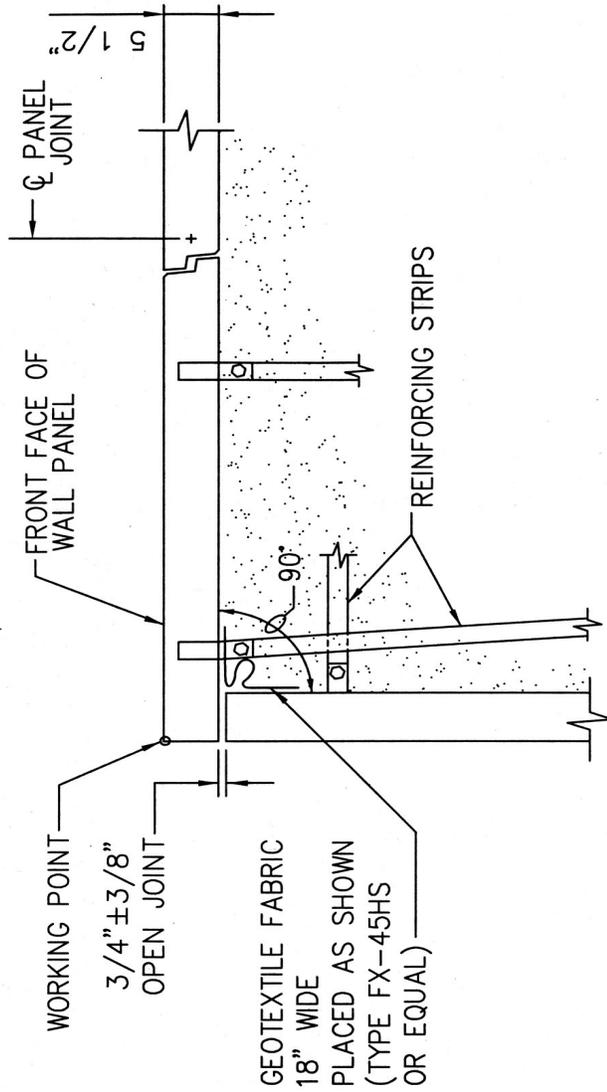




**270° BUTT JOINT DETAIL**

SCALE: 3/4" = 1'-0"

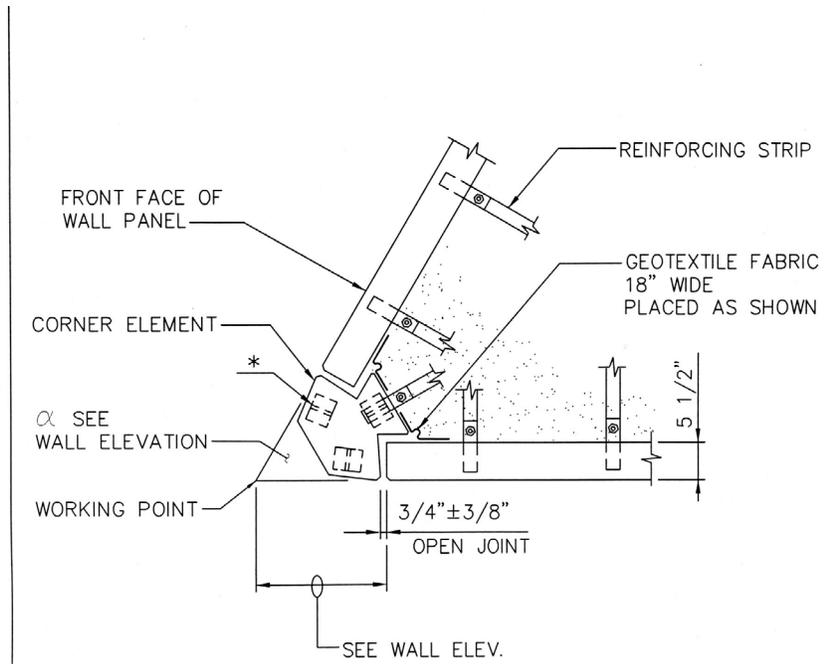
 <p>The Reinforced Earth Company  <small>2614 Foothold Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1775</small></p>	DESCRIPTION	DATE :
	270° BUTT JOINT DETAIL	3/04
		SHEET NO.
		0002



# 90° BUTT JOINT DETAIL

SCALE: 3/4" = 1'-0"

 <p>The Reinforced Earth Company  <small>2014 Professional Center Drive Suite 1000, Vienna, Virginia 22182 (703) 822-1772</small></p>	DESCRIPTION	DATE :
	90° BUTT JOINT DETAIL	3/04
		SHEET NO. 0001

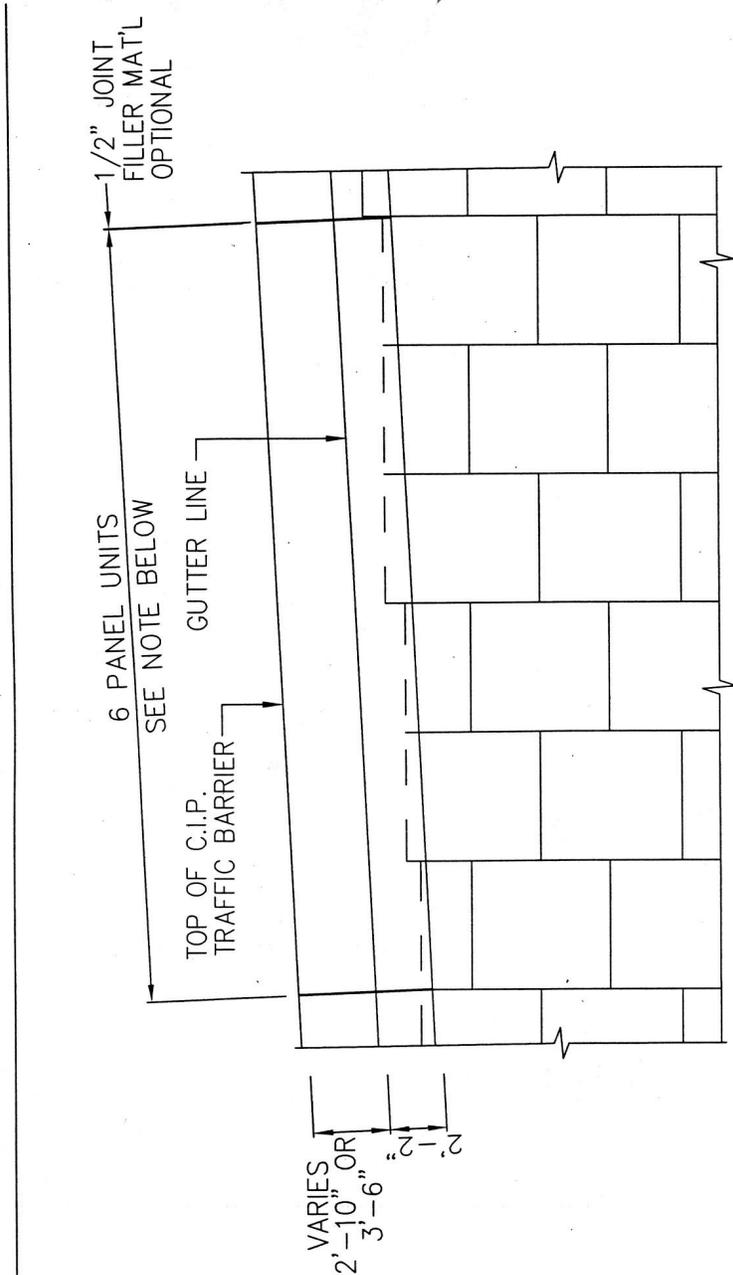


### ACUTE CORNER ELEMENT DETAIL

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 <small>2014 Redwood Center Drive Suite 100, Yarnes, Virginia 22091 (703) 881-1111</small>	DESCRIPTION	DATE :
	ACUTE CORNER ELEMENT DETAIL	3/04
		SHEET NO.
		0004

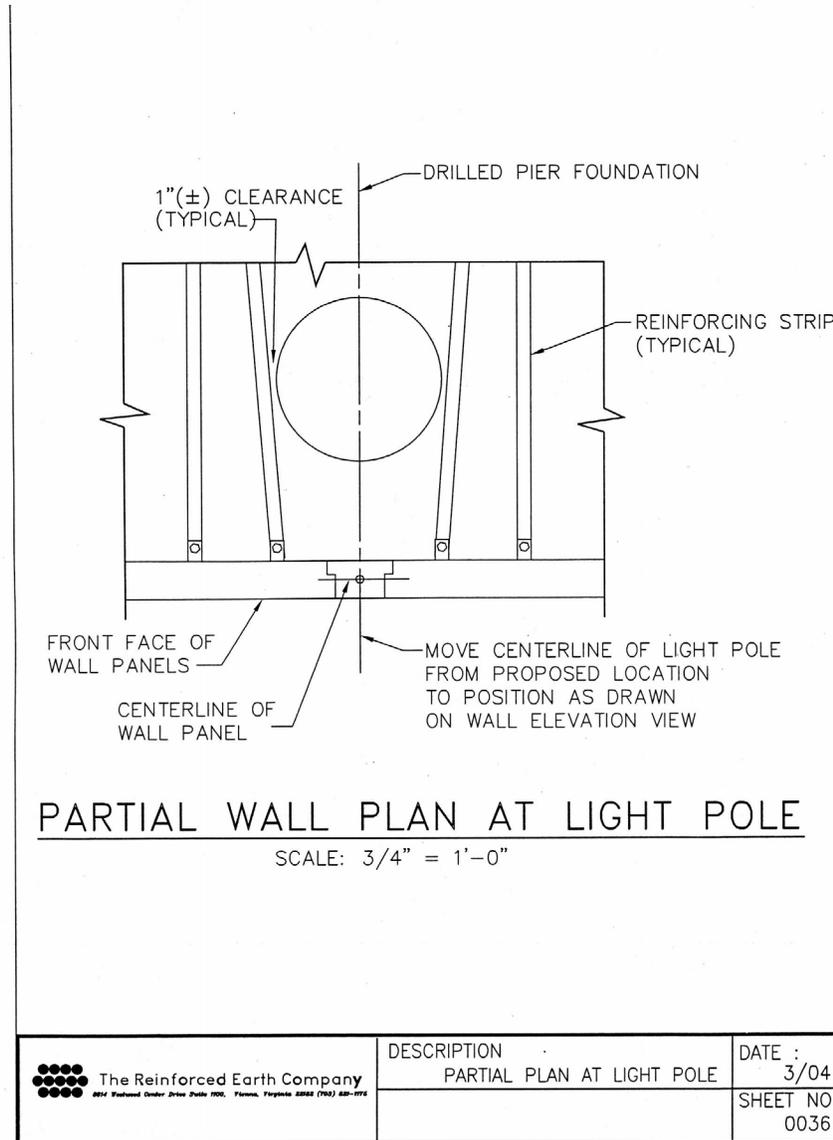


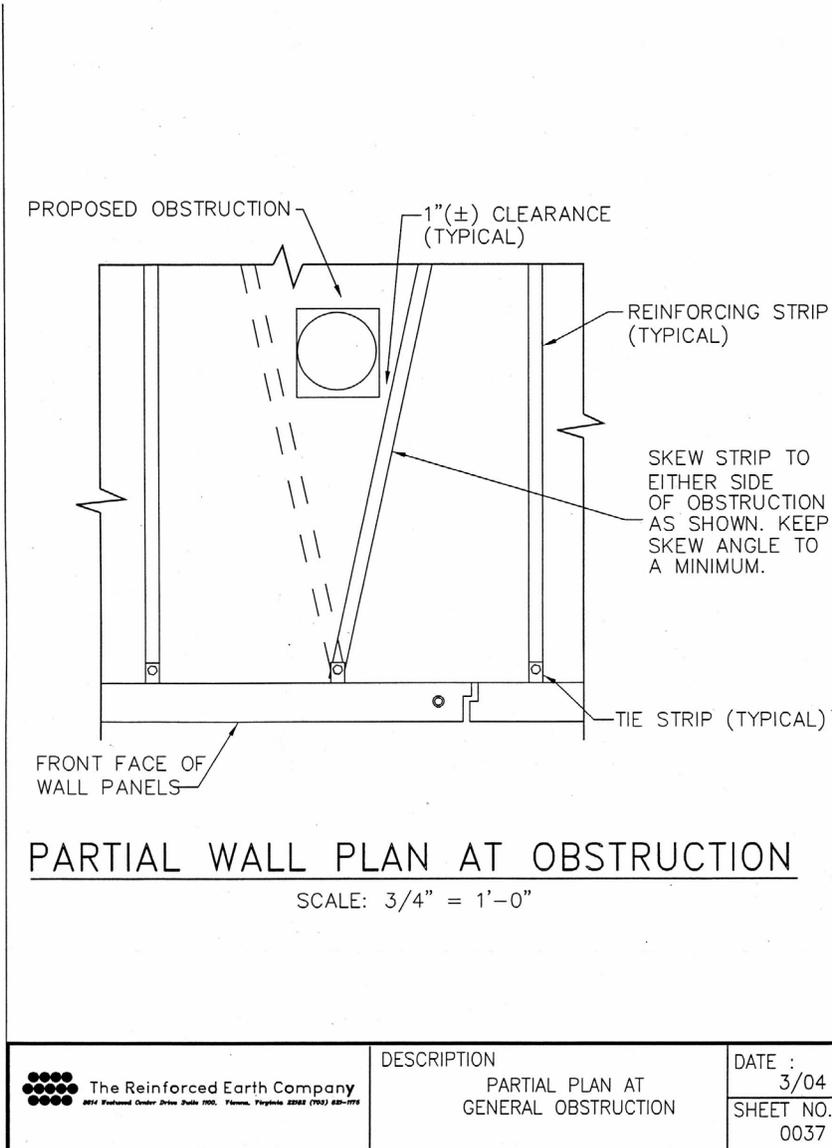
NOTE:  
JOINTS IN PAVEMENT OR JUNCTION SLAB SHALL COINCIDE WITH JOINTS IN BARRIER.

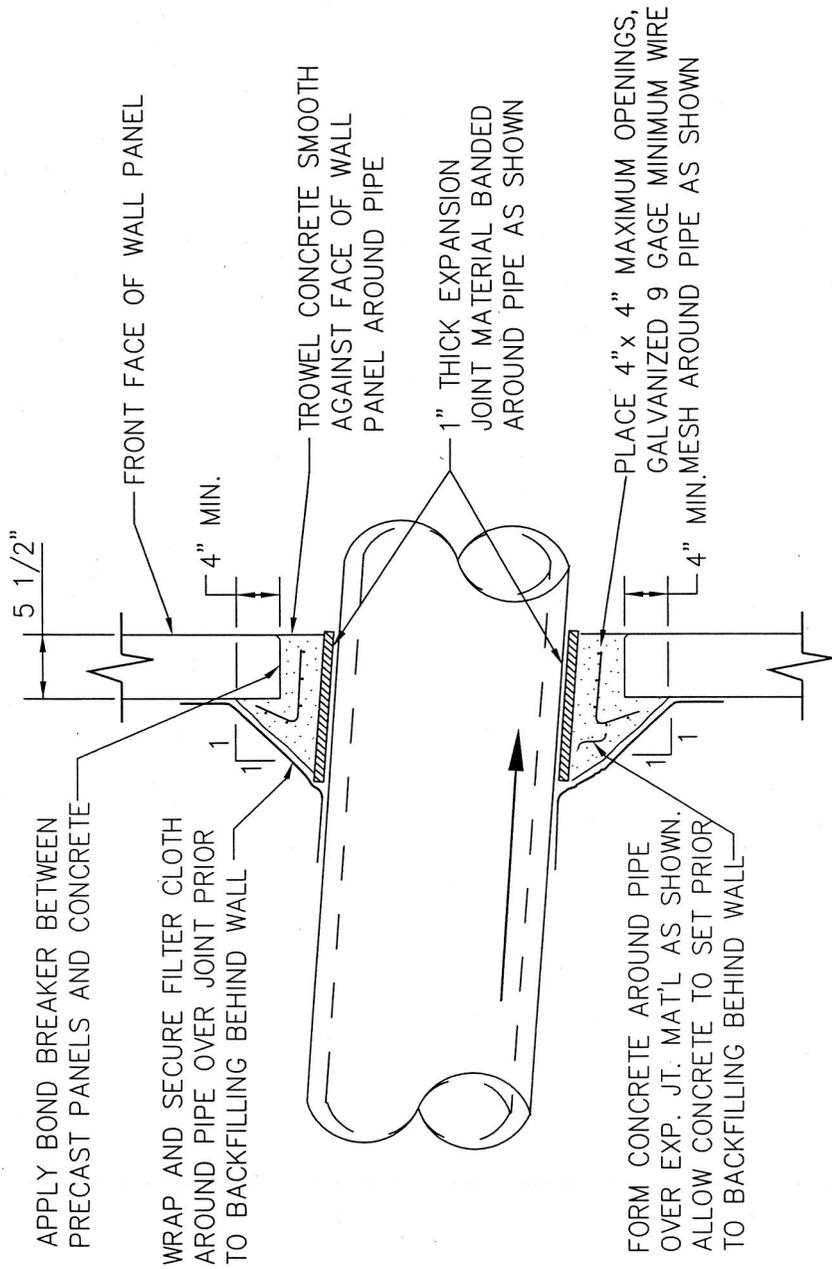
C.I.P. TRAFFIC BARRIER  
PARTIAL ELEVATION

SCALE: 3/16" = 1'-0"

 <p>The Reinforced Earth Company 8814 Feathered Center Drive Suite 1700, Vienna, Virginia 22182 (703) 851-1776</p>	DESCRIPTION C.I.P. BARRIER PARTIAL ELEVATION SQUARE PANELS	DATE : 3/04
		SHEET NO. 0035





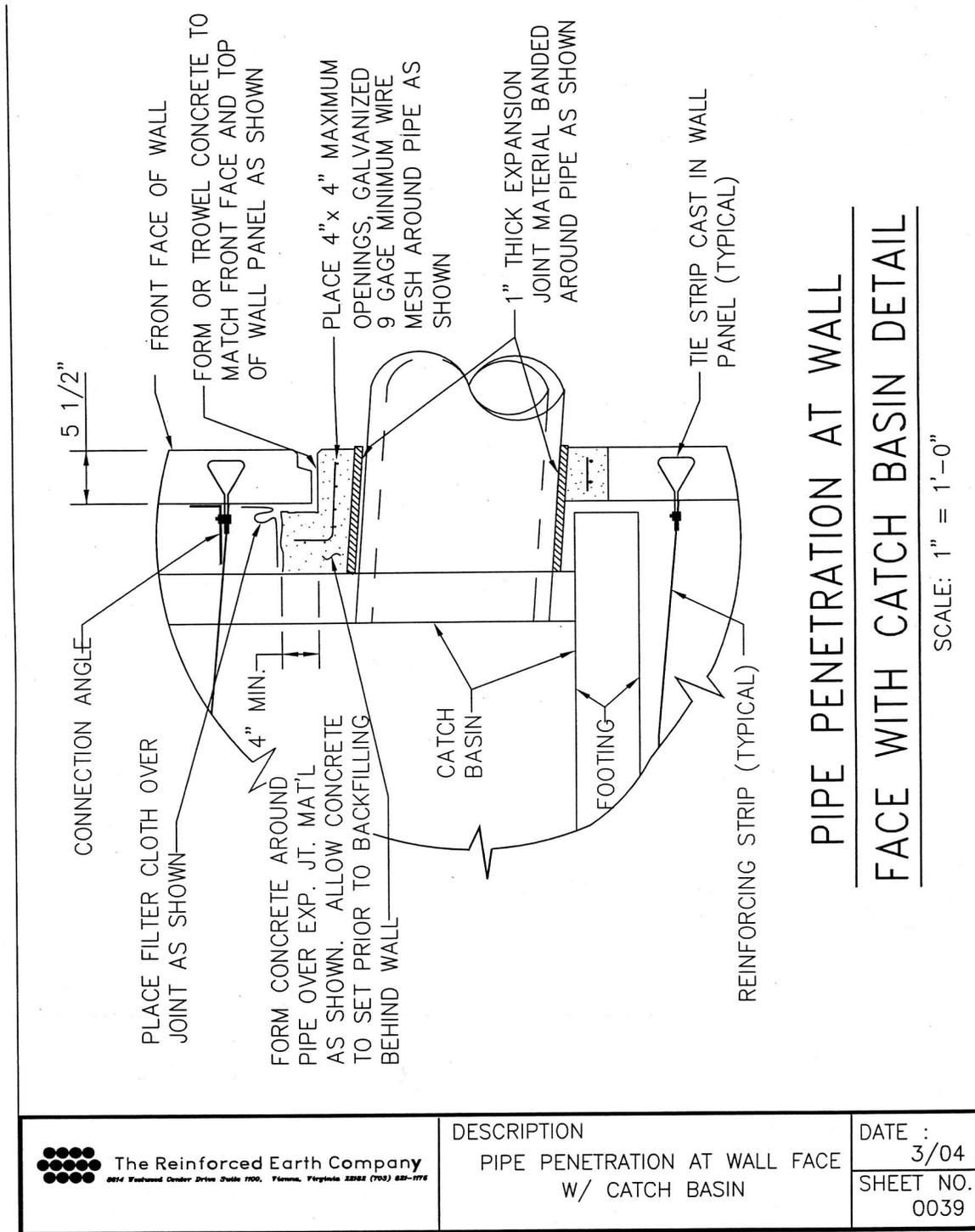


### PIPE PENETRATION AT WALL FACE DETAIL

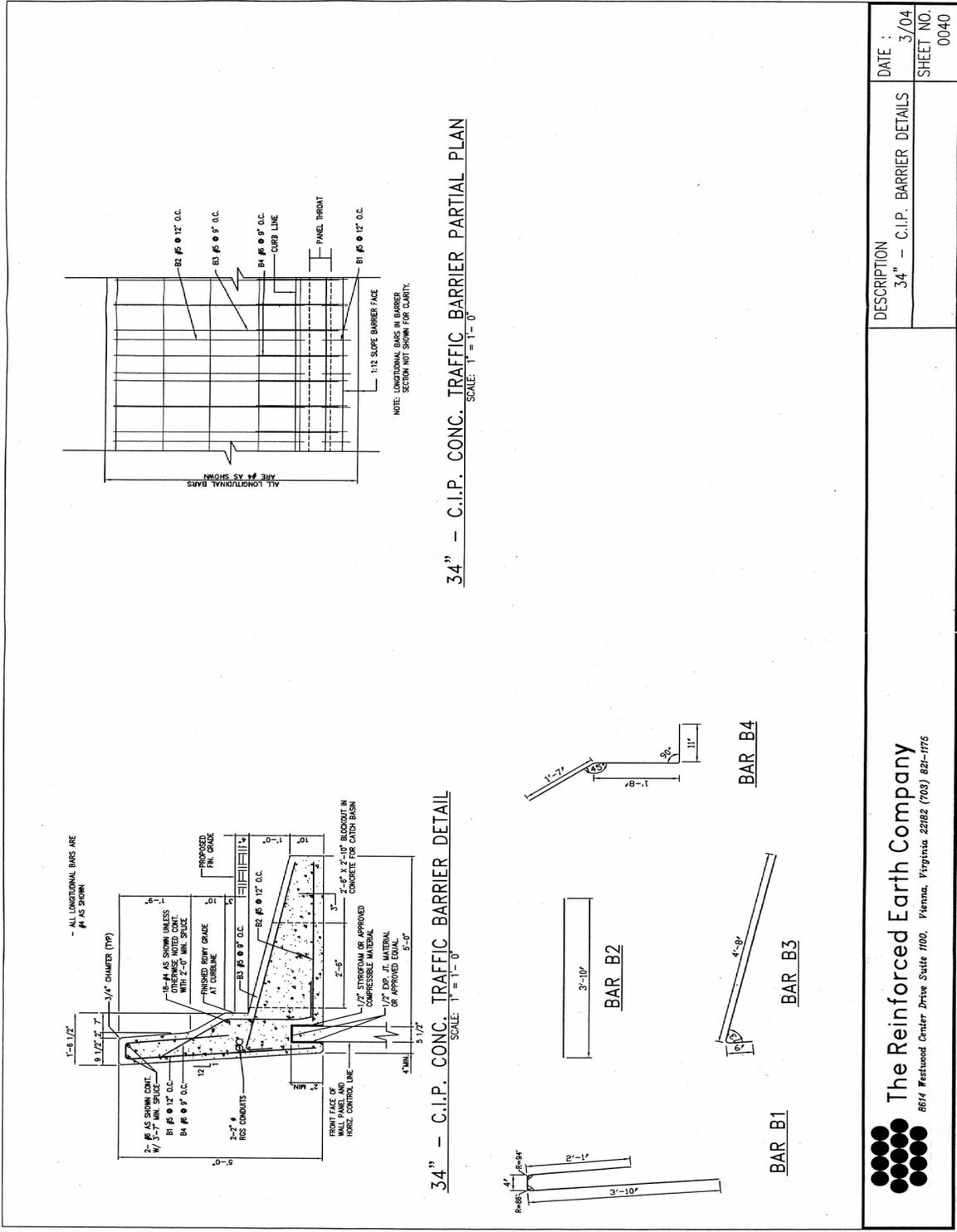
(TREATMENT FOR CONCRETE PIPE SHOWN, CORRUGATED STEEL PIPE SIMILAR)

SCALE: 1" = 1'-0"

 <p>The Reinforced Earth Company  <small>8814 Westwood Center Drive Suite 700, Vienna, Virginia 22182 (703) 821-7776</small></p>	DESCRIPTION	DATE :
	PIPE PENETRATION AT WALL FACE	3/04
		SHEET NO.
		0038



 <p>The Reinforced Earth Company 2014 Woodrow Center Drive Suite 1100, Vienna, Virginia 22182 (703) 829-1776</p>	DESCRIPTION	DATE :
	PIPE PENETRATION AT WALL FACE W/ CATCH BASIN	3/04
		SHEET NO. 0039



DATE :	3/04
SHEET NO.	0040

DESCRIPTION	34" - C.I.P. BARRIER DETAILS
-------------	------------------------------

**The Reinforced Earth Company**  
 8614 Westwood Center Drive Suite 1100, Vienna, Virginia 22182 (703) 821-1775







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.

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

### Approved Connection Strength Design Values for Tensar Ares Walls

Table 15-M-1

$T_{ac}$ , the long-term connection strength, shall be calculated as follows for the Tensar ARES wall:

$$T_{ac} = \frac{T_{MARV} \cdot CR_u}{RF} \quad (15\text{-(Tensar ARES)-1})$$

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

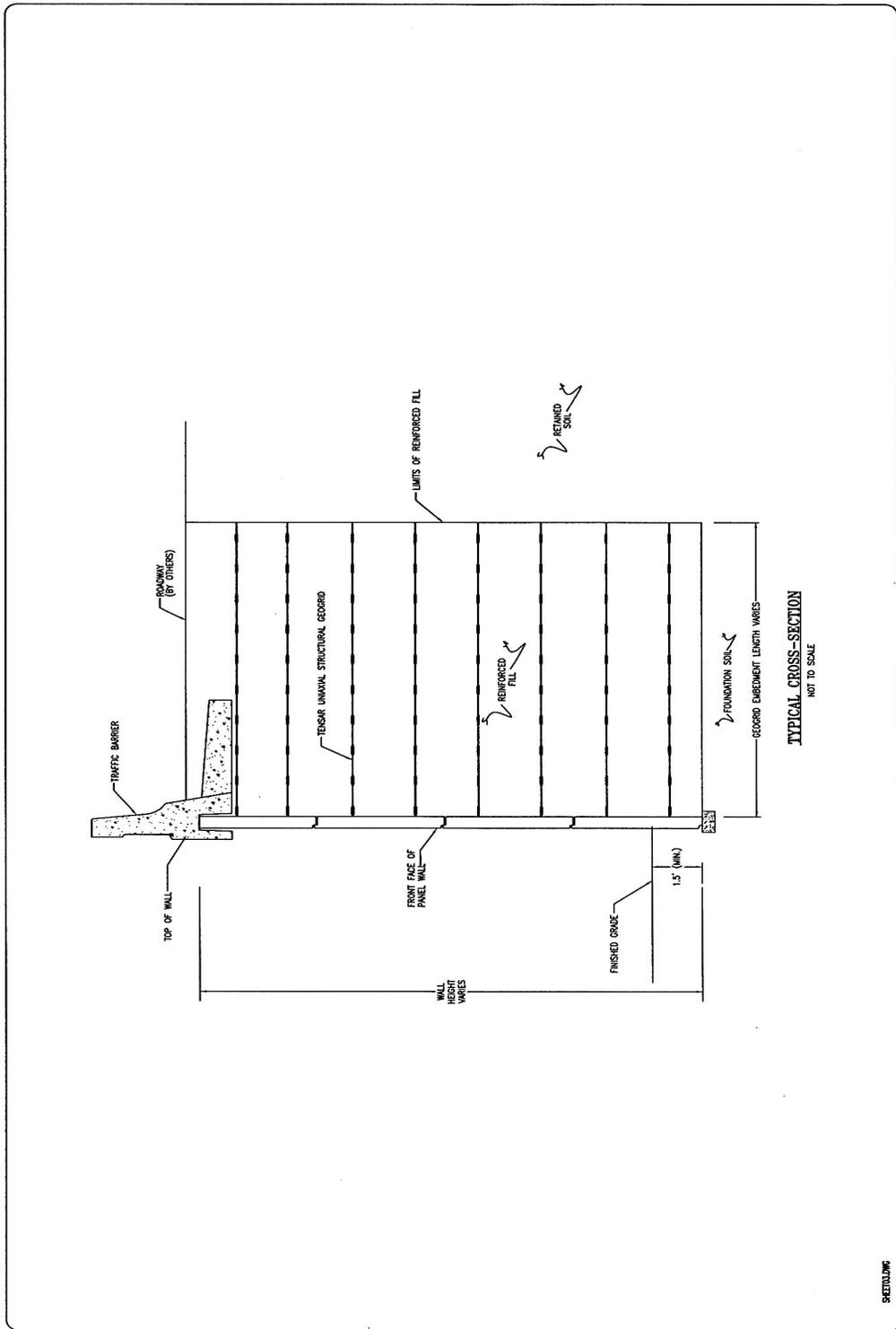
$RF_{CR}$  = Creep reduction factor for the geosynthetic

$RF_D$  = 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 penetrate, 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.





SHEET/DRAWING

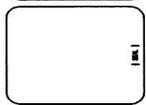
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS

ARES ARTICULATED PANEL CROSS-SECTION

Sheet Number 3 of 19

Project Number	9/0/04
Issue No.	1
Issue Date	04/2005
Drawn by	SD
Checked by	TKK
Quantity	1

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**Tensar**  
 Tensar Earth Technologies, Inc.  
 5835 Remington Drive, Suite 200  
 Alexandria, VA 22304  
 (703) 295-1200

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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

**ALL PANELS ARE SHOWN BACK FACE VIEW**

**REINFORCING STEEL REQUIREMENTS**  
 STANDARD STEEL LAYOUT  
 HORIZONTAL: #4 BARS (60 KSI) @ 1' - 8 3/4" (MAX.)  
 VERTICAL: #3 BARS (60 KSI) @ 10" O.C. (MAX.)  
 STANDARD WME LAYOUT  
 4x4-W/LOW-ALLOY WELDED WIRE MESH  
 FABRICATION PER ASTM A-185

**NOTE:** ALL TOP PANELS WILL HAVE 2 #4 DOWELS  
 CAST 8" INTO THE TOP OF EACH PANEL.  
 GEORGRID TAB LOCATIONS ARE PROJECT SPECIFIC

REVISIONS / ISSUE

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(Rev. 1)

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 Tensar Earth Technologies, Inc.  
 5800 University Drive, Suite 200  
 Morrisville, North Carolina 27560  
 (919) 286-1200

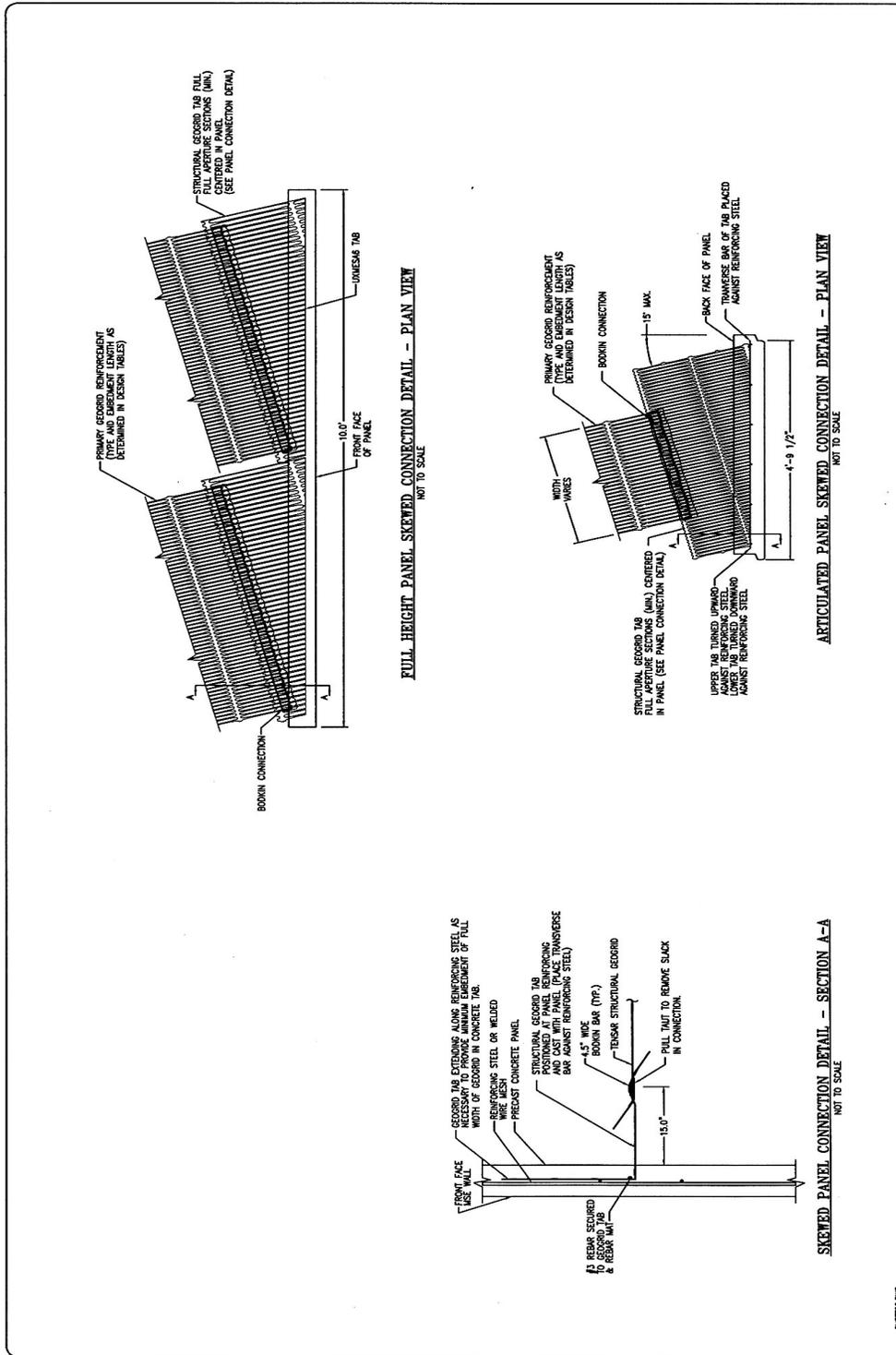
SHEET NO. 15M-03-08-001-001  
 SHEET TOTAL 15M-03-08-001-001

**STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS**

**ARES ARTICULATED PANELS**

Sheet Number  
**5 of 19**





SHEETWORK

**STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS**

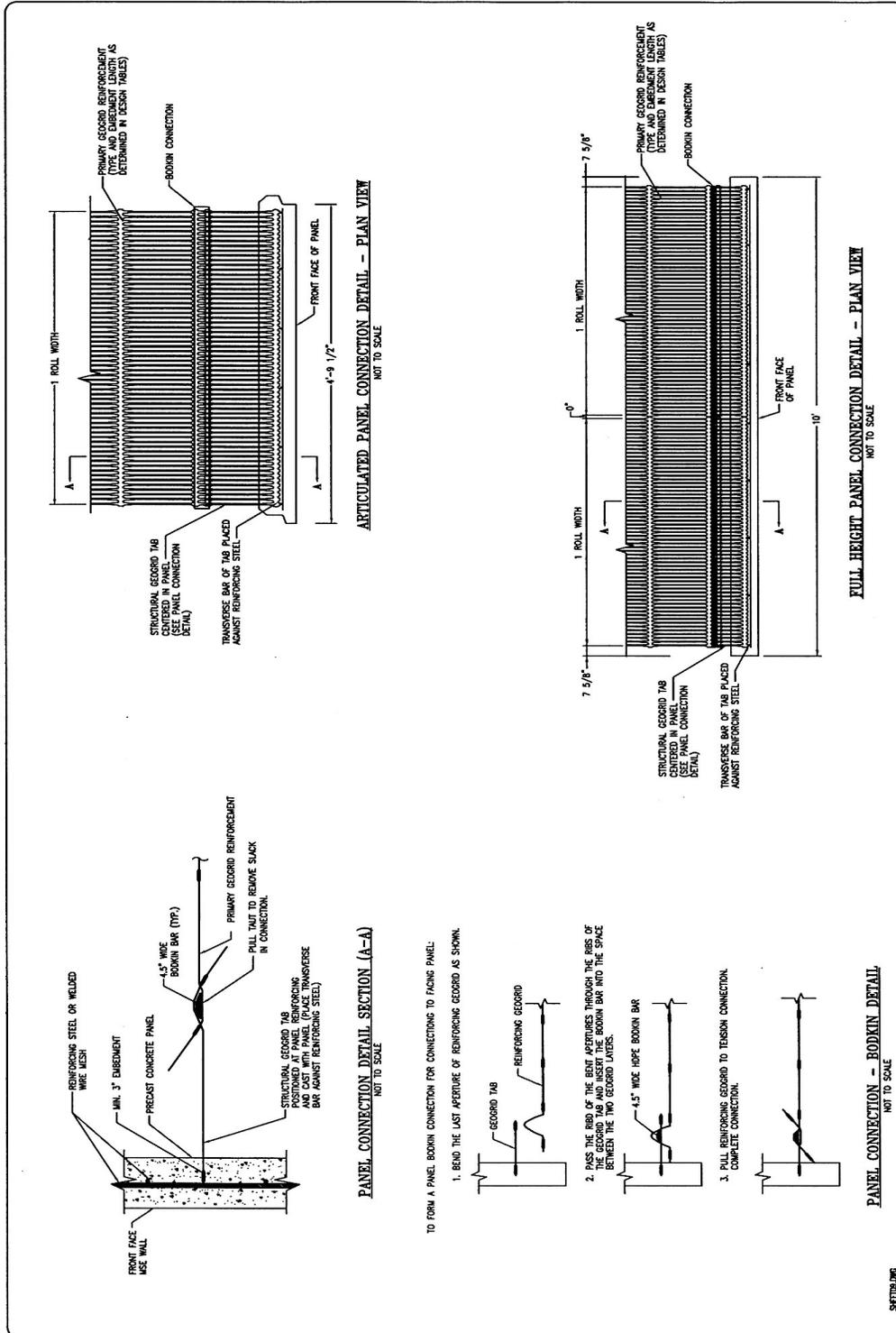
**GEORGD PANEL CONNECTION**

Sheet Number: **8 of 19**

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Denver, Colorado 80208  
(303) 254-1500

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STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS

GEORGRID PANEL CONNECTION

Sheet Number 9 of 19

Project Number: 8/1/04  
 Date: 8/1/04  
 As Shown  
 Prepared by: GLL  
 Checked by: TWT

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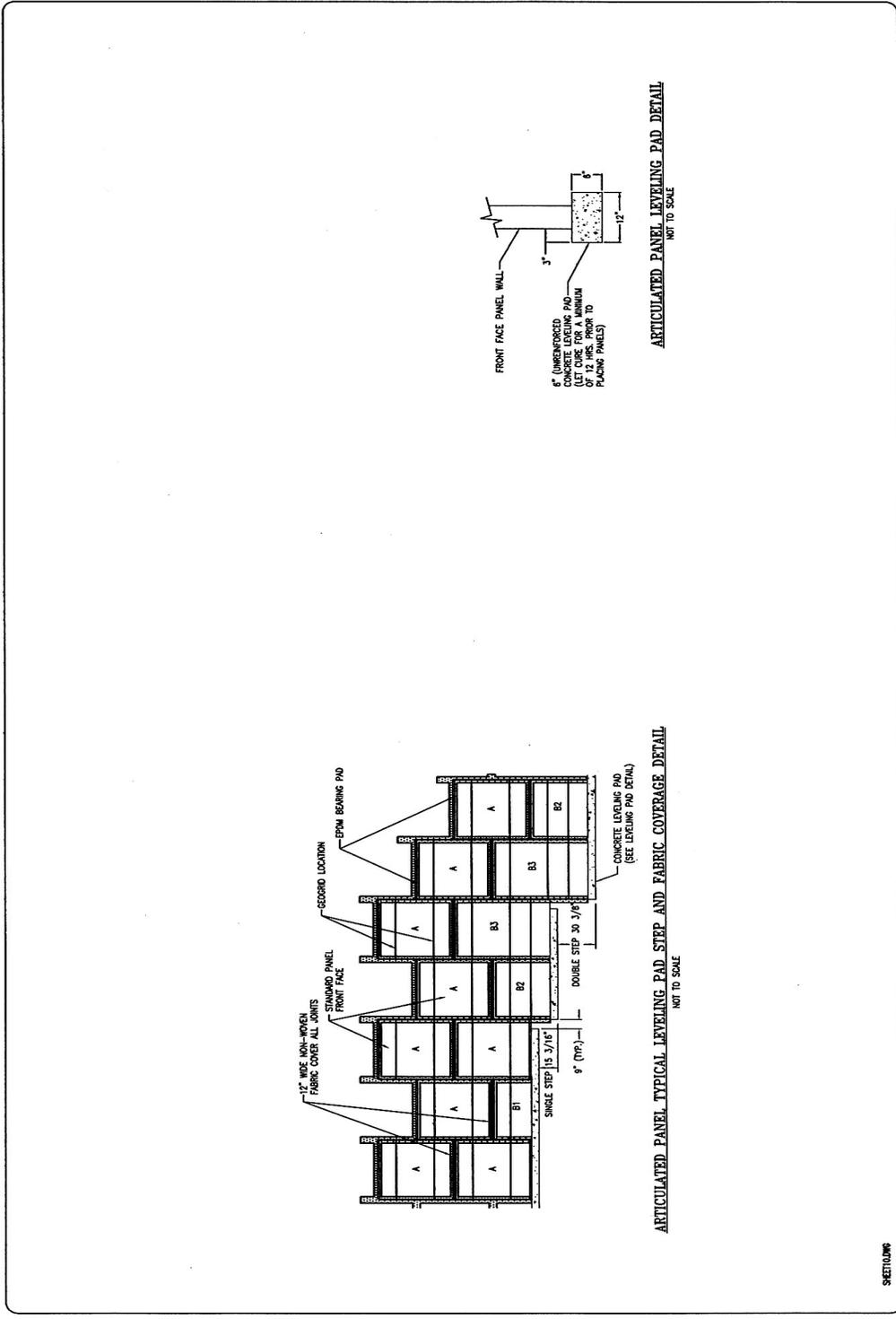
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0	8/1/04	ISSUED FOR REVIEW

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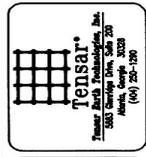
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS

ARES ARTICULATED PANEL LEVELING PAD

Sheet Number 10 of 19

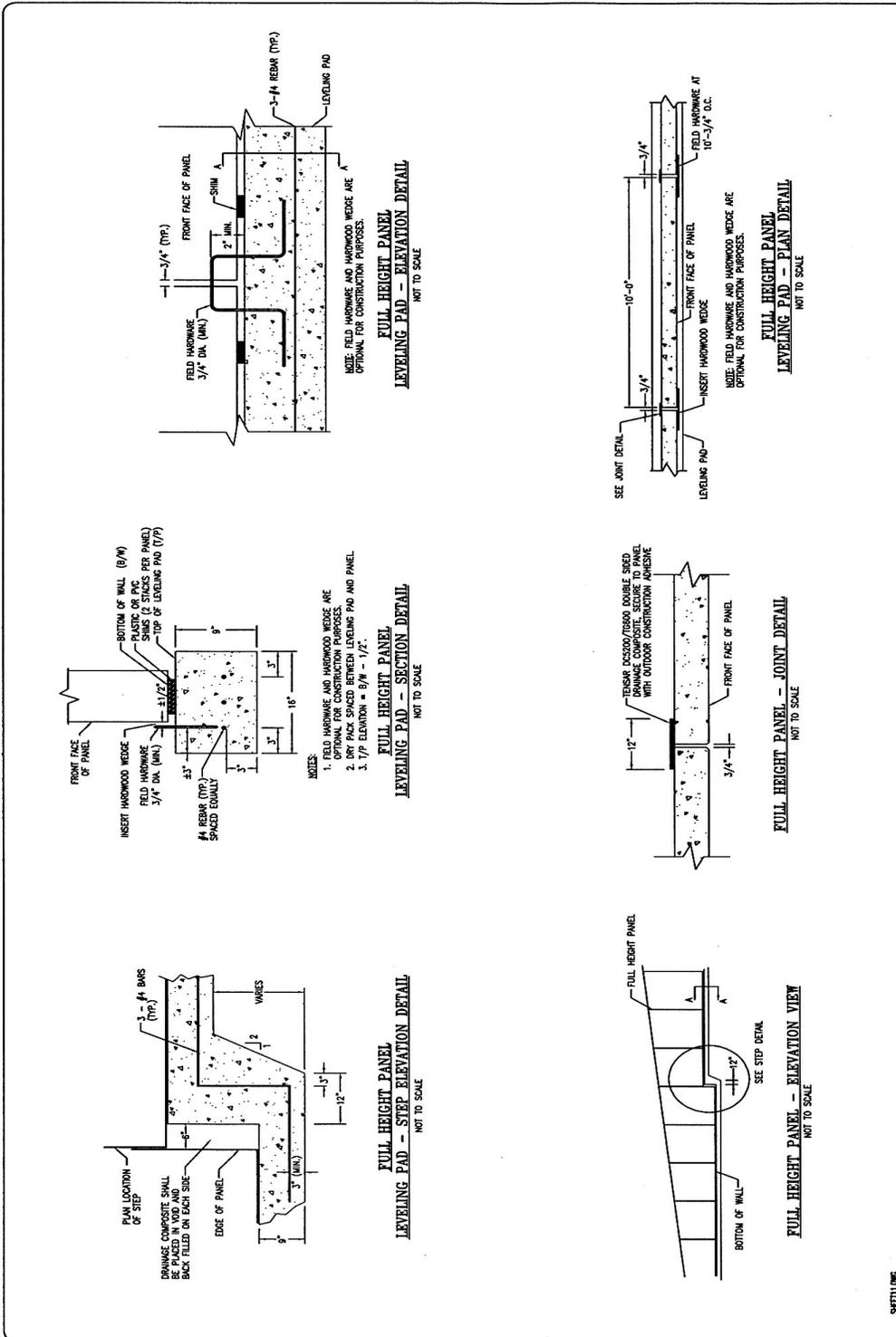
Page Number	Issue	Issued For Review
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REVISIONS	ISSUE



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SHEET 11.DWG

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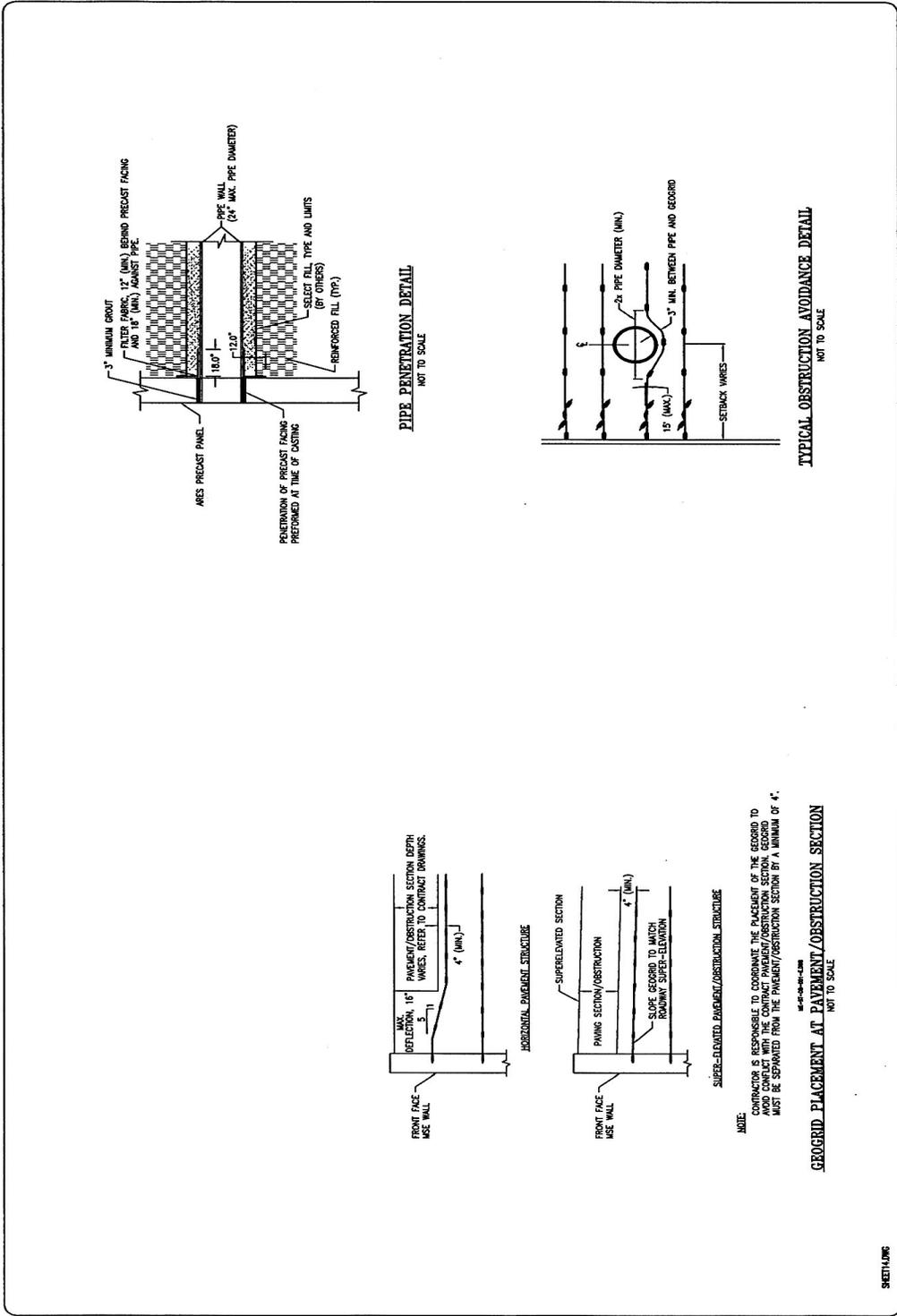
**STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS**

**ARES FULL HEIGHT PANEL LEVELING PAD**

Sheet Number  
**11 of 19**







**STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS**

Project Number: 07/04  
 Date: 5/2/04  
 Drawn by: A. Shum  
 Checked by: G.D.  
 Scale: 1/4" = 1'-0"  
 Created by:

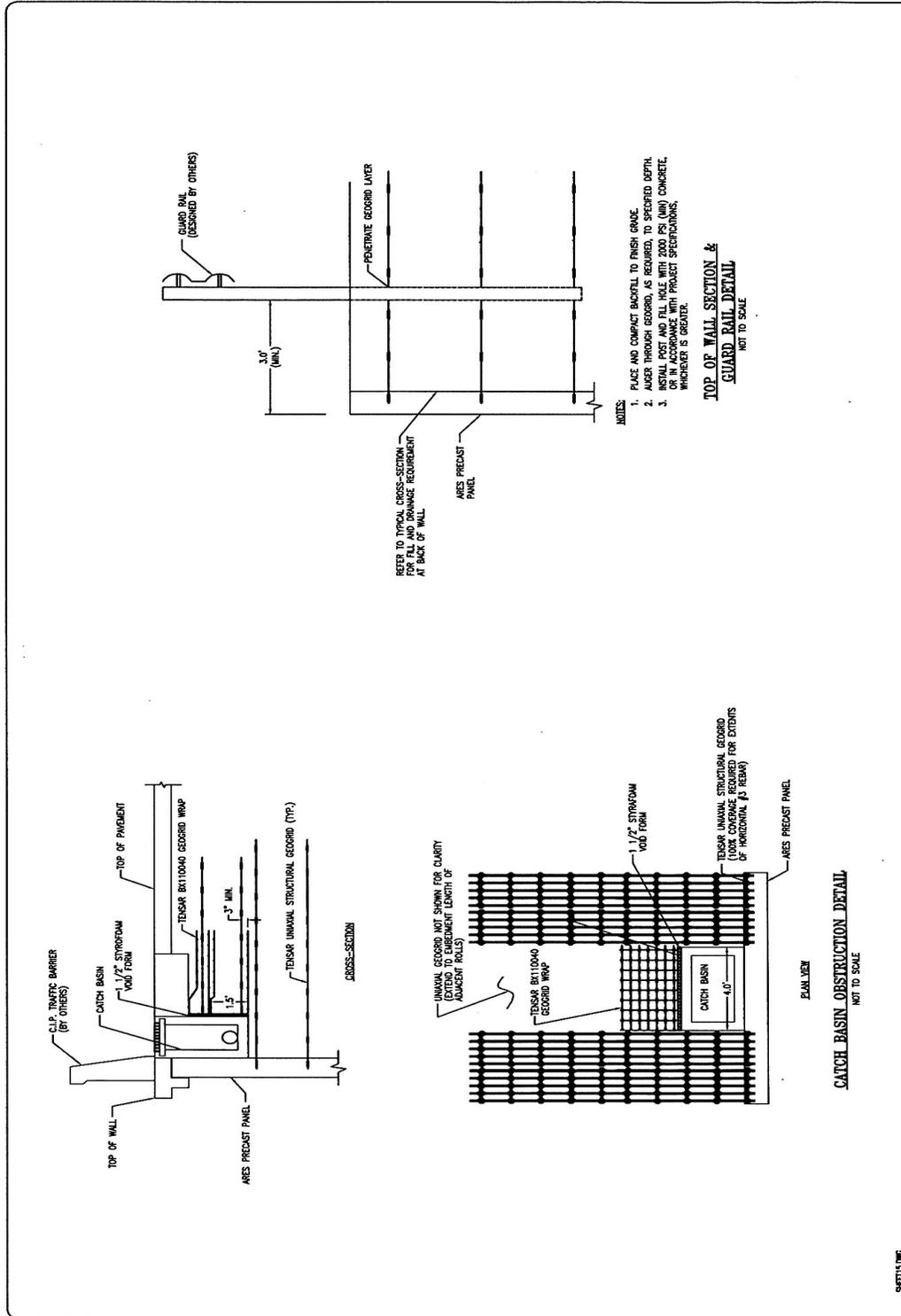
**OBSTRUCTIONS**

Sheet Number: 14 of 19

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 Atlanta, Georgia 30328  
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**STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS**

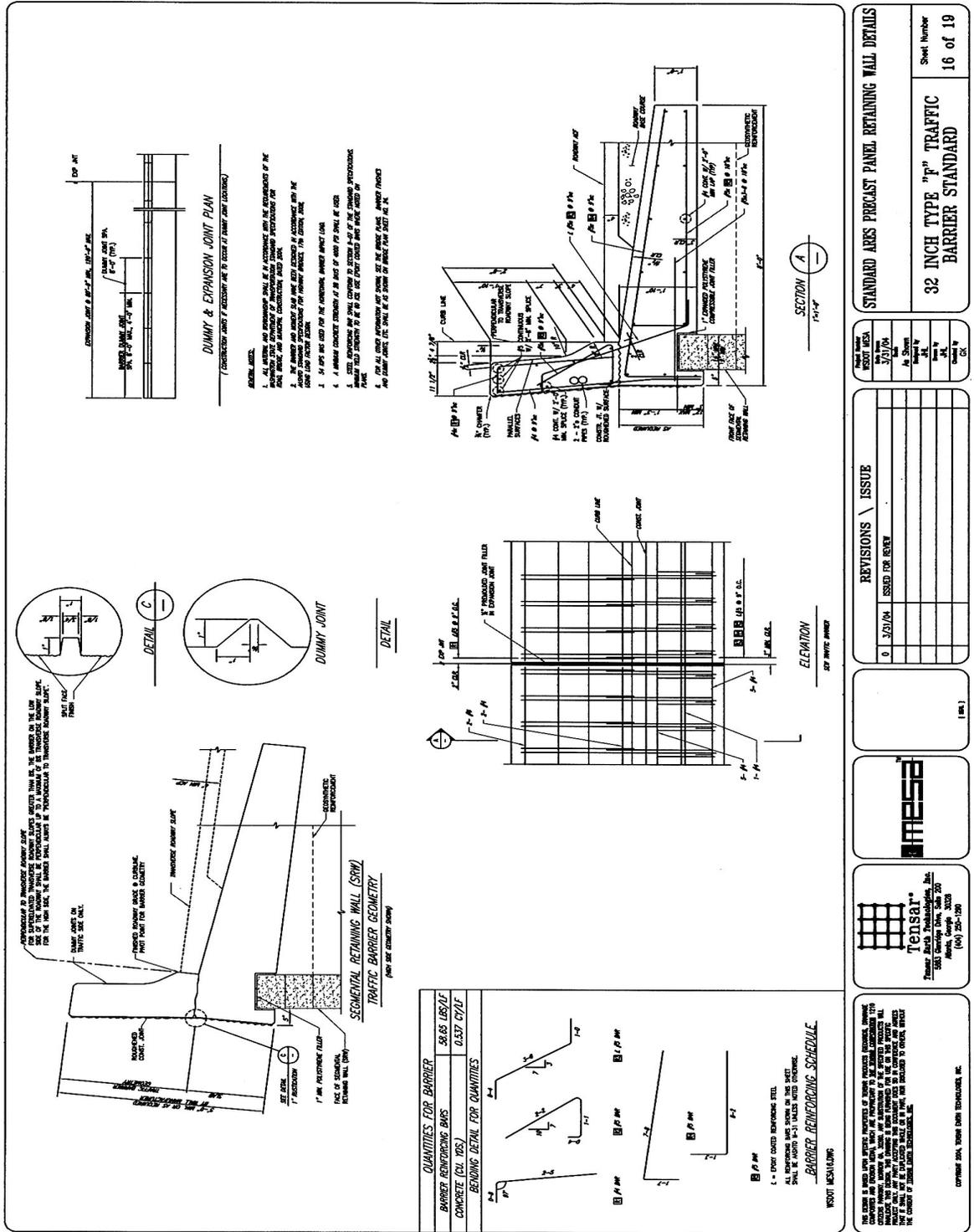
**TYPICAL DETAILS**

Project Number: 07/04  
 Date: 8/6/04  
 By: [Signature]  
 Checked by: [Signature]  
 Title: [Signature]  
 Scale: 1/4" = 1'-0"  
 Sheet Number: 15 of 19

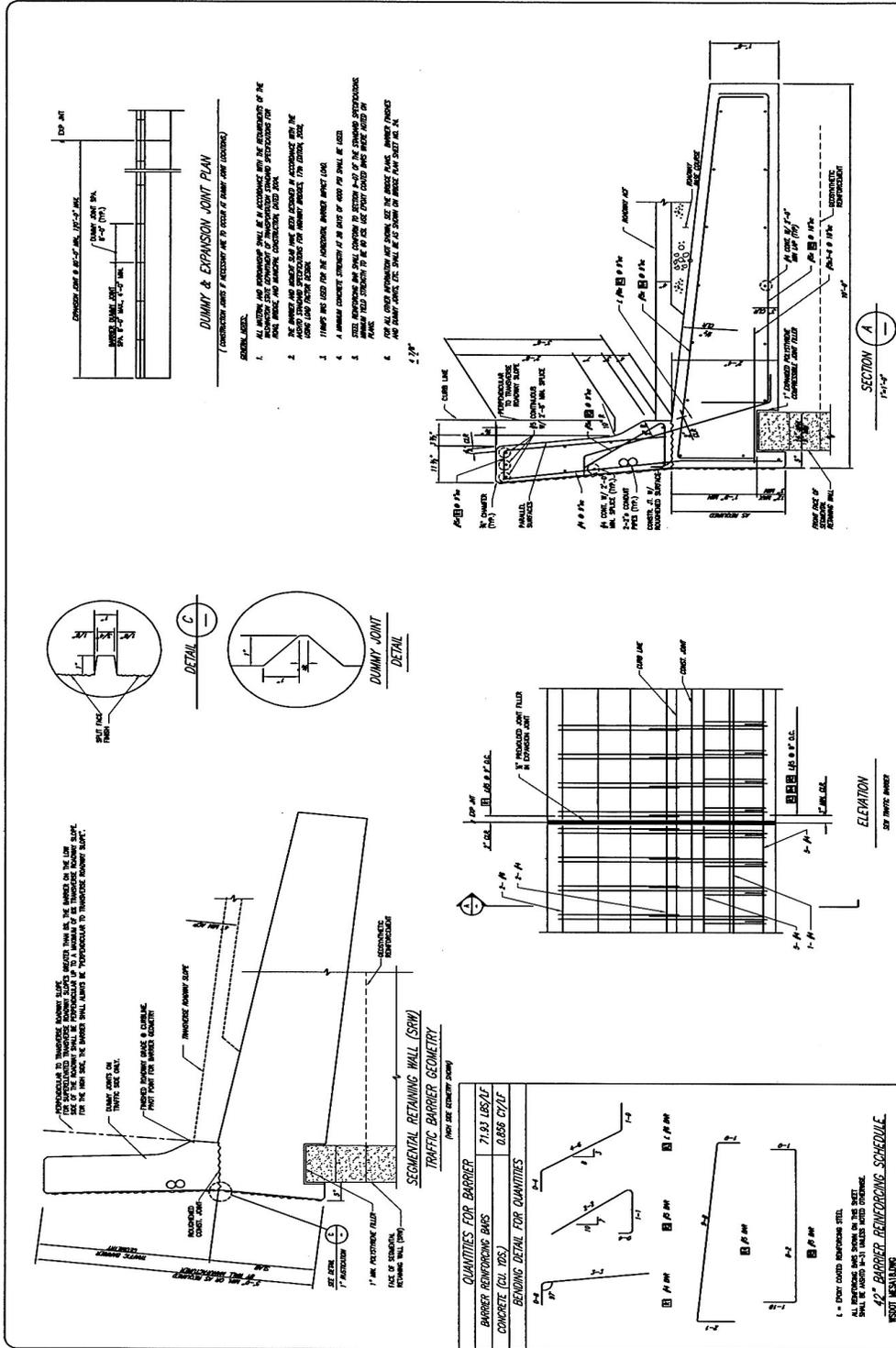
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 Atlanta, Georgia 30320  
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**STANDARD ARCS PRECAST PANEL RETAINING WALL DETAILS**

Sheet Number  
18 of 19

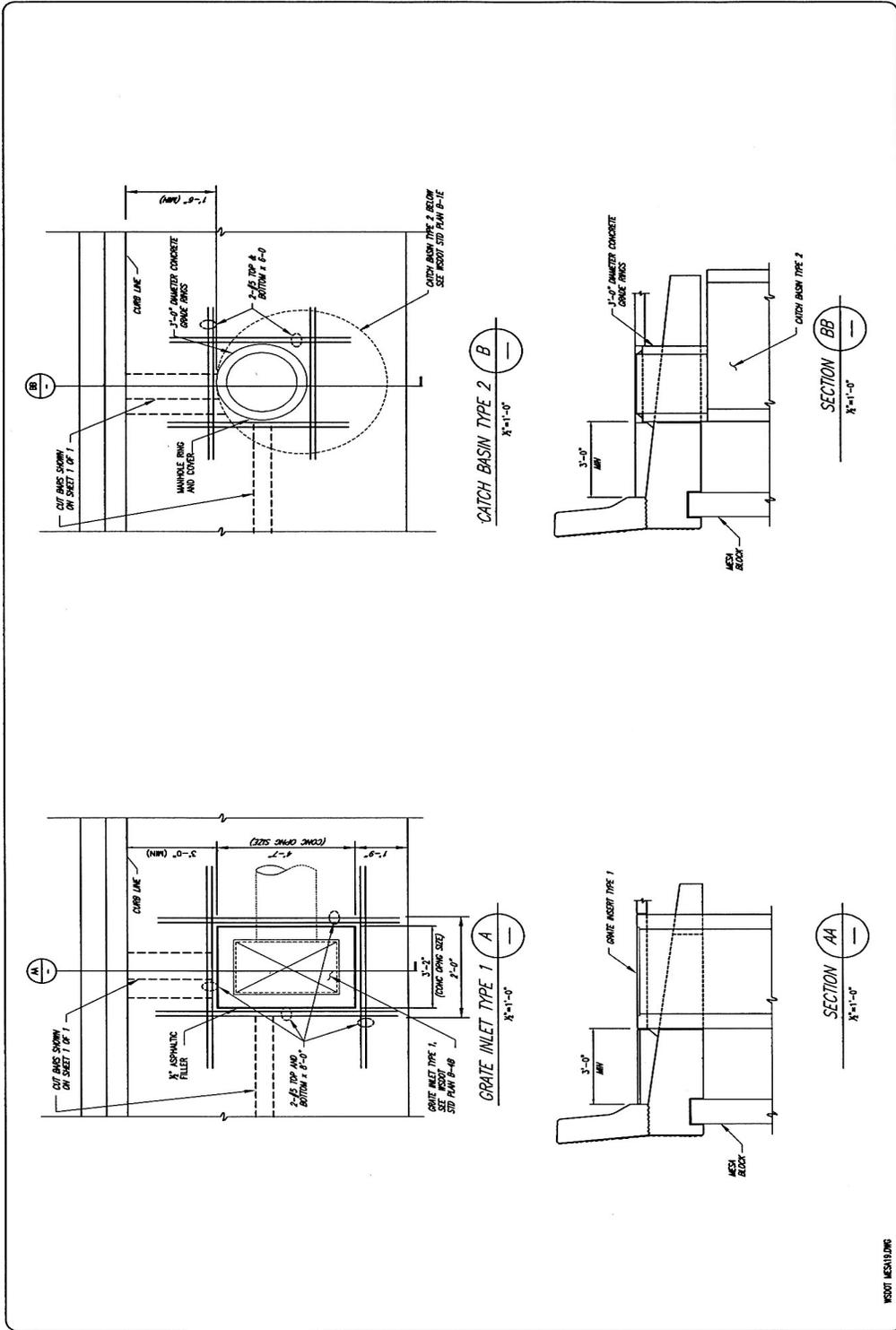
**42 INCH TYPE TRAFFIC BARRIER STANDARD**

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**TELSAR**  
Tensar North Technologies, Inc.  
5800 Gateway Drive, Suite 200  
Charlotte, NC 28217  
(704) 225-1200

**MESA**

CONTRACTOR: TENSAR DATA TECHNOLOGIES, INC.



WOOD MESHING

FOR USE IN THIS PROJECT, THE FOLLOWING IS THE SPECIFICATION FOR THE WOOD MESHING TO BE USED IN THE CONSTRUCTION OF THE WALLS. THE MESHING SHALL BE MANUFACTURED BY TENSAR, INC. OR ITS AFFILIATED COMPANIES. THE MESHING SHALL BE MANUFACTURED IN ACCORDANCE WITH THE REQUIREMENTS OF THE TENSAR ARES WALL SYSTEM. THE MESHING SHALL BE MANUFACTURED IN ACCORDANCE WITH THE REQUIREMENTS OF THE TENSAR ARES WALL SYSTEM. THE MESHING SHALL BE MANUFACTURED IN ACCORDANCE WITH THE REQUIREMENTS OF THE TENSAR ARES WALL SYSTEM.

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Tensar Earth Technologies, Inc.  
5000 University Drive, Suite 200  
Ann Arbor, MI 48106  
(734) 241-1200

**MESA**

STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS

42 INCH TYPE 19" TRAFFIC BARRIER STANDARD

Sheet Number  
19 of 19

NO.	DATE	ISSUE
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## ***Preapproved Wall Appendix: Specific Appendix 15-N Requirements and Details for Nelson Walls***

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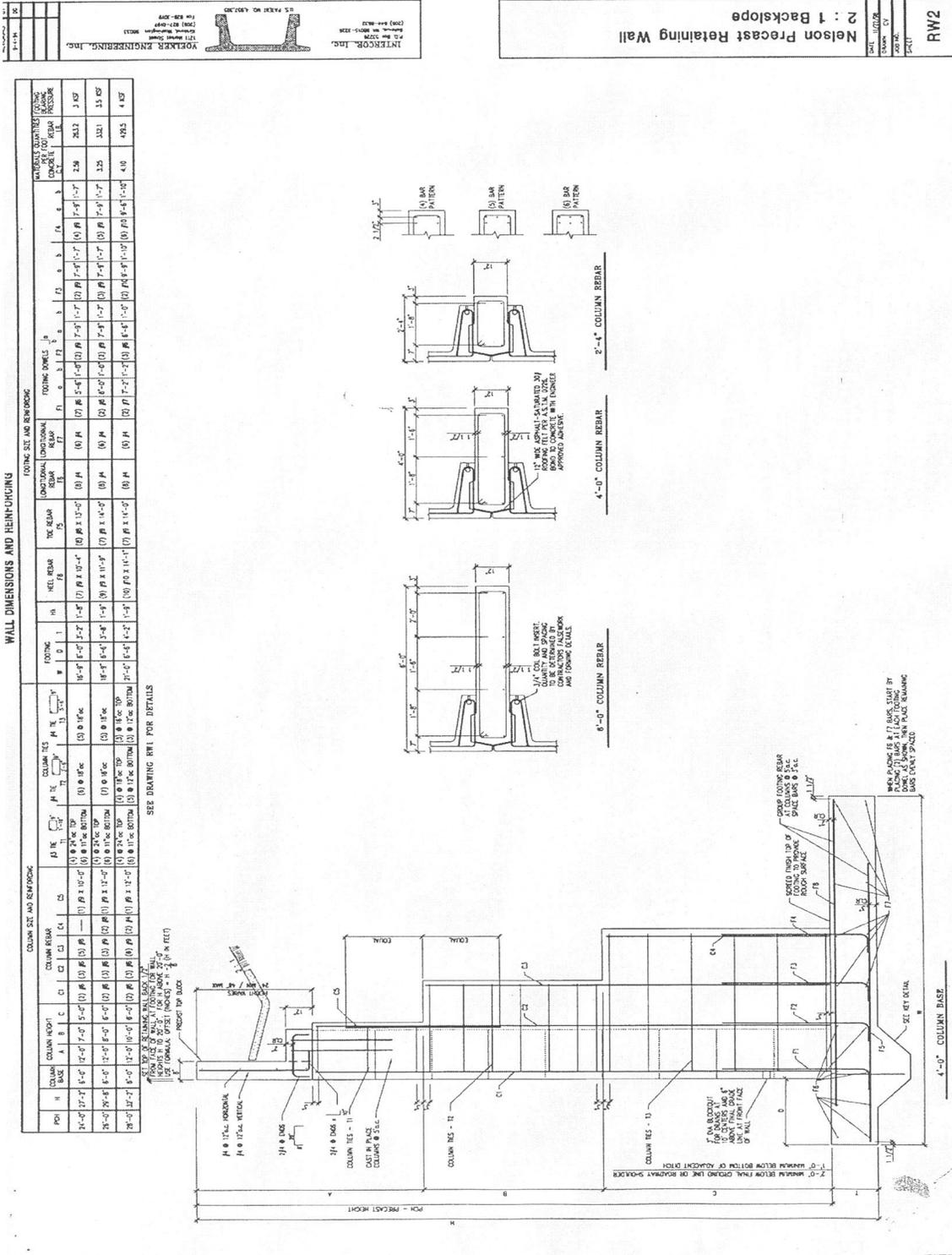
In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply to the design of the Nelson Wall:

No HITEC evaluation report is currently available for this wall system. However, in general, this wall system is used as a precast concrete substitute for the WSDOT Standard Plan Reinforced Concrete Cantilever Wall. The design procedures used for Nelson Walls are based on 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.

The preapproved height for this wall system (28 feet) is less than the standard preapproved height of 33 feet for proprietary wall systems. Use of this wall system for heights greater than 28 feet requires approval by the State Bridge Design Engineer and the State Geotechnical Engineer.

Approved details for the Nelson Wall system are provided in the following plan sheets. Note that no approved details for penetration of culverts or other objects through the wall face were provided. Therefore, this wall system is not preapproved for such situations. This wall system is preapproved for placement of traffic barriers on top of the wall.











# **Preapproved Wall Appendix: Specific Requirements and Details for Tensar Welded Wire Form Walls**

## **Appendix 15-O**

---

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 as listed 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, and on no more than 50% of the layers in a given section of wall.
- 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.

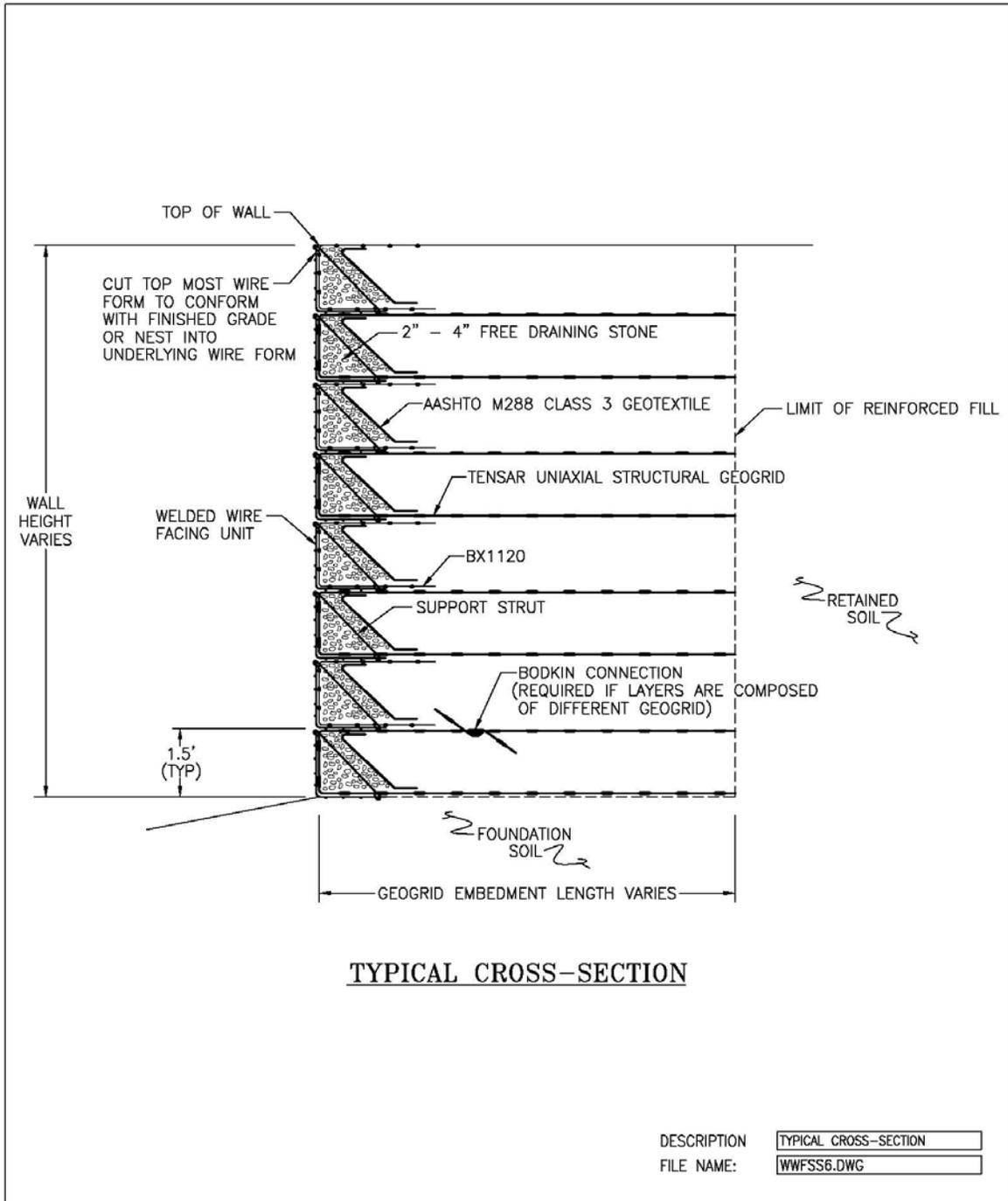
Tensar Primary Soil Reinforcement Geogrid Product	Tensar Product to Which Soil Reinforcement is Connected	Connection Strength Reduction Factor, CRu
UMESA3/UX1400HS	UMESA6/UX1700HS	1.0
UMESA4/UX1500HS	UMESA6/UX1700HS	1.0
UMESA5/UX1600HS	UMESA6/UX1700HS	1.0
UMESA6/UX1700HS	UMESA6/UX1700HS	0.91
UMESA3/UX1400HS	UMESA3/UX1400HS	0.85
UMESA4/UX1500HS	UMESA4/UX1500HS	0.79
UMESA5/UX1600HS	UMESA5/UX1600HS	0.87
UMESA6/UX1700HS	UMESA6/UX1700HS	0.91

**Approved Bodkin Connection Strength Reduction Factors  
for Tensar Welded Wire Form Walls**

*Table 15-O-1*

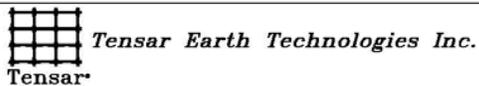
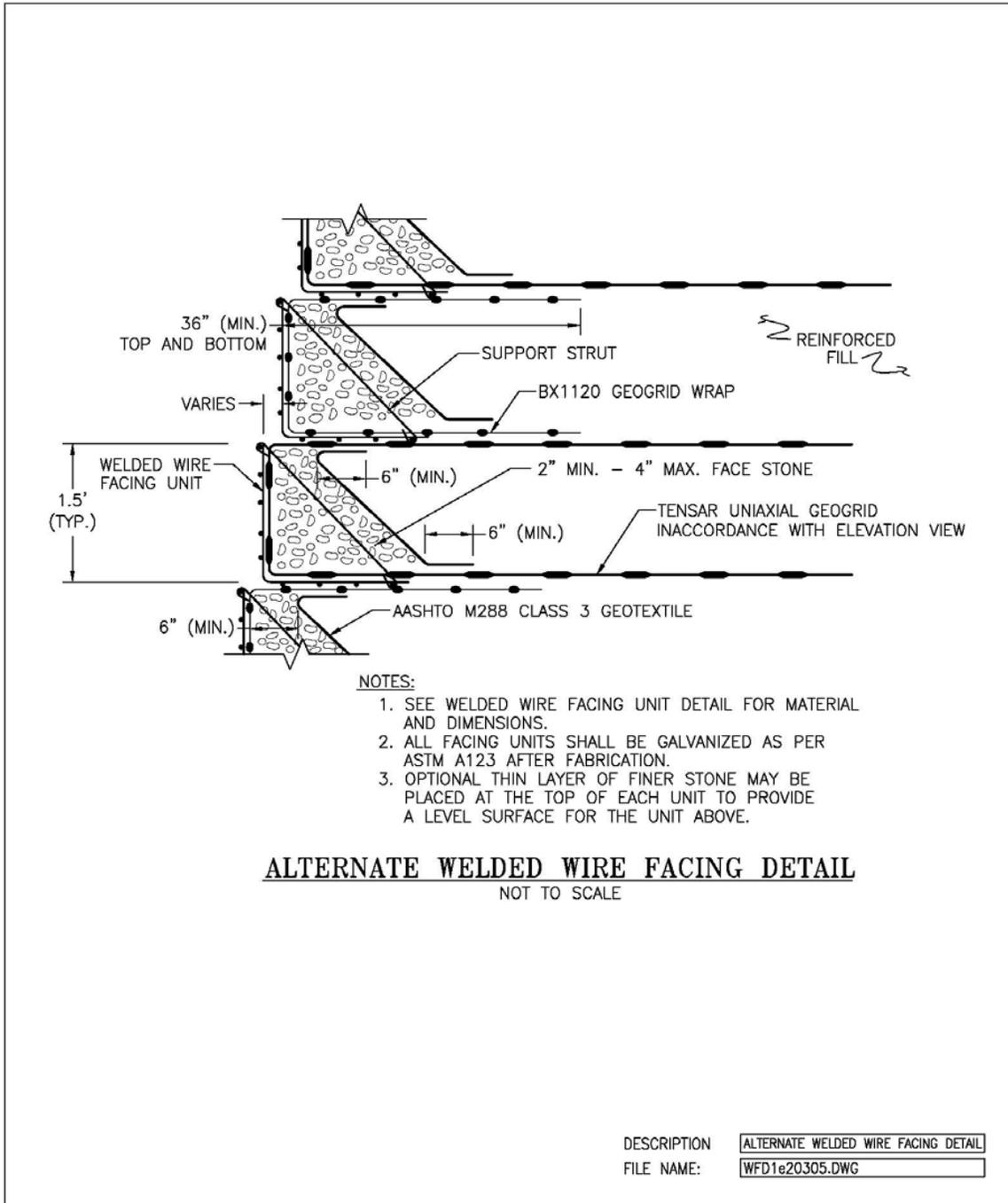
Approved details for the Tensar Welded Wire Form Wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- 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 be 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 plan sheet.
- Regarding the plantable face alternate plan details on page 6, this alternative shall 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 a minimum 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 not occur.

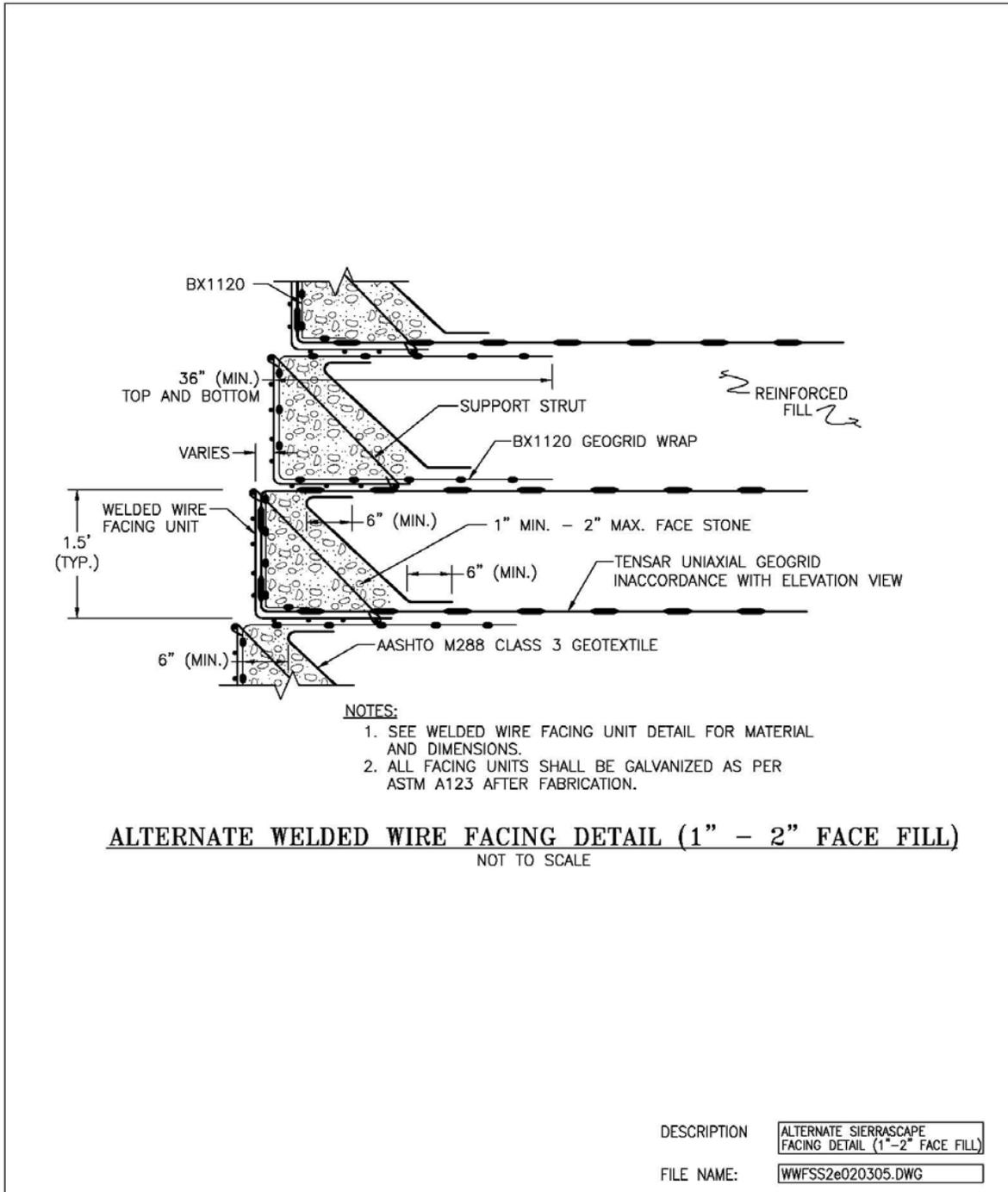


Tensar Earth Technologies Inc.

TYPICAL CROSS-SECTION

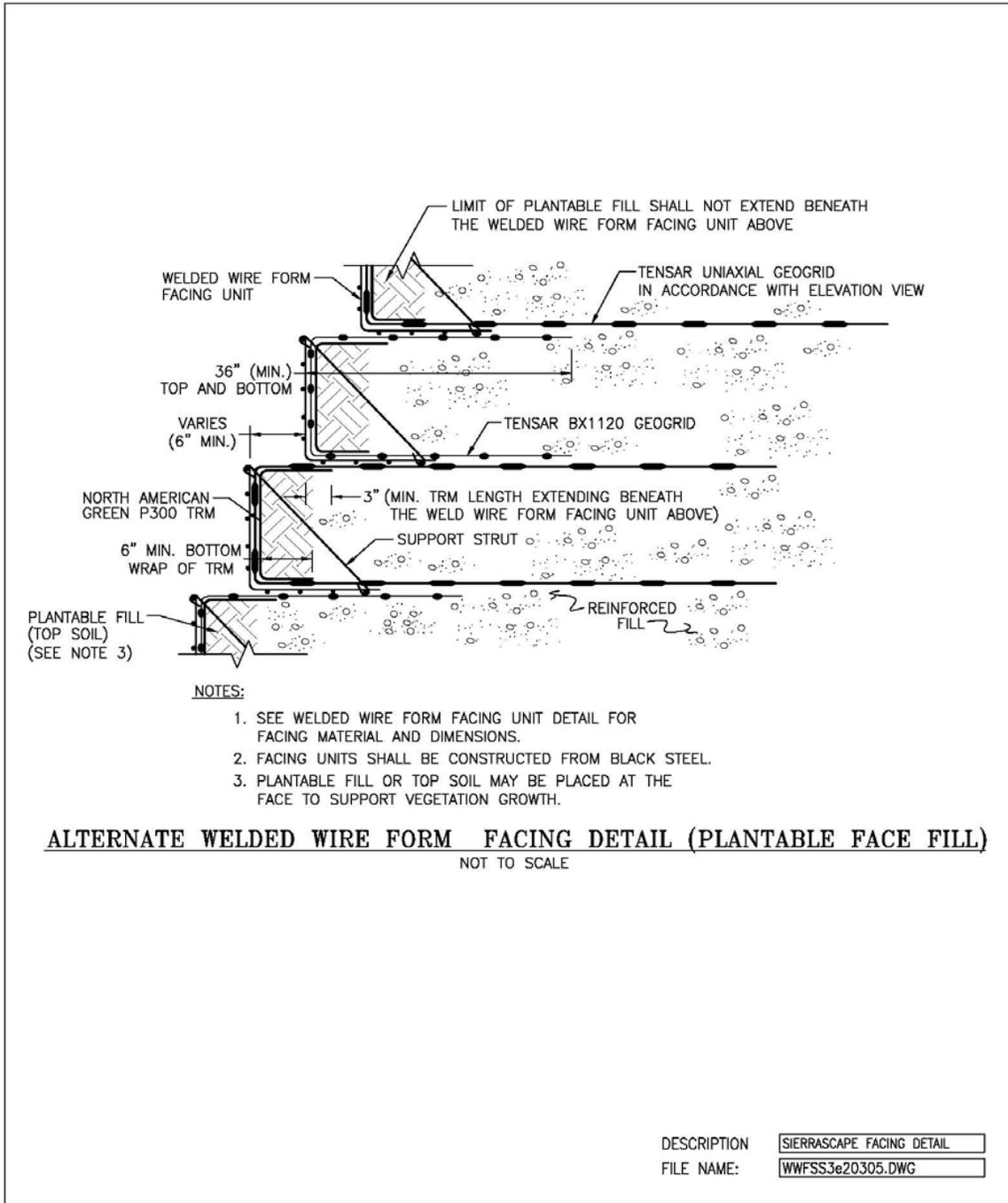


TYPICAL DETAIL



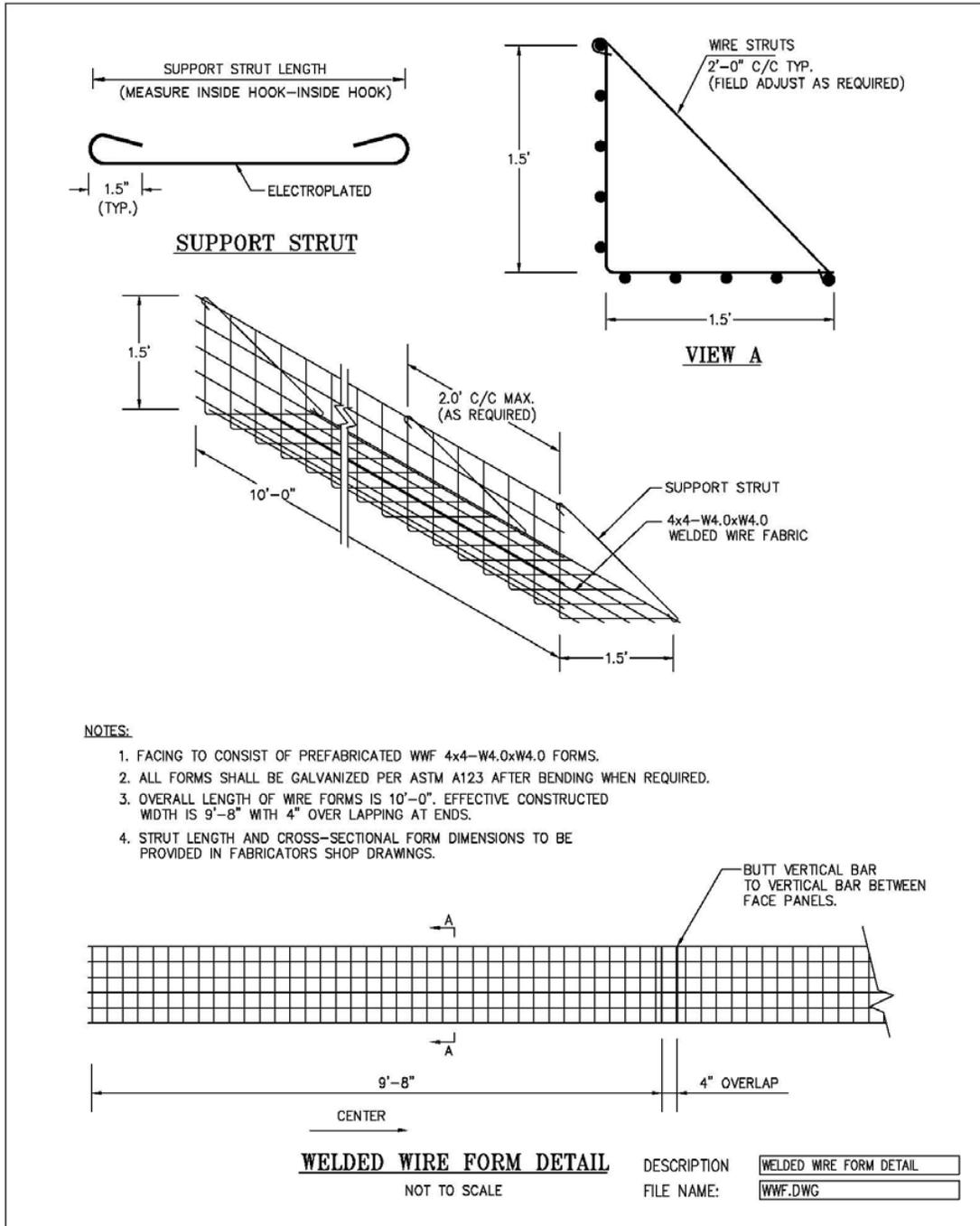
Tensar Earth Technologies Inc.

TYPICAL DETAIL



Tensar Earth Technologies Inc.

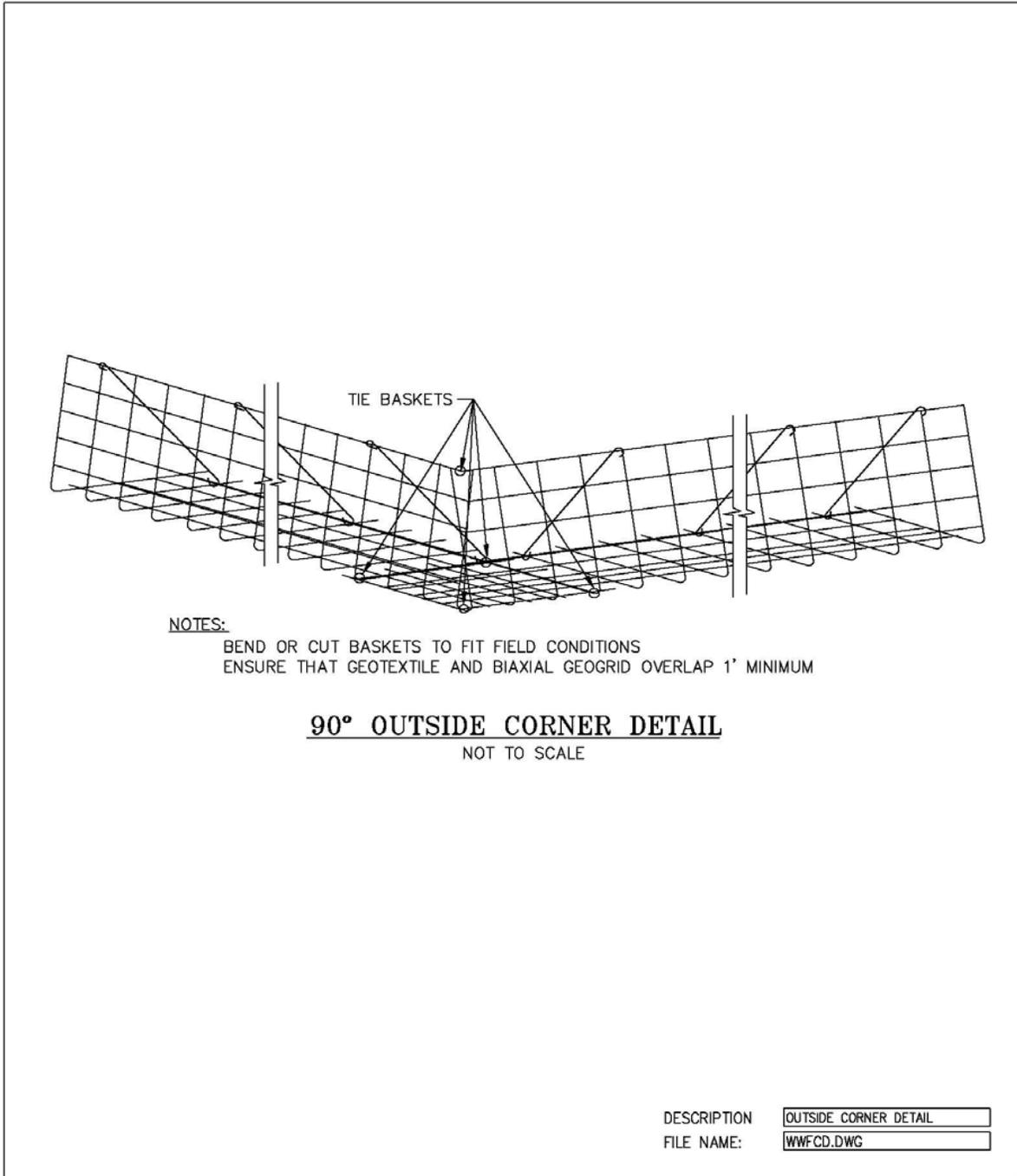
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Tensar Earth Technologies Inc.

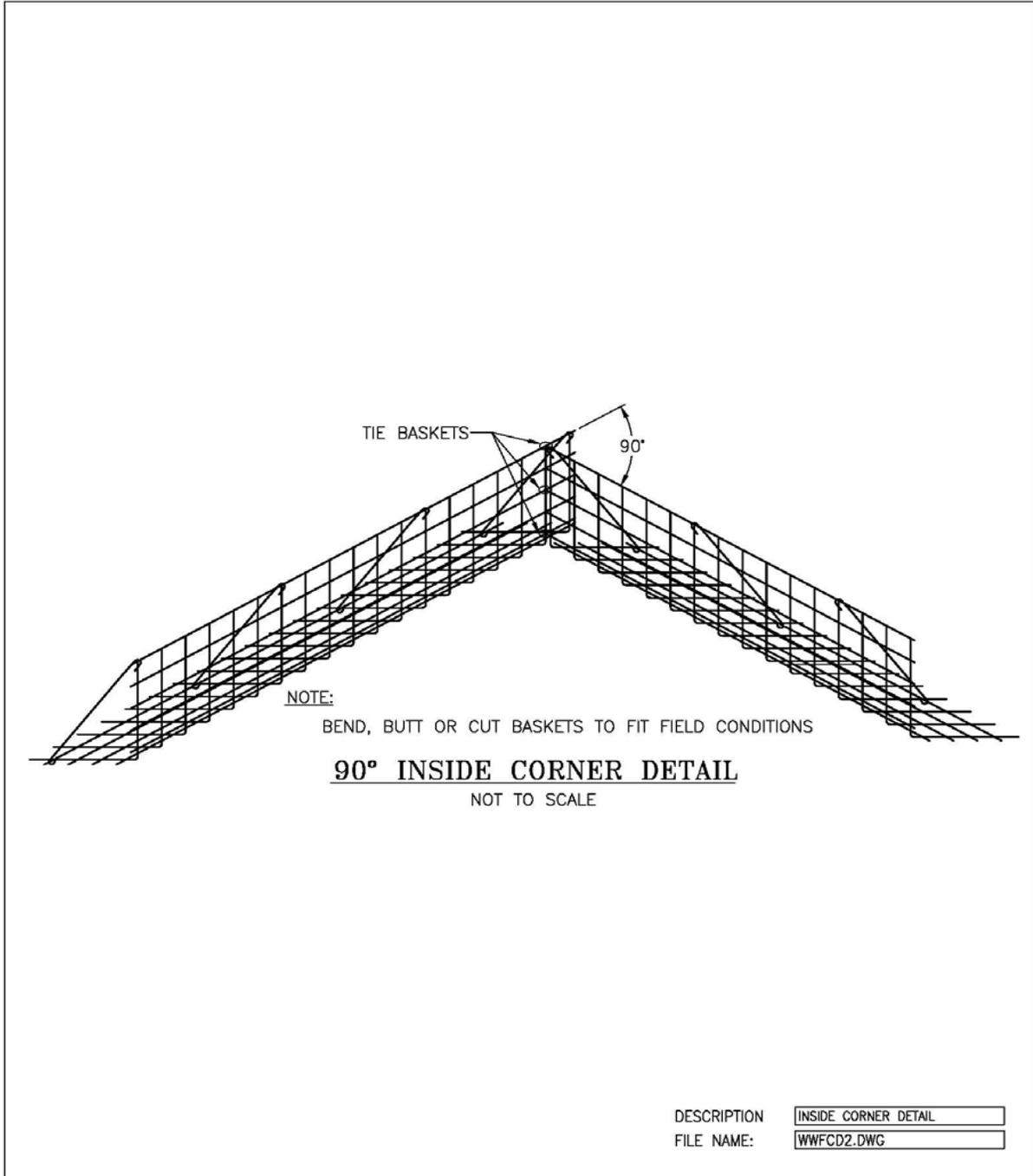
Tensar

TYPICAL DETAIL



Tensar Earth Technologies Inc.

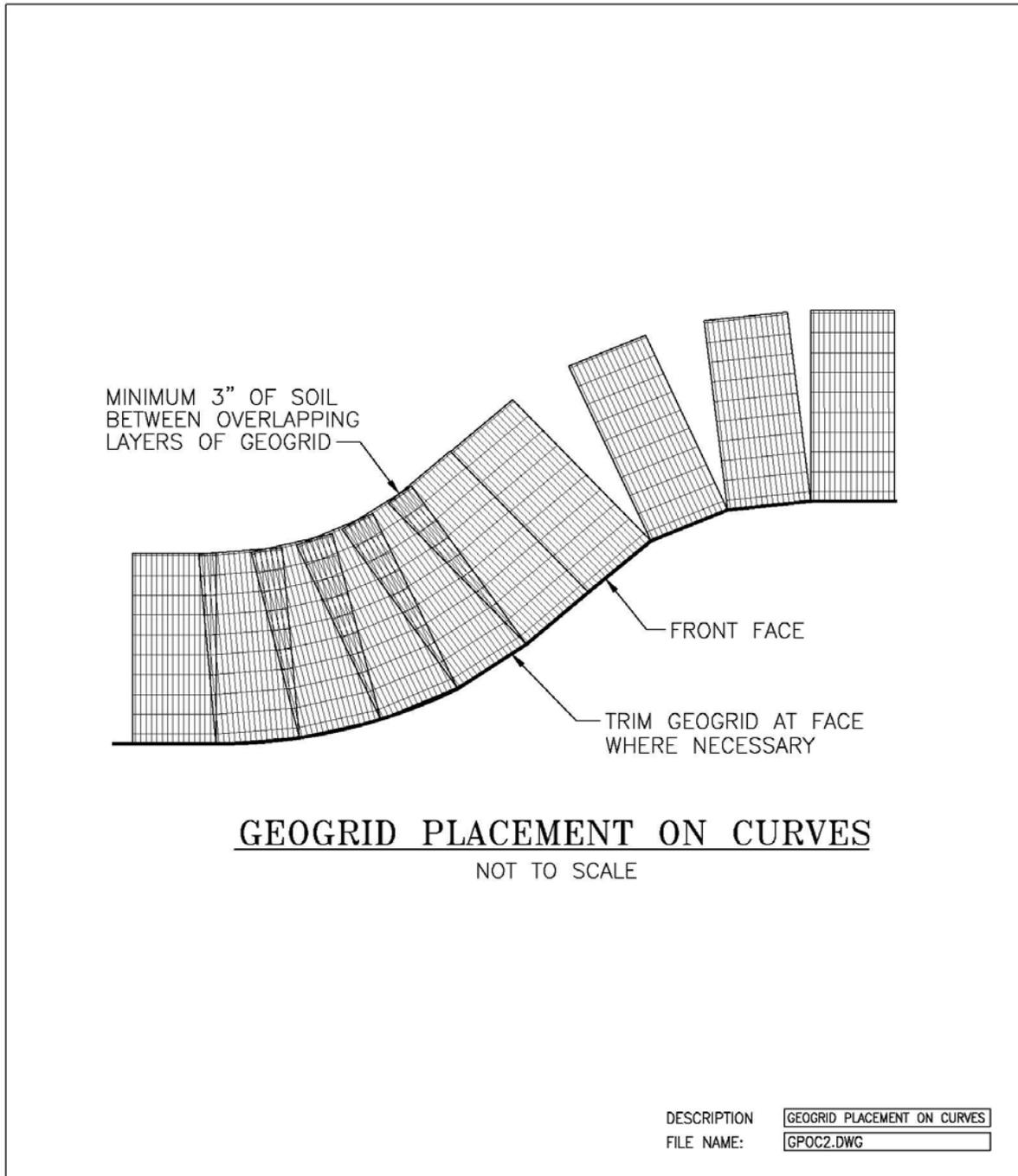
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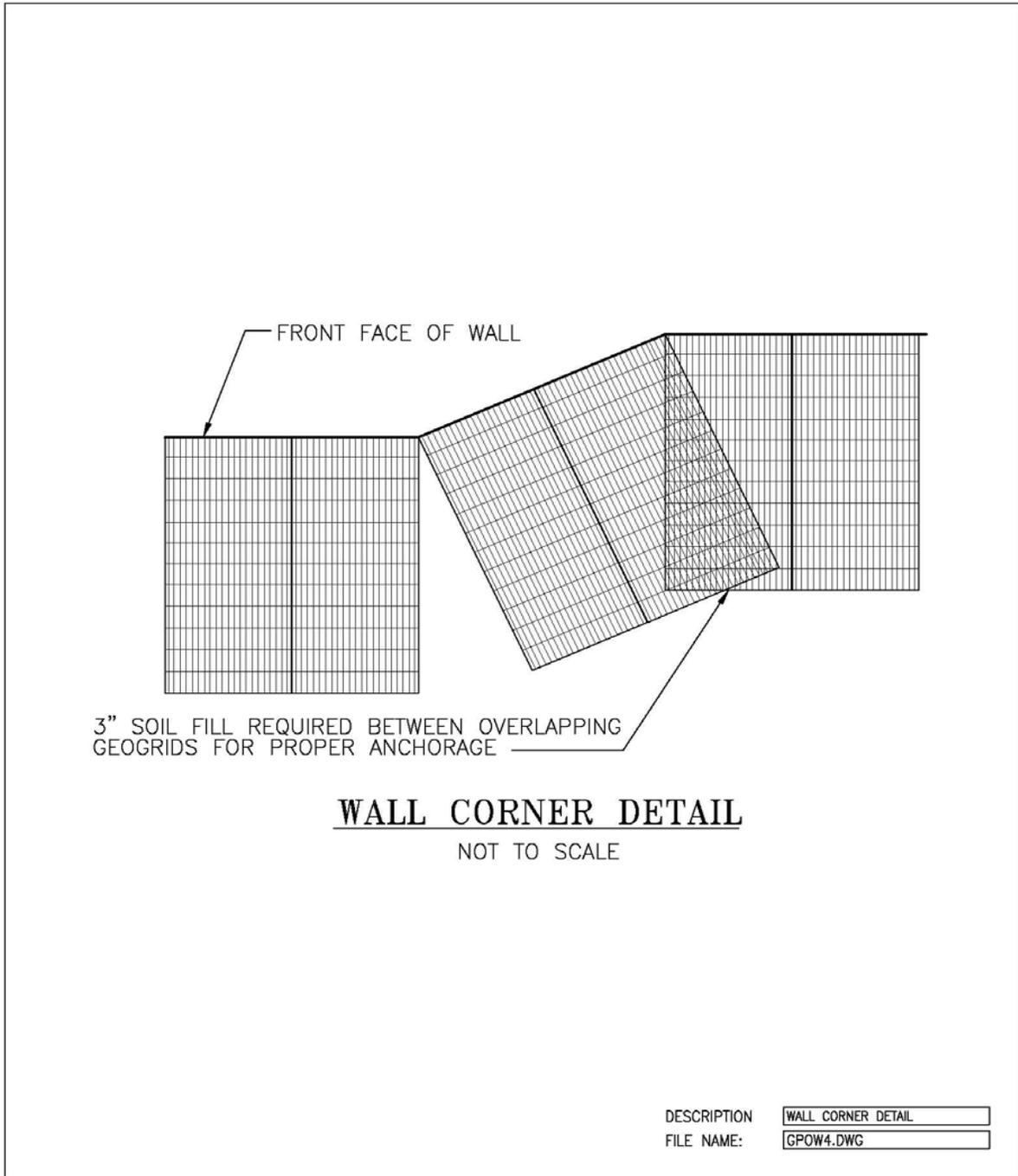
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TYPICAL DETAIL



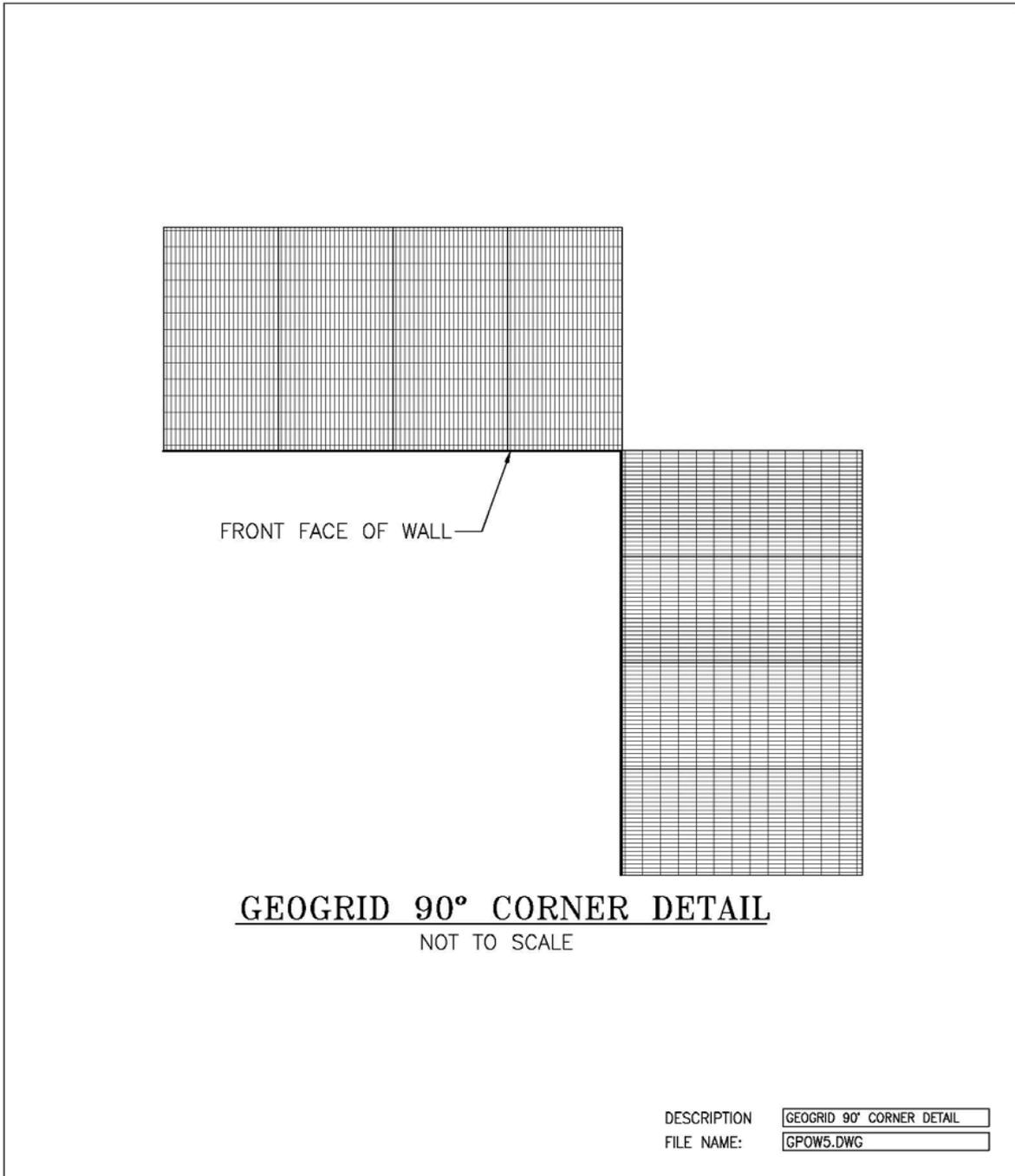
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TYPICAL DETAIL



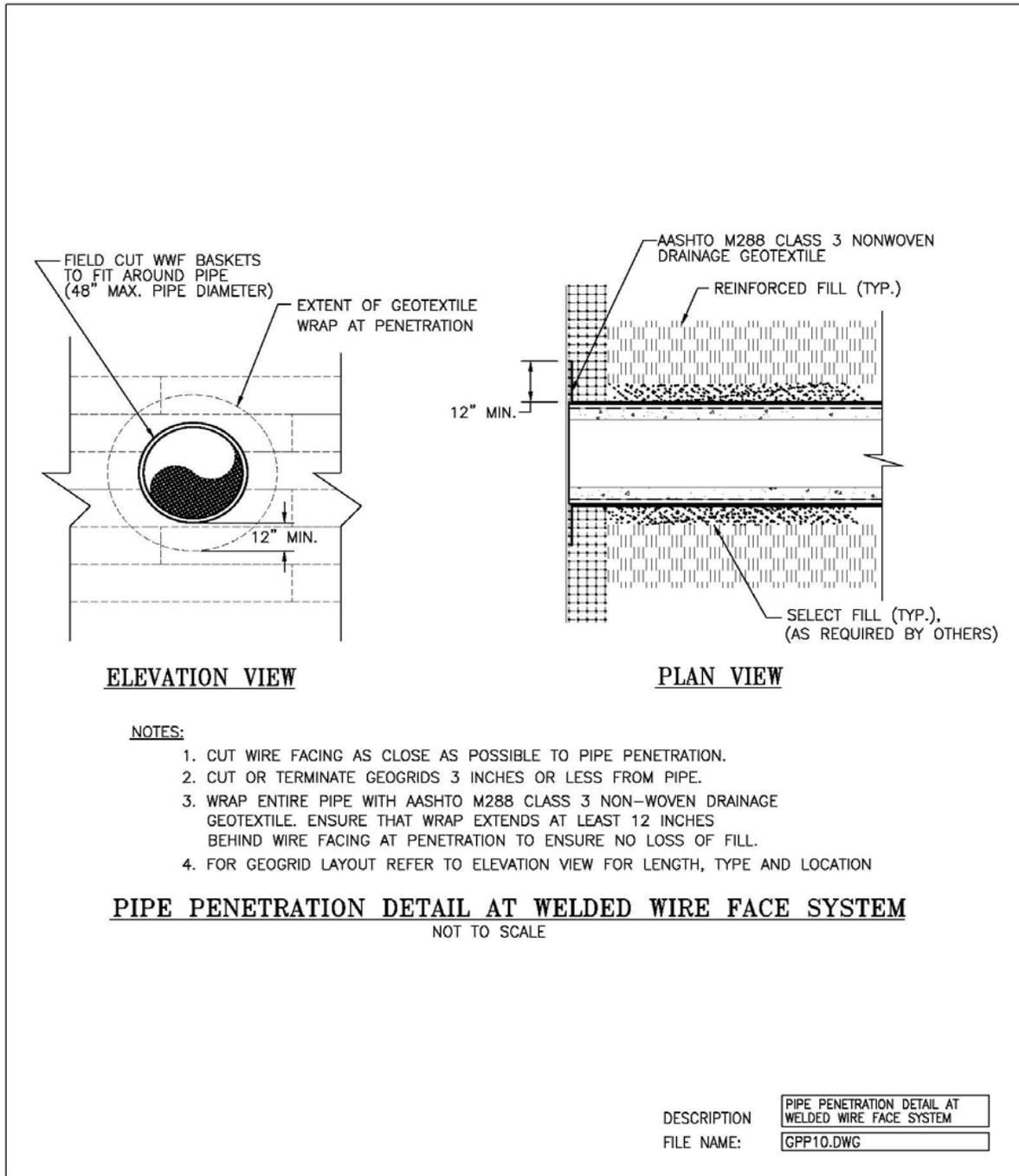
Tensar Earth Technologies Inc.

TYPICAL DETAIL



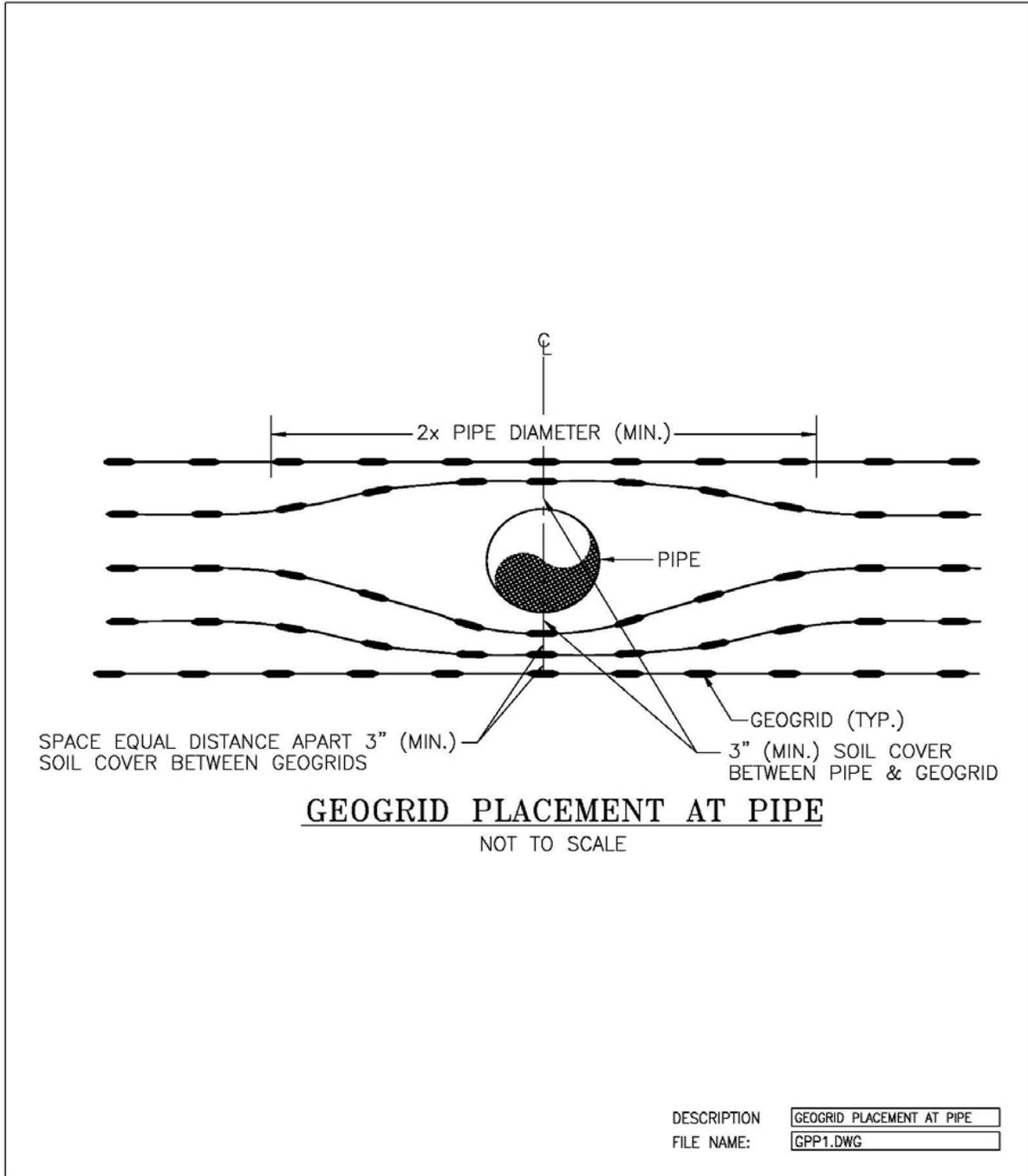
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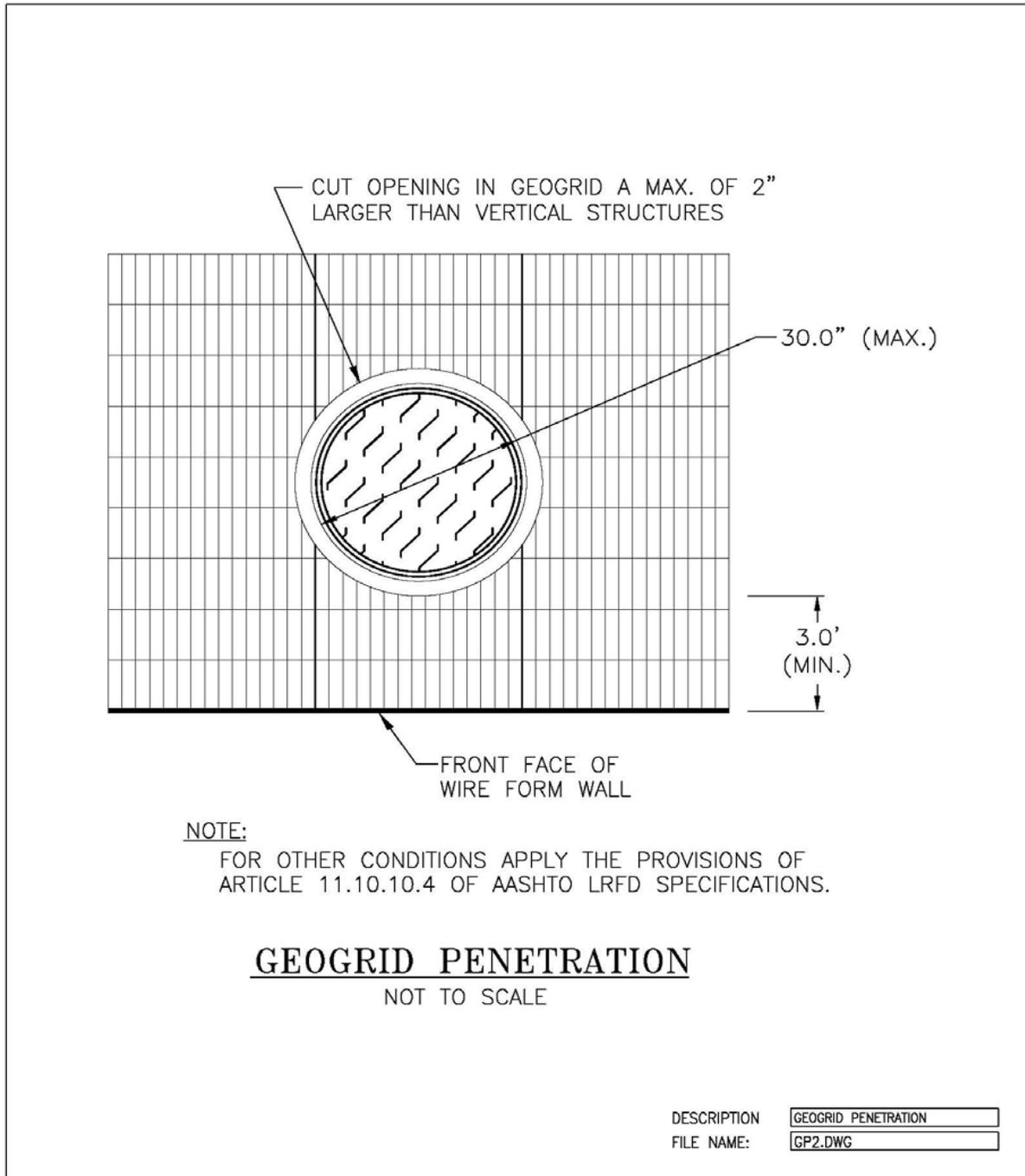
Tensar Earth Technologies Inc.

TYPICAL DETAIL



Tensar Earth Technologies Inc.

TYPICAL DETAIL

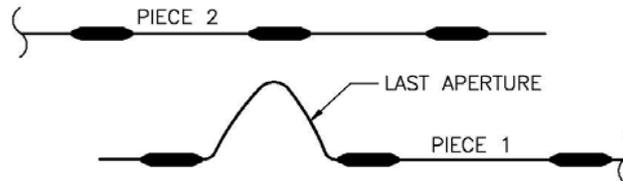


Tensar Earth Technologies Inc.

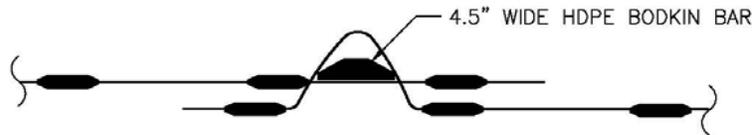
TYPICAL DETAIL

TO FORM A BODKIN CONNECTION:

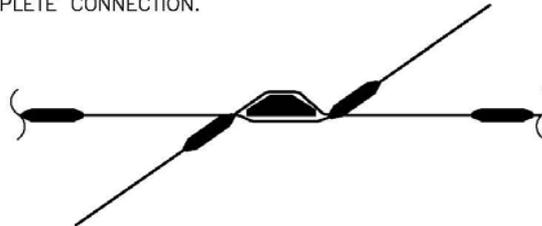
1. BEND THE LAST APERTURE OF ONE PIECE OF GEOGRID IN HALF.



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

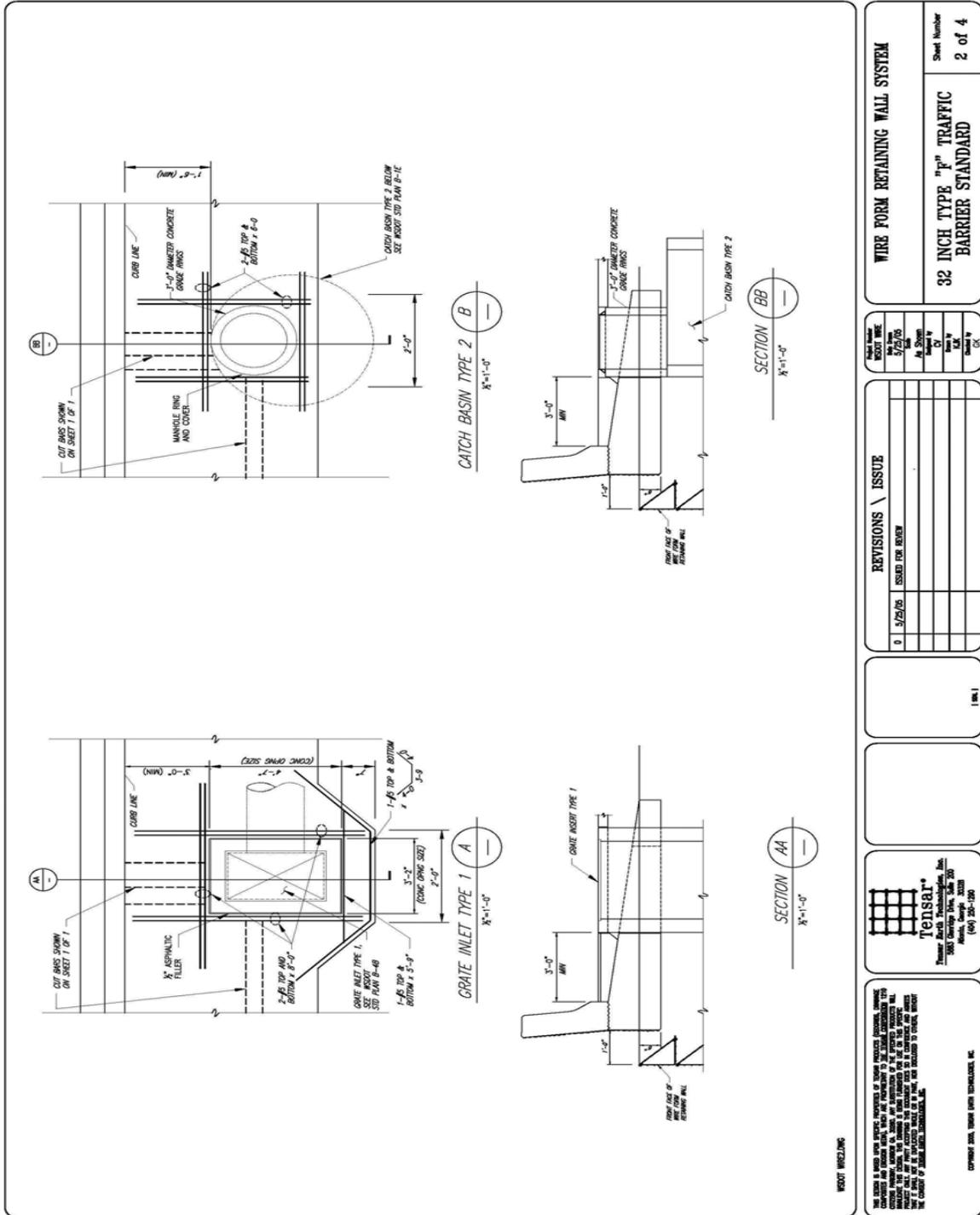
DESCRIPTION	BODKIN CONNECTION
FILE NAME:	BC.DWG



Tensar Earth Technologies Inc.

TYPICAL DETAIL

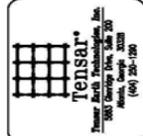




WOODWORKING

THIS SYSTEM IS DESIGNED AND MANUFACTURED BY TENSAR CORPORATION, 2000 TENSAR DRIVE, HUNTSVILLE, ALABAMA 35894. ALL DIMENSIONS ARE IN FEET AND INCHES UNLESS OTHERWISE SPECIFIED. THIS SYSTEM IS DESIGNED TO BE USED IN CONFORMANCE WITH THE STANDARD SPECIFICATIONS FOR BRIDGE AND STRUCTURAL CONSTRUCTION, 1995 EDITION, AS AMENDED, AND THE STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGE AND STRUCTURAL CONSTRUCTION, 1995 EDITION, AS AMENDED. THE COMPANY OF THE USER SHALL BE RESPONSIBLE FOR THE DESIGN OF THE WALL AND FOUNDATION.

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REVISIONS / ISSUE
0 - 5/25/05 ISSUED FOR REVIEW

DATE	BY	CHKD BY	APP'D BY

WIRE FORM RETAINING WALL SYSTEM

32 INCH TYPE "F" TRAFFIC BARRIER STANDARD

Sheet Number  
2 of 4

**ELEVATION**  
SEE NOTE ABOVE

**SECTION A-A**  
1/4" = 1'-0"

**WIRE FORM RETAINING WALL (SPW)**  
TRAFFIC BARRIER GEOMETRY  
FROM THE CONCRETE BARRIERS

QUANTITIES FOR BARRIER	71.93 LBS/LF
BARRIER REINFORCING BARS	0.866 CY/CY
CONCRETE (CU YDS.)	
BENDING DETAIL TOP QUANTITIES	

1 - 5/8" DIAM. REINFORCING STEEL  
SHALL BE WAPTES W-31 UNLESS NOTED OTHERWISE.  
**42" BARRIER REINFORCING SCHEDULE**  
WOOD MODELING

**DUMMY & EXPANSION JOINT PLAN**  
(CONCRETE JOINTS IF NECESSARY ARE TO BE SET AT DUMMY JOINT LOCATIONS)

**GENERAL NOTES:**

- ALL MATERIALS AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION AND THE TENSAR WIRE FORM WALL SYSTEM PERFORMANCE FOR BARRIERS AND MANHOLE ENCLOSURES, 42" BARRIER.
- THE BARRIERS AND MANHOLE SLABS ARE ALSO DESIGNED IN ACCORDANCE WITH THE ABOVE STANDARD SPECIFICATIONS FOR MANHOLE BARRIERS, 75% STRENGTH.
- TRIMMS ARE USED FOR THE PROTECTIVE BARRIER IMPACT SLAB.
- A MINIMUM CONCRETE STRENGTH AT THE JOINTS OF 4000 PSI SHALL BE USED.
- STRENGTH DEVELOPMENT SHALL BE MONITORED AND REPORTED TO THE CONTRACTOR FOR RECORDS. FIELD TESTING SHALL BE CONDUCTED AS STATED IN THE STANDARD SPECIFICATIONS FOR CONSTRUCTION OF BARRIERS AND MANHOLE ENCLOSURES, 42" BARRIER.
- FOR ALL OTHER INFORMATION NOT SHOWN ON THE BARRIER PLANS, BARRIER FINISHES AND DUMMY JOINTS, ETC. SHALL BE AS SHOWN ON BARRIERS PART SHEET 15-1A.

**TRAFFIC BARRIER GEOMETRY**

**DUMMY JOINT DETAIL**

**REVISIONS / ISSUE**

NO.	DATE	ISSUED FOR REVISION
0	5/25/08	ISSUED FOR REVISION

**WIRE FORM RETAINING WALL SYSTEM**

**42 INCH TYPE "F" TRAFFIC BARRIER STANDARD**

Sheet Number  
3 of 4



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.

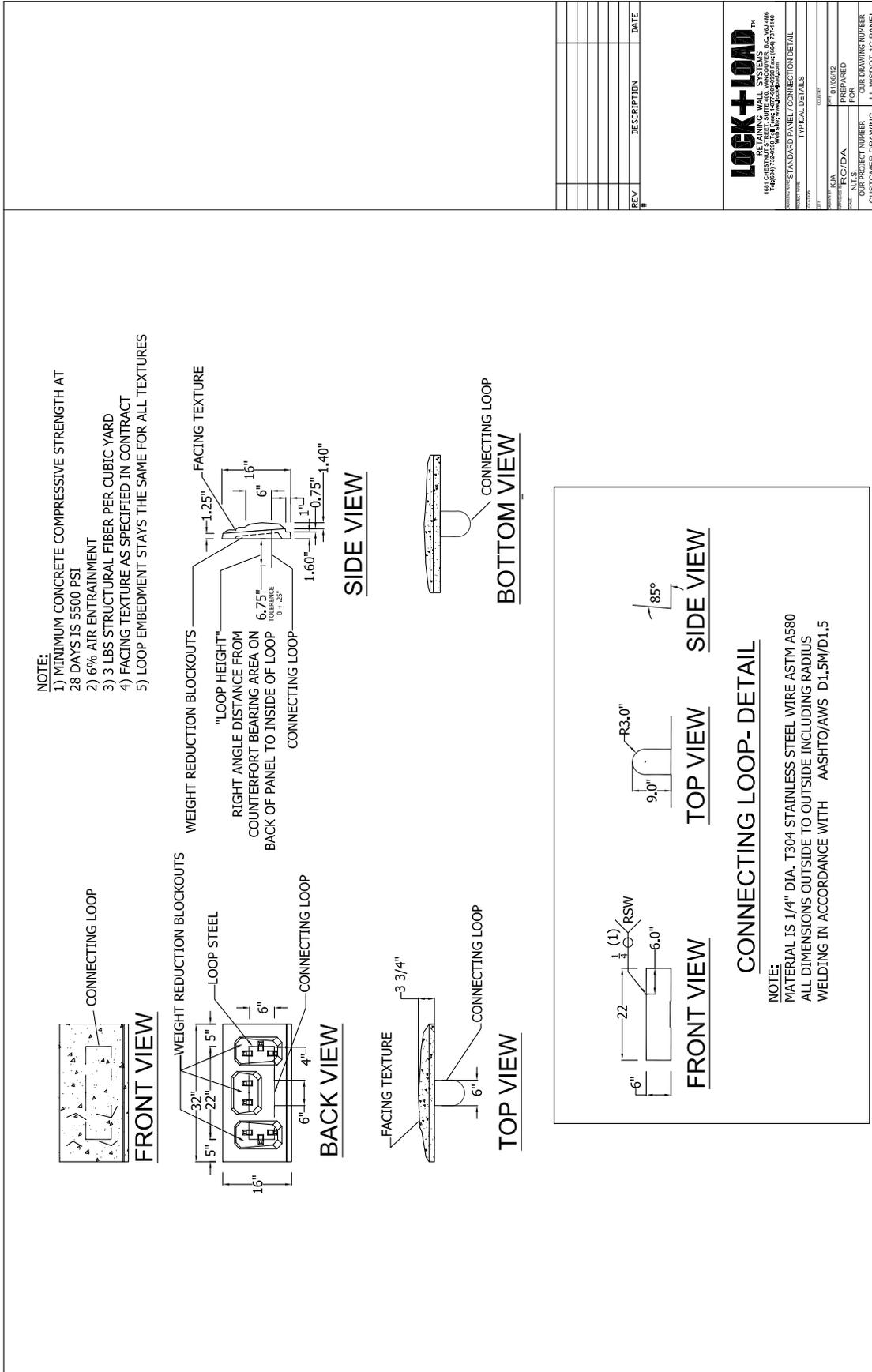
The Lock and Load Wall system shall only be used at locations where the wall will be above the water table.

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).



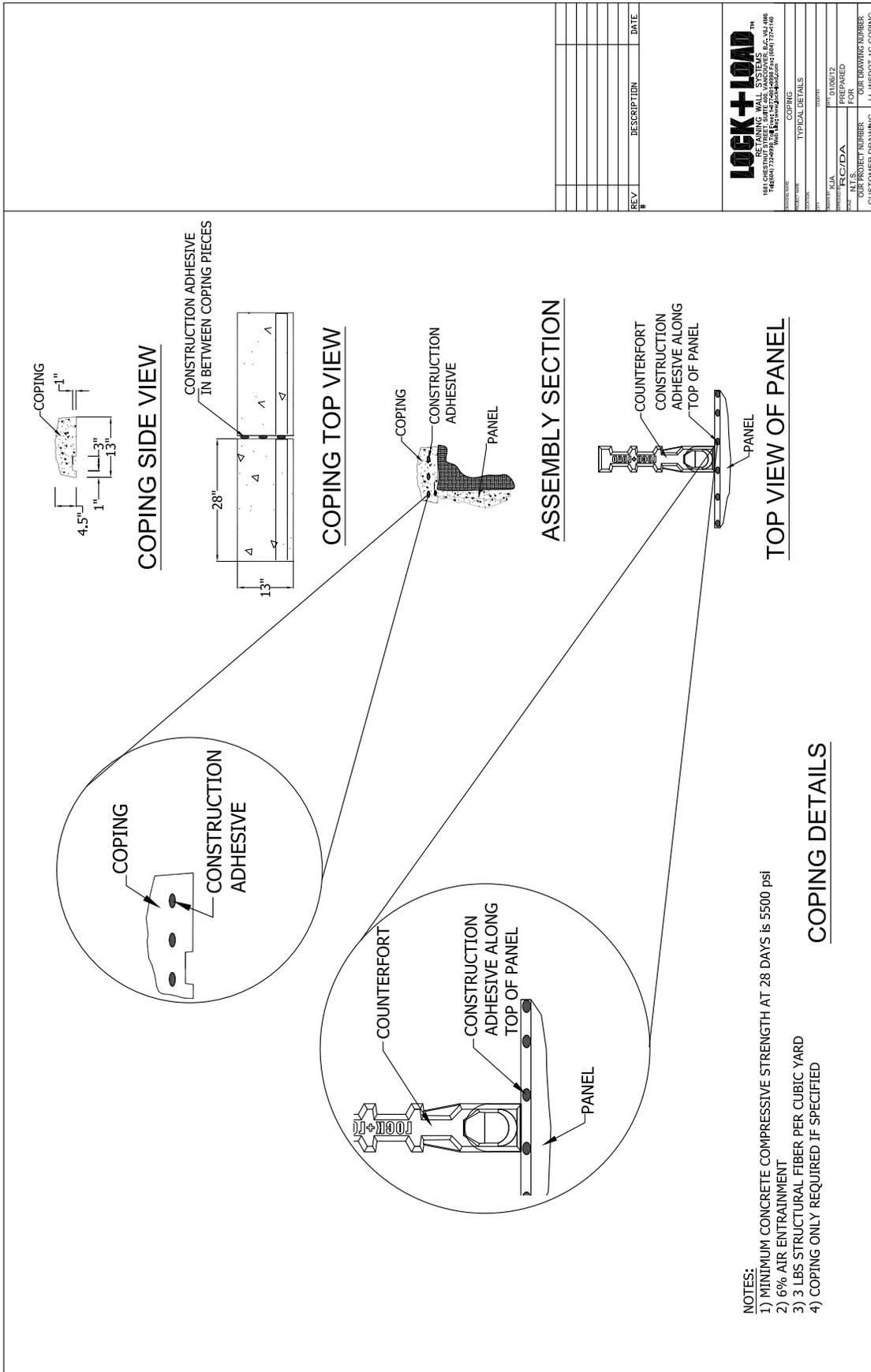






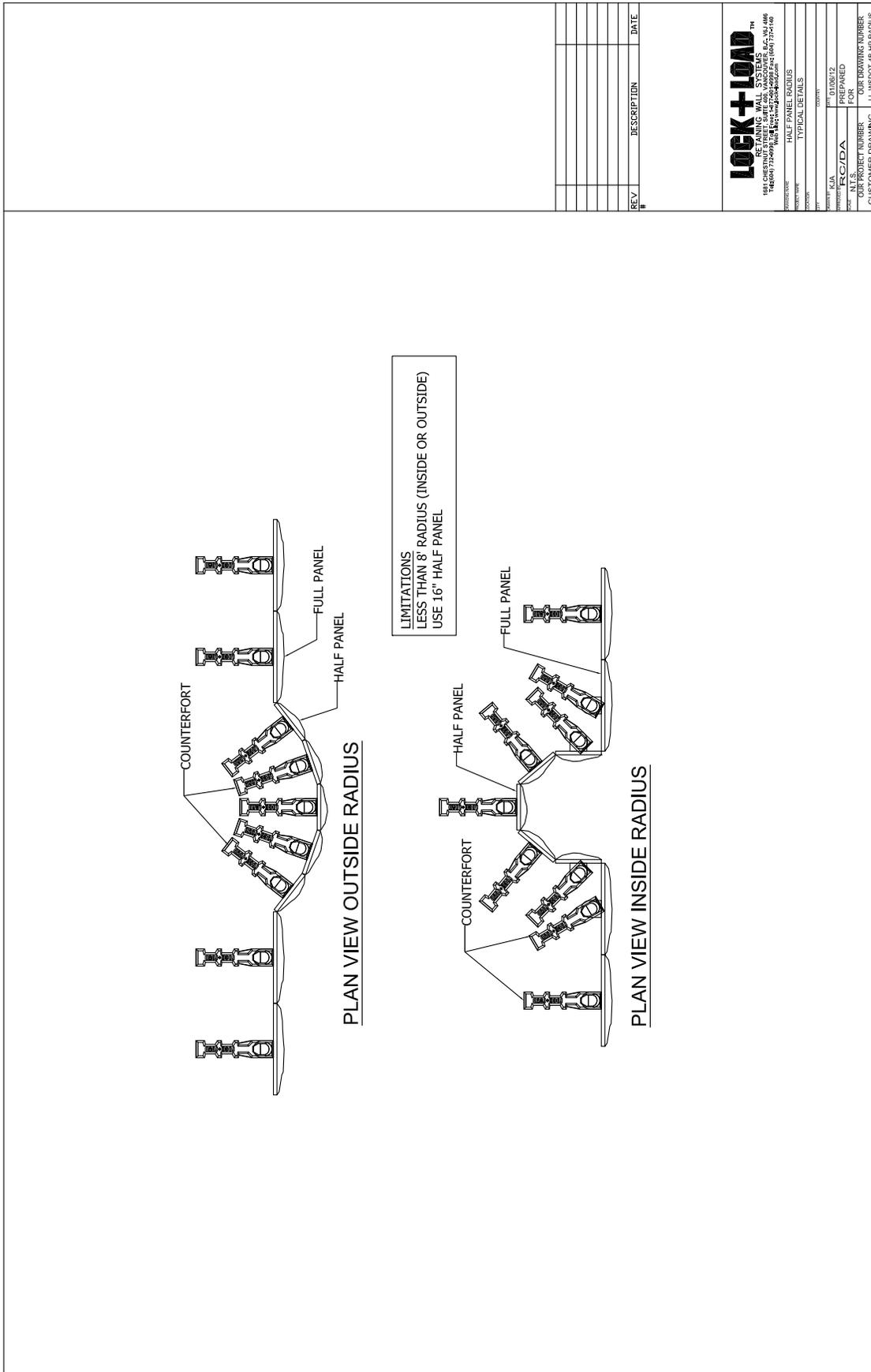


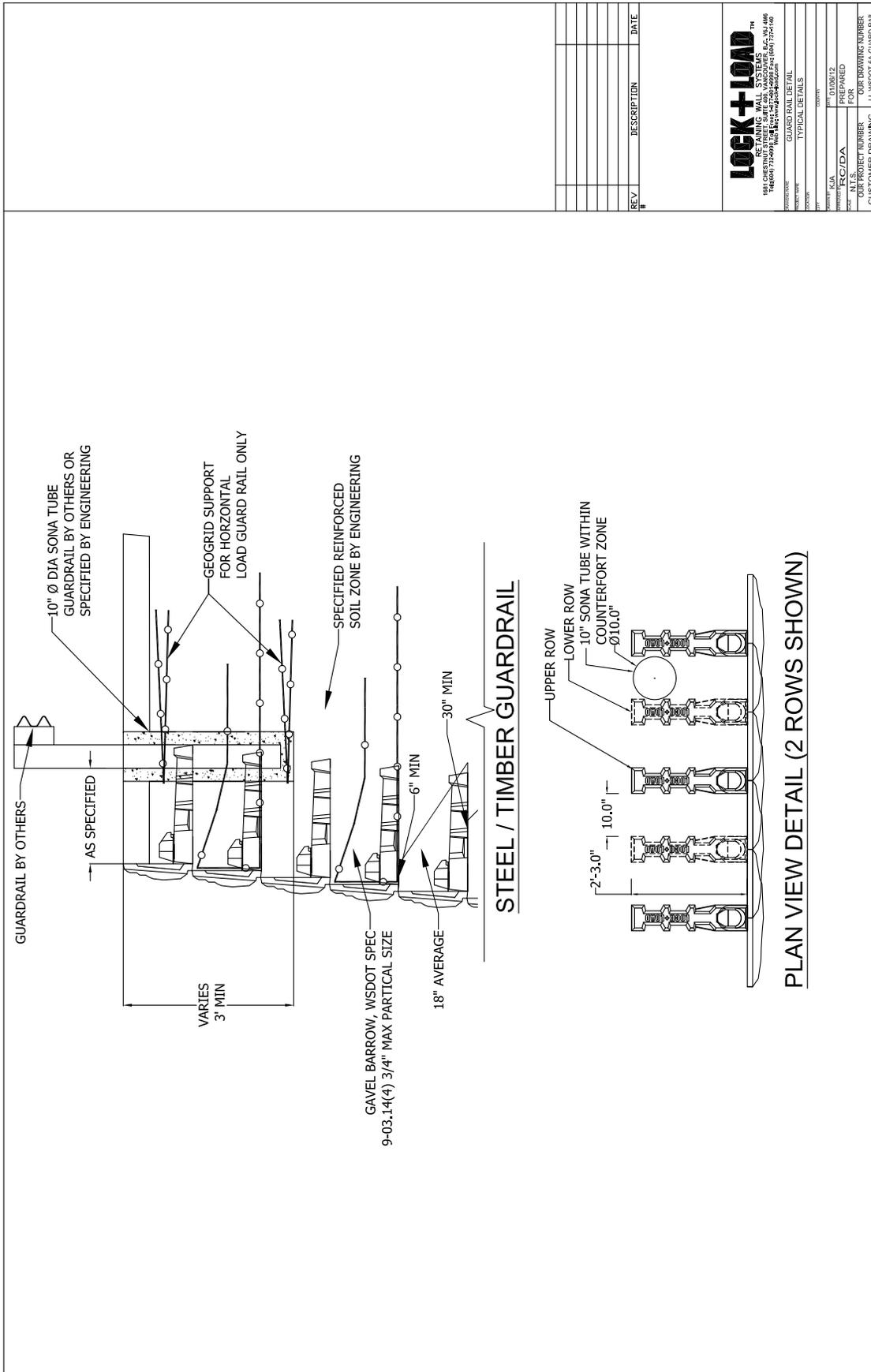












REV	DESCRIPTION	DATE

**LOCK+LOAD**  
INCORPORATING THE  
 BEST PRACTICES OF  
 THE INDUSTRY

1881 CHRISTIAN AVENUE, SUITE 200  
 TACOMA, WA 98403  
 TEL: (253) 724-9100 FAX: (253) 724-9101  
 WWW.LOCKANDLOAD.COM

PROJECT: GUARD RAIL DETAIL  
 DRAWING NO.: TYPICAL DETAILS  
 DATE:  

DESIGNED BY:  

CHECKED BY:  

APPROVED BY:  

DATE PREPARED:  

DATE FOR:  

PROJECT NUMBER:  

CUSTOMER DRAWING NUMBER:  

CUSTOMER DRAWING:  

LOCK+LOAD WSDOT 56-GUARD RAIL



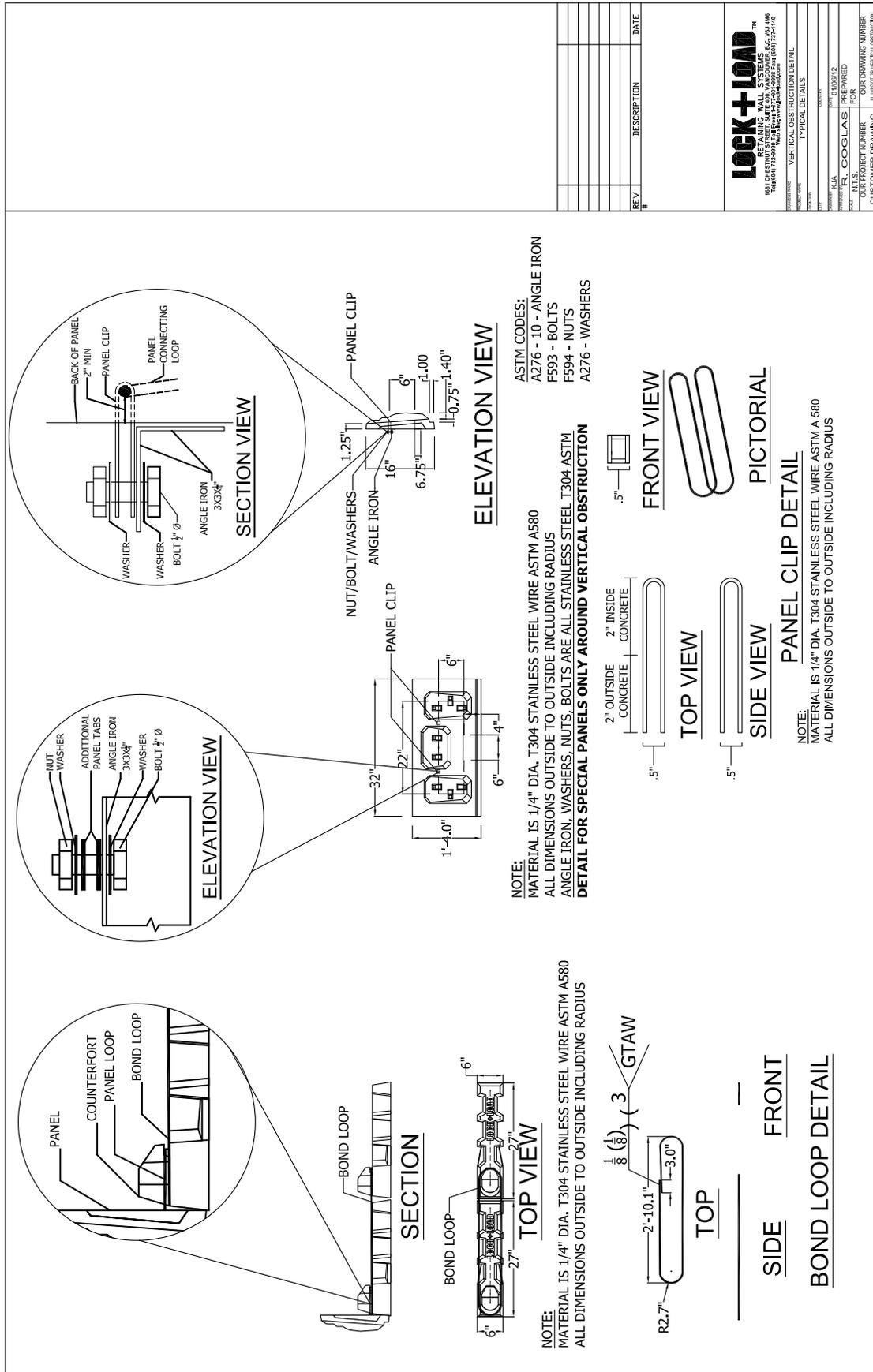










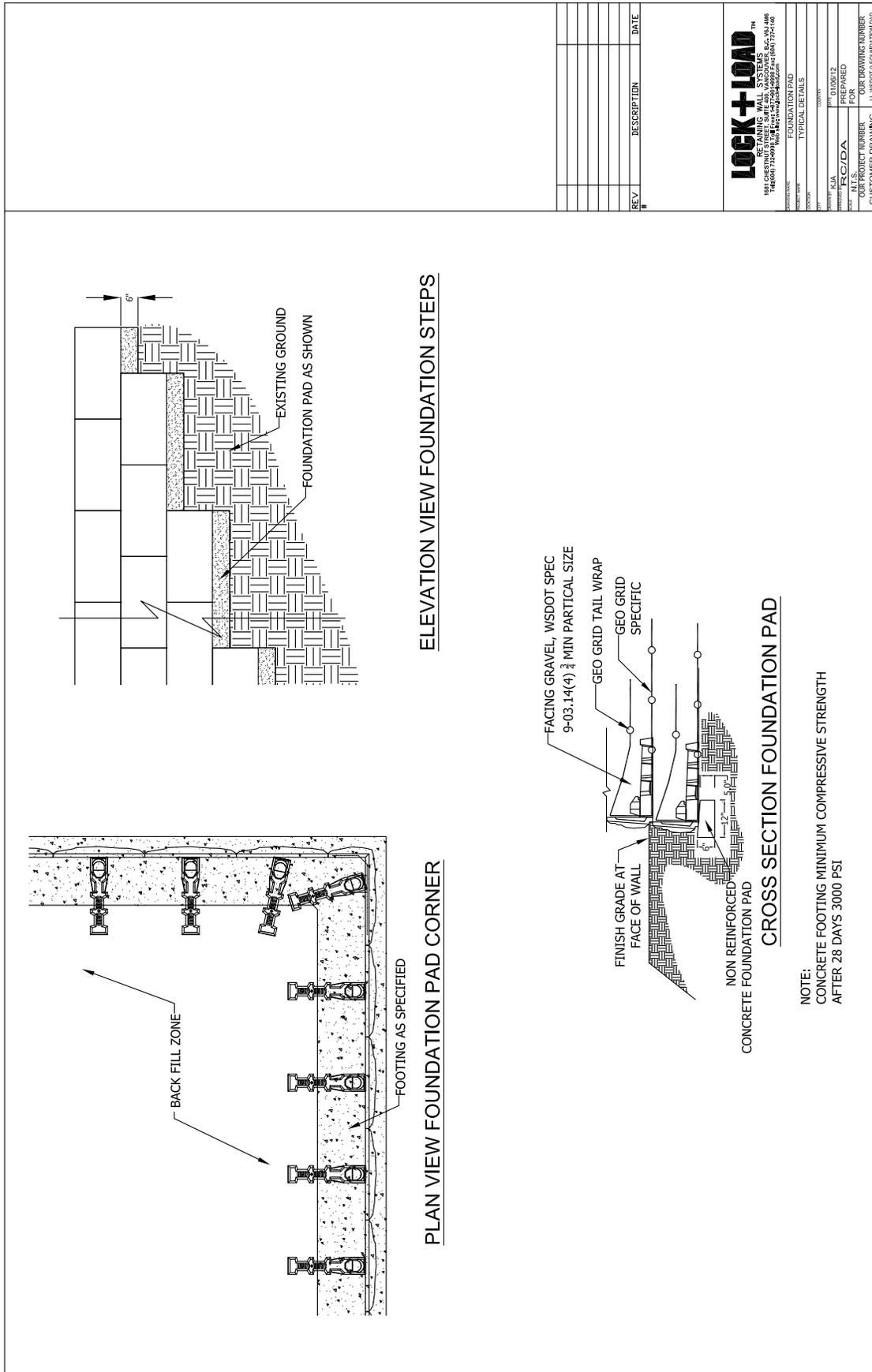


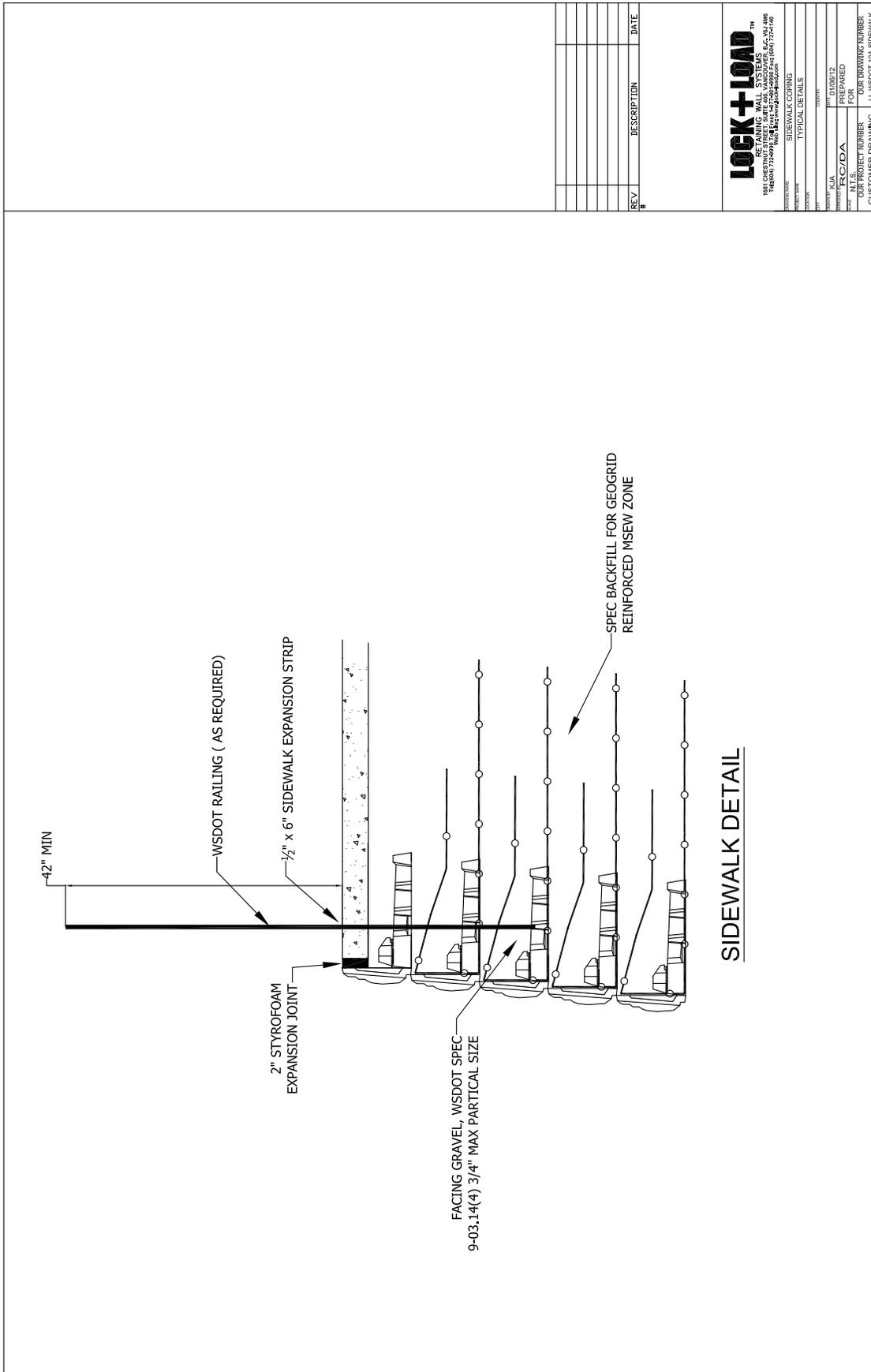
REV	DESCRIPTION	DATE

**LOCK+LOAD**  
 181 CHRISTIAN AVENUE  
 WEST PALM BEACH, FL 33411  
 TEL: 561.833.1100  
 FAX: 561.833.1101  
 WWW.LOCKANDLOAD.COM

PROJECT: VERTICAL OBSTRUCTION DETAIL  
 DRAWN BY: [ ]  
 CHECKED BY: [ ]  
 DATE: [ ]  
 FOR: R. COGLIASI  
 CUSTOMER DRAWING NUMBER: [ ]  
 CUSTOMER PROJECT NUMBER: [ ]  
 CUSTOMER DRAWING TITLE: [ ]



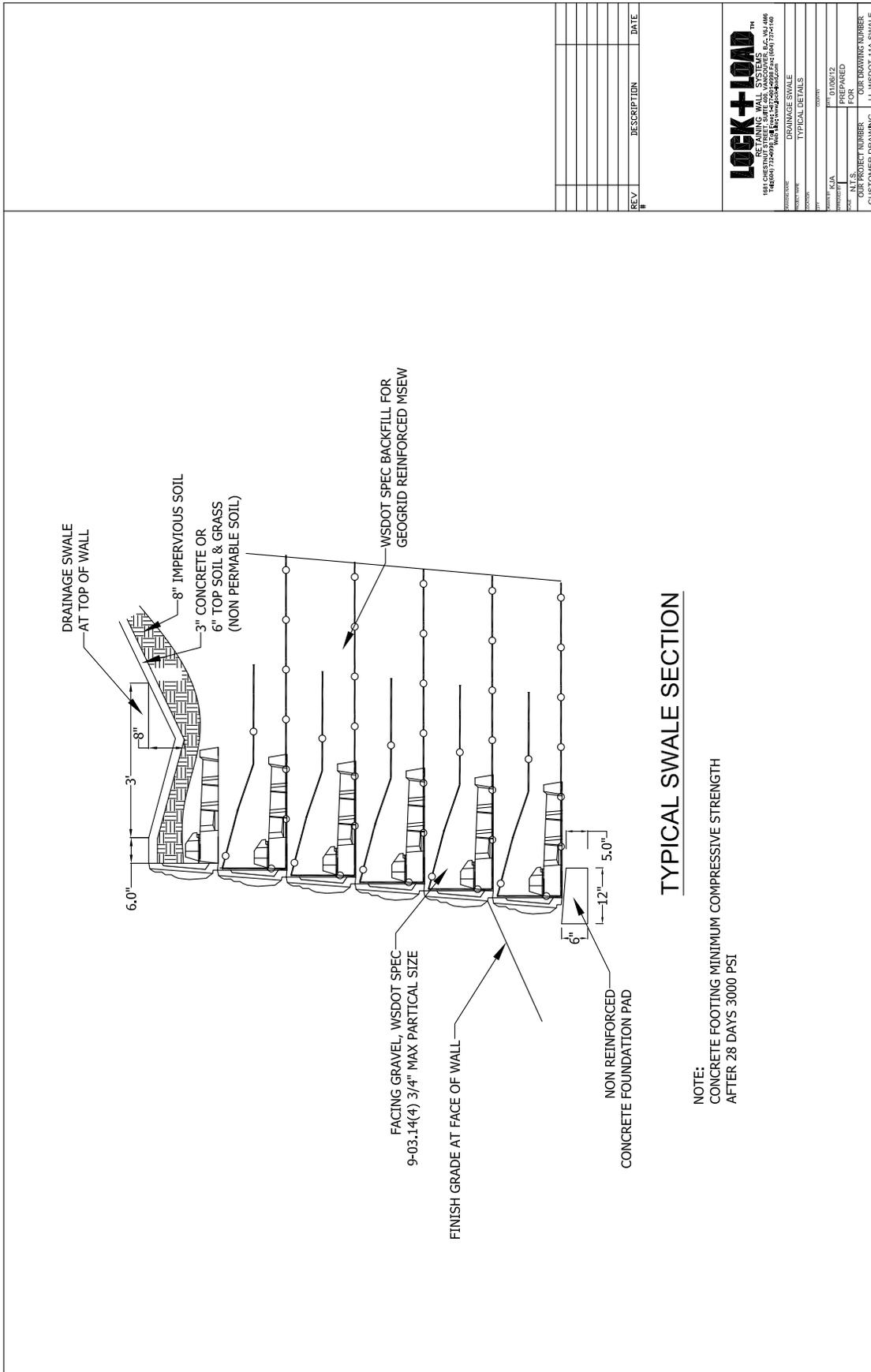




REV	DESCRIPTION	DATE

**LOCK+LOAD**  
 THE  
 PREPARED BY: WSDOT  
 1881 CHRISTY AVENUE, SUITE 200  
 TACOMA, WA 98403-1000  
 TEL: (253) 734-6000 FAX: (253) 734-6001

PROJECT: SIDEWALK CORING  
 DRAWING: TYPICAL DETAILS  
 SHEET: 15-P-01  
 DATE: 10/13/13  
 PREPARED BY: R/C/D/A  
 CHECKED BY: N.L.S.  
 OUR PROJECT NUMBER: 00R DRAWING NUMBER: LL-WSDOT-10A-SIDEWALK  
 CUSTOMER DRAWING: LL-WSDOT-10A-SIDEWALK



**TYPICAL SWALE SECTION**

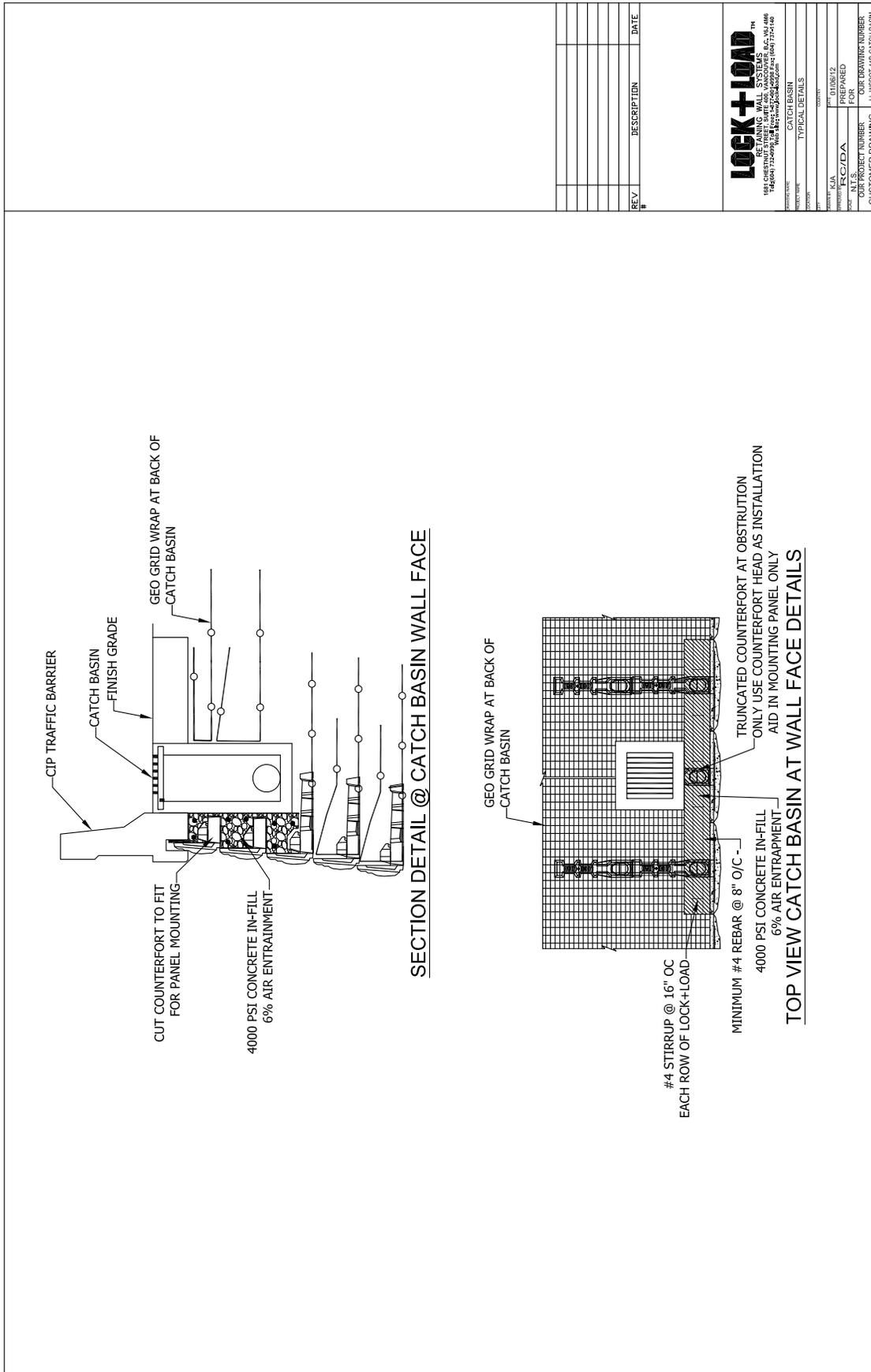
NOTE:  
CONCRETE FOOTING MINIMUM COMPRESSIVE STRENGTH  
AFTER 28 DAYS 3000 PSI

REV	DESCRIPTION	DATE

**LOCK+LOAD**  
 THE  
 PREPARED BY: [ ]  
 1881 CHRISTIAN AVENUE, SUITE 200, WISCONSIN, WI 53090  
 TEL: (262) 734-9900 FAX: (262) 734-9901  
 WWW.LOCKANDLOAD.COM

PROJECT: [ ]  
 DRAWING NUMBER: [ ]  
 CUSTOMER DRAWING: [ ]

DATE: [ ]  
 DRAWN BY: [ ]  
 CHECKED BY: [ ]  
 PREPARED BY: [ ]  
 FOR: [ ]



REV.	DESCRIPTION	DATE

**LOCK+LOAD**  
 RETAINING WALL SYSTEMS  
 181 CHESTER AVENUE, SUITE 100  
 WASHINGTON, DC 20005-4202  
 TEL: 202-775-1100 FAX: 202-775-1101  
 WWW.LOCKANDLOAD.COM

PROJECT NAME: CATCH BASIN  
 DRAWING NUMBER: TYPICAL DETAILS  
 SHEET NUMBER: 15-1000-1  
 DATE: 01/08/12  
 PREPARED BY: JCA  
 CHECKED BY: NLS  
 OUR PROJECT NUMBER: 14-1000-1-1  
 CUSTOMER DRAWING: LANSDDP-CATCH BASIN





# Preapproved Wall Appendix: Specific Requirements and Details for SSL Concrete Panel Walls

## Appendix 15-Q

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,  $F_y$ , 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  $F_y$  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  $CR_u$  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 Plus™ 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.

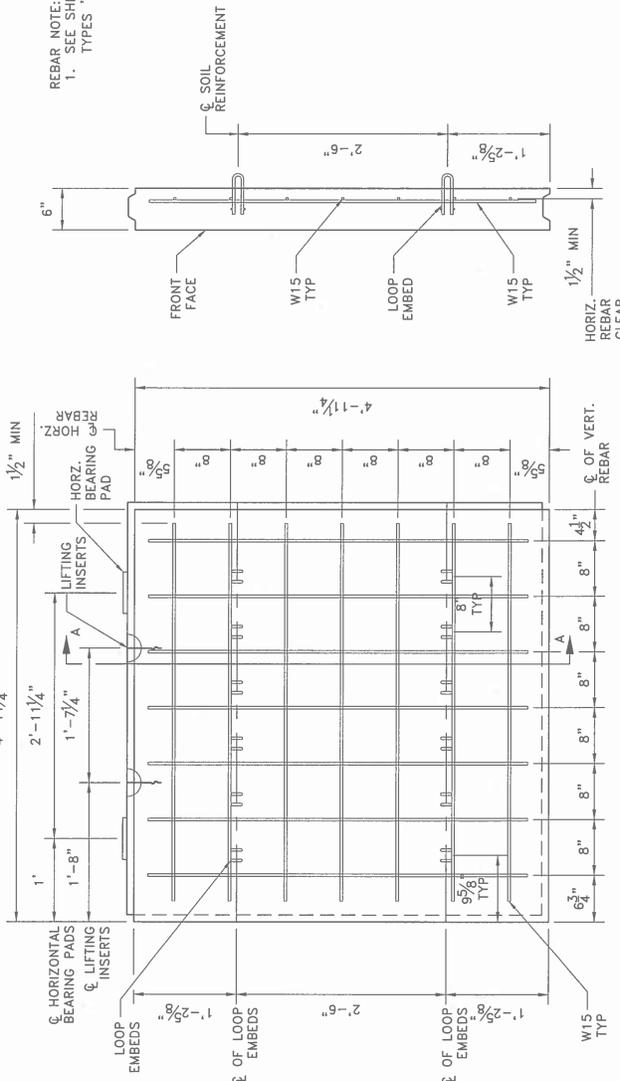


PANEL REINFORCEMENT TABLE

Panel Type	Minimum Vert. Area/ft <sup>2</sup>	Minimum Horiz. Area
A	0.88 in <sup>2</sup>	0.99 in <sup>2</sup>
A2	0.99 in <sup>2</sup>	0.55 in <sup>2</sup>
X	1.40 in <sup>2</sup>	1.40 in <sup>2</sup>
X2	1.40 in <sup>2</sup>	0.77 in <sup>2</sup>
Y	1.76 in <sup>2</sup>	1.10 in <sup>2</sup>
Y2	1.76 in <sup>2</sup>	0.55 in <sup>2</sup>

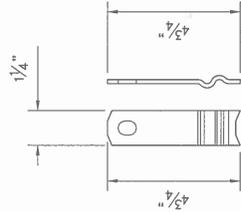
9" MAX SPACING BETWEEN BARS  
PANEL REINFORCEMENT MAY BE DEFORMED BARS, SMOOTH WIRE OR DEFORMED WIRE

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.

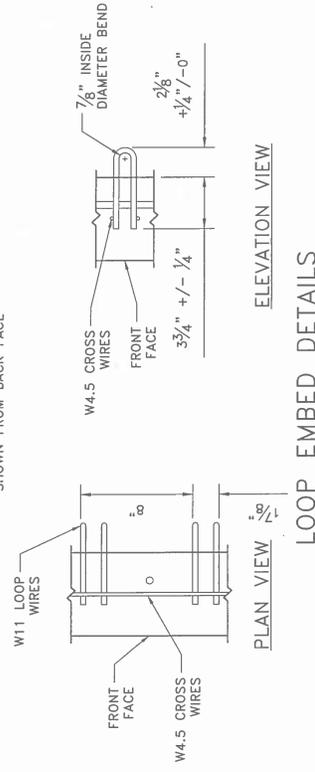
LIFTING INSERT DETAIL



SECTION A-A

STANDARD "A" PANEL

SHOWN FROM BACK FACE



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
  - 1/2" MAX. DIFFERENCE BETWEEN DIAGONALS

SURFACE FINISH:

- FINISH AT FRONT FACE
  - ± 1/8" IN 5'

CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES



DANIEL MITCHELL  
05/13/13

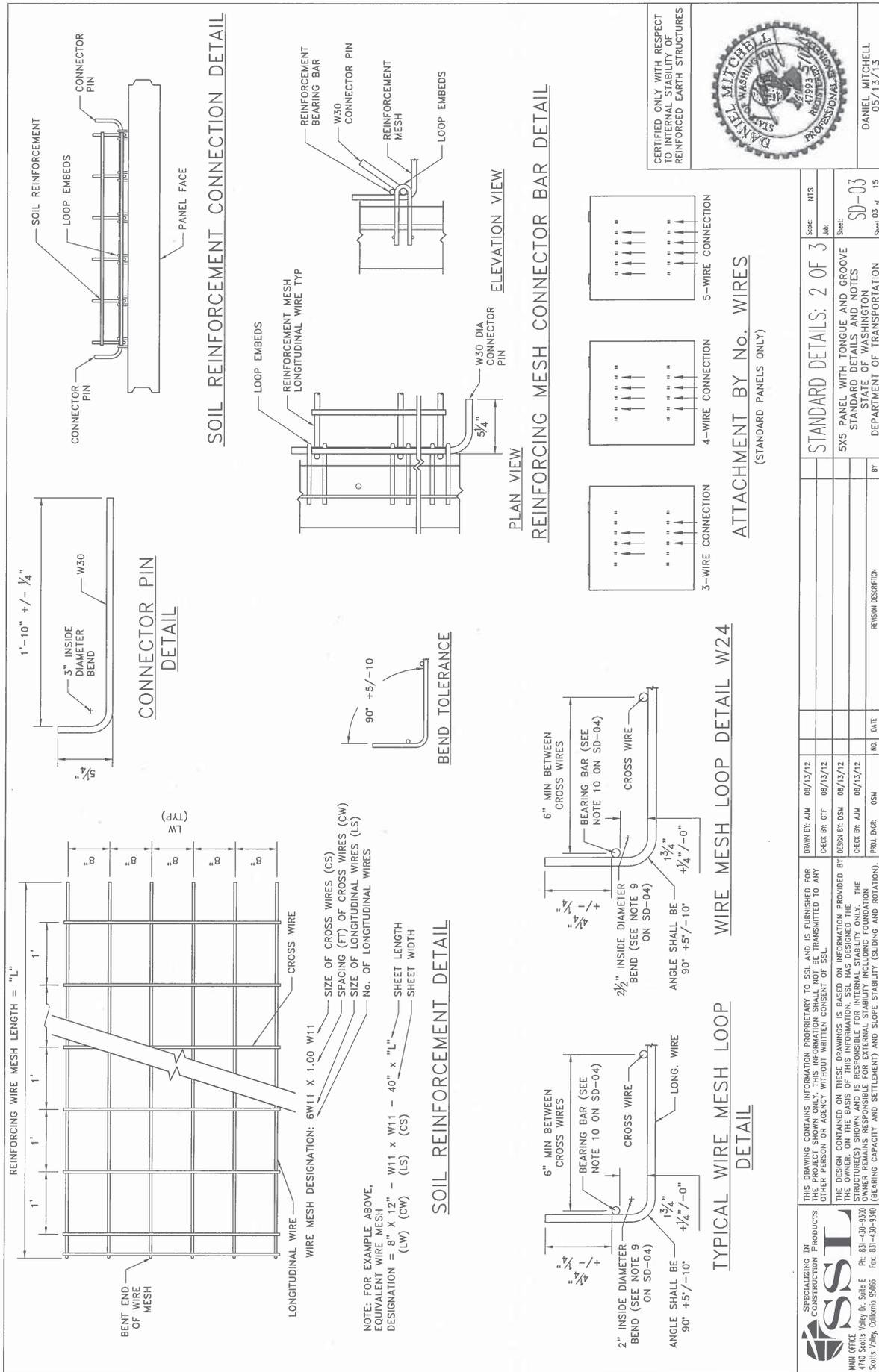
Scale: NTS  
Job: SD-02  
Sheet: 02 of 15

STANDARD DETAILS: 1 OF 3  
5X5 PANEL WITH TONGUE AND GROOVE STANDARD DETAILS AND NOTES  
STATE OF WASHINGTON  
DEPARTMENT OF TRANSPORTATION

NO.	DATE	REVISION DESCRIPTION

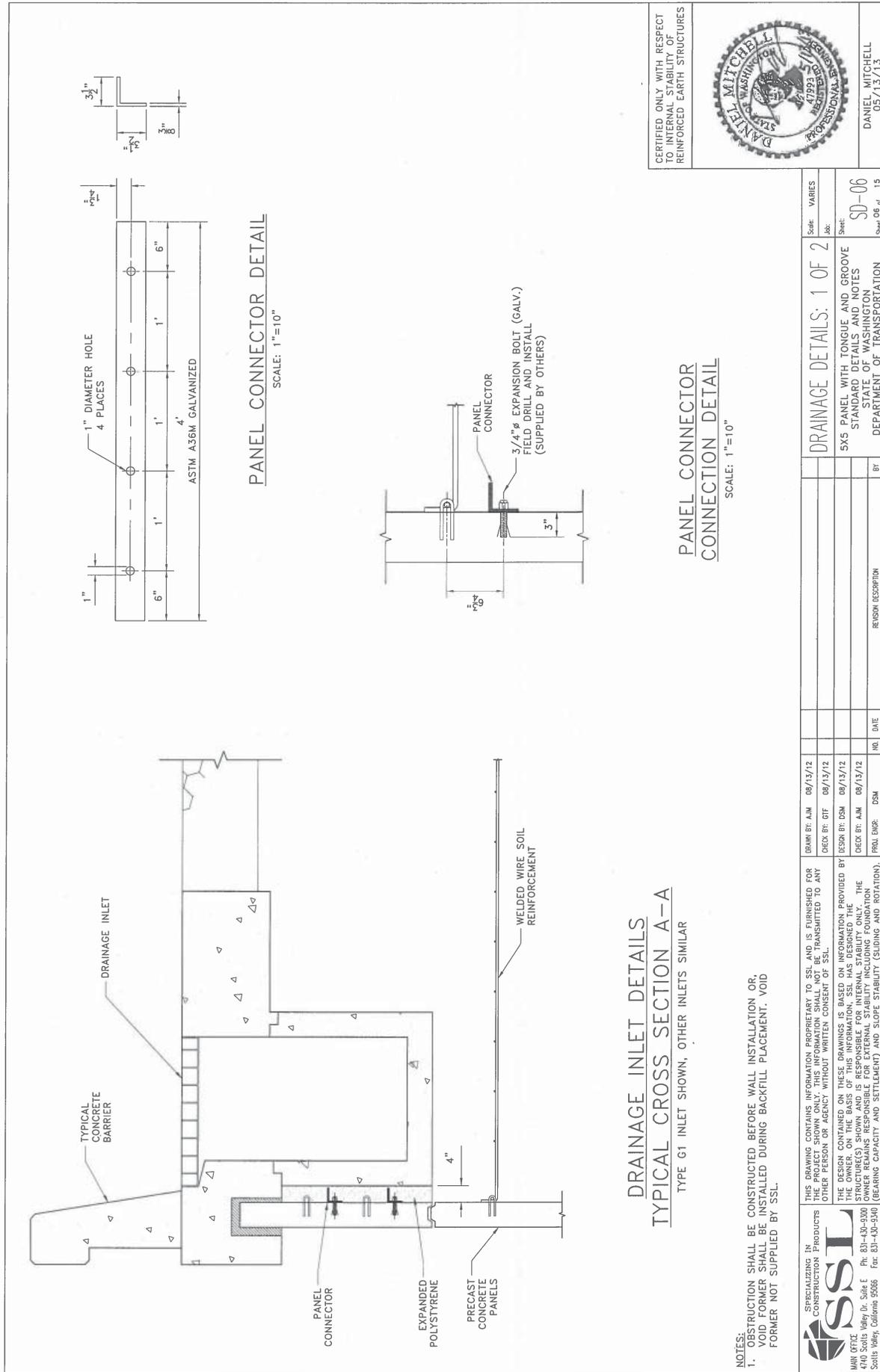
DATE	BY	DESCRIPTION
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08/13/12	CSM	CHECK BY
08/13/12	CSM	DESIGN BY
08/13/12	GTF	CHECK BY
08/13/12	AM	DESIGN BY

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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 USER OF THESE DRAWINGS SHALL BE RESPONSIBLE FOR EXTERNAL STABILITY, THE BEARING CAPACITY AND SETTLEMENT, AND SLOPE STABILITY (SLIDING AND ROTATION).  
SSL  
SPECIALIZING IN  
CONSTRUCTION PRODUCTS  
4740 Scotts Valley Dr., Suite E  
Scotts Valley, California 95666  
Ph: 831-439-3300  
Fax: 831-439-3340

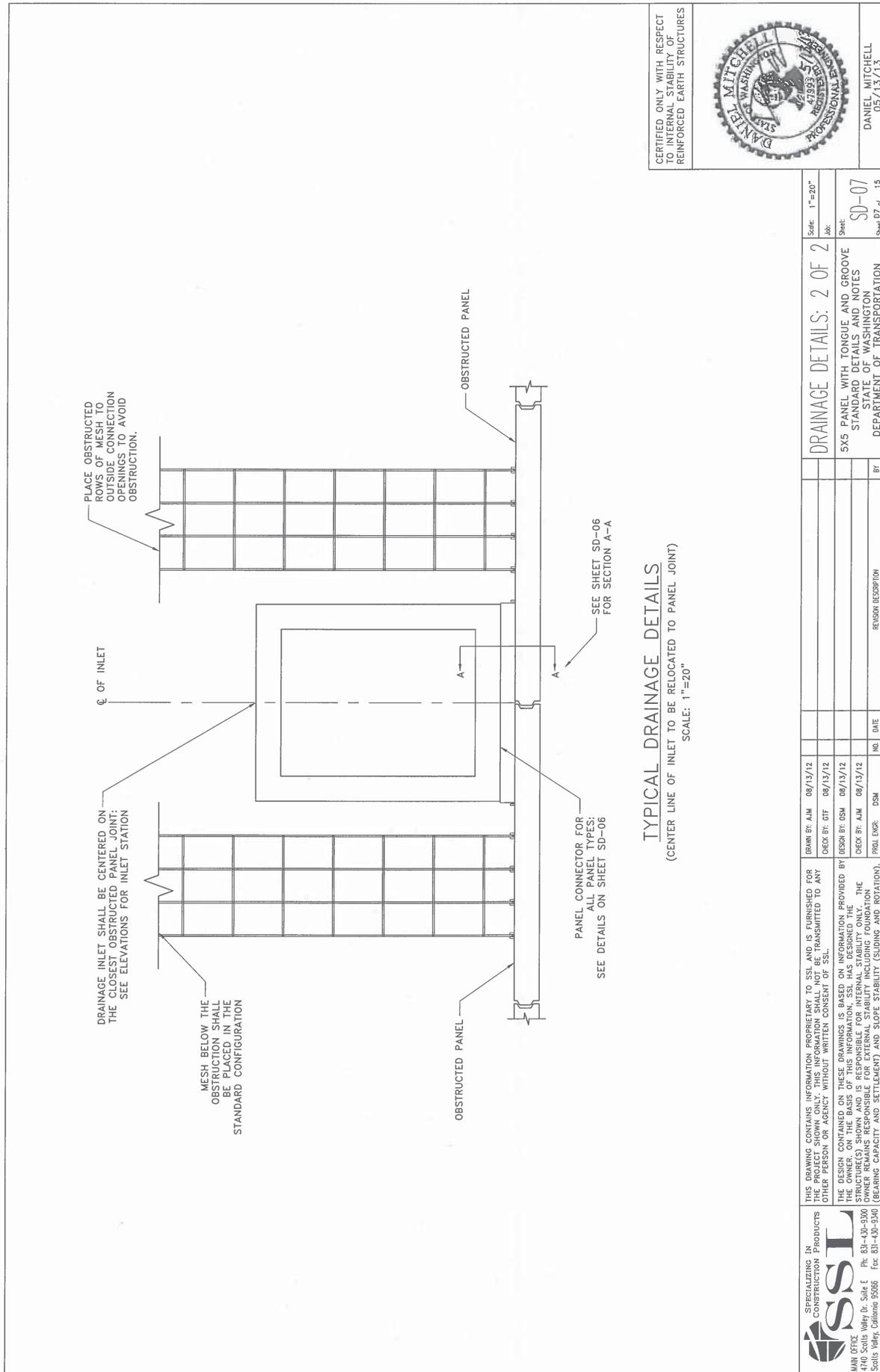








<p>Scale: VARIES</p> <p>Job: SD-06</p> <p>Sheet: 06 of 15</p>	<p>DRAINAGE DETAILS: 1 OF 2</p> <p>SX5 PANEL WITH TONGUE AND GROOVE STAIRCASE DETAIL, STATE OF WASHINGTON, DEPARTMENT OF TRANSPORTATION</p>
<p>DRUM BY: AM 08/13/12</p> <p>CHECK BY: GTF 08/13/12</p> <p>DESIGN BY: DSM 08/13/12</p> <p>CHECK BY: AM 08/13/12</p> <p>PROJ. ENGR. DSM</p>	<p>REASON DESCRIPTION</p>



CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES



DANIEL MITCHELL  
05/13/13

Scale: 1"=20"  
Job: DRAINAGE DETAILS: 2 OF 2  
Sheet: 50-07  
Sheet 07 of 15

5X5 PANEL WITH TONGUE AND GROOVE STANDARD DETAILS FOR WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

NO.	DATE	BY	REVISION DESCRIPTION

DESIGN BY: AJM	08/13/12		
CHECK BY: GTF	08/13/12		
DESIGN BY: DSM	08/13/12		
CHECK BY: AJM	08/13/12		
PROJECT: DSM			

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DESIGNED BY: AJM  
CHECKED BY: GTF  
PROJECT: DSM

PREPARED BY:  
SSL  
CONSTRUCTION PRODUCTS  
MAN OFFICE  
1740 Scotts Valley Dr., Suite E  
Scotts Valley, California 95066  
Ph: 831-430-9300  
Fax: 831-430-9340



**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.

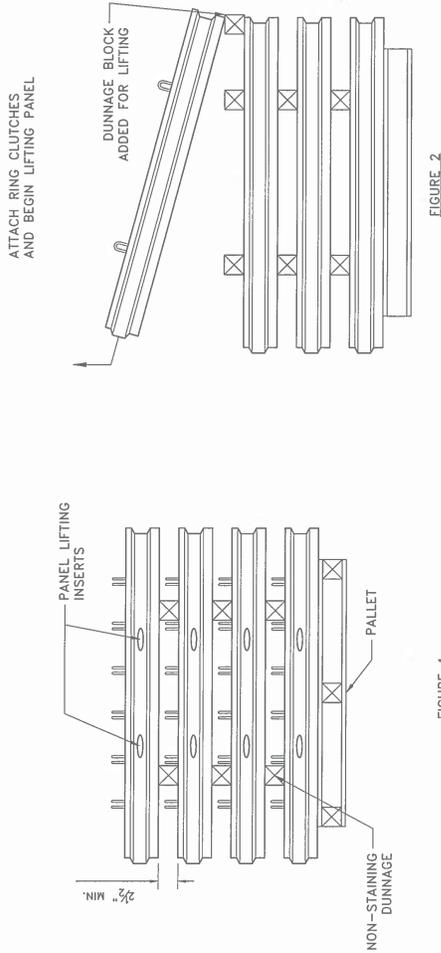


FIGURE 1

**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 THAN 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.

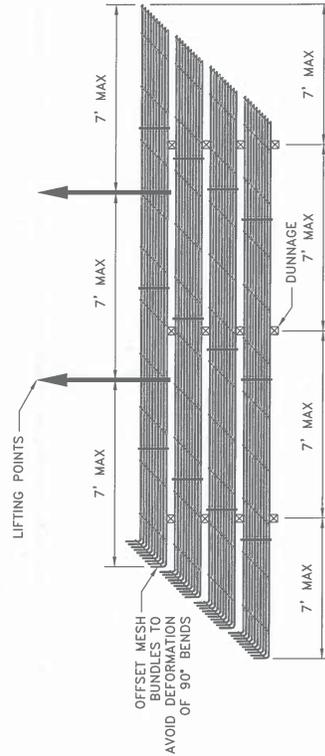


FIGURE 3

ATTACH RING CLUTCHES AND BEGIN LIFTING PANEL

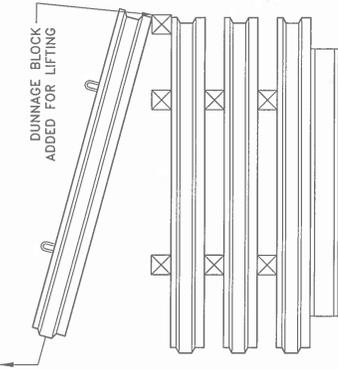


FIGURE 2

CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES



DANIEL MITCHELL  
05/13/13

<p><b>SSS</b> SPECIALTIES IN CONSTRUCTION PRODUCTS 4740 Scotts Valley Dr., Suite E Scotts Valley, California 95066 Ph: 831-430-9300 Fax: 831-430-9340</p>		<p>THIS DRAWING CONTAINS INFORMATION PROPRIETARY TO SSS AND IS FURNISHED FOR THE PROJECT SHOWN ONLY. THIS INFORMATION SHALL NOT BE TRANSMITTED TO ANY OTHER PERSON OR AGENCY WITHOUT WRITTEN CONSENT OF SSS.</p> <p>THE DESIGN CONTAINED ON THESE DRAWINGS IS BASED ON INFORMATION PROVIDED BY THE OWNER. ON THE BASIS OF THIS INFORMATION, SSS HAS DESIGNED THE STRUCTURE(S) SHOWN AND IS RESPONSIBLE FOR INTERNAL STABILITY ONLY. THE USER OF THESE DRAWINGS SHALL BE RESPONSIBLE FOR EXTERNAL STABILITY, OVERLOADING, BEARING CAPACITY AND SETTLEMENT, AND SLOPE STABILITY (SLIDING AND ROTATION).</p>	<p>DESIGN 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</p>	<p>NO. DATE BY</p>	<p>REASON DESCRIPTION</p>	<p>DEPARTMENT OF TRANSPORTATION</p>	<p>STATE OF WASHINGTON</p>	<p>NOTES 5X5 PANEL WITH TONGUE AND GROOVE STAIRS SEE SHEET 50-09 FOR DETAILS</p>	<p>Scale: NTS Job: 50-09 Sheet 08 of 15</p>
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6.0 ERECTION SEQUENCE

SITE PREPARATION  
 THE MSE PLUS™ WALL IS TO BE PLACED SHOULD BE GRADED LEVEL, THE AREA WHERE THE WALL IS TO BE PLACED SHOULD BE GRADED TO MEET 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  
 CONSTRUCTION OF THE LEVELING PADS ALONG THE LINES AND GRADES SHOWN ON SSL'S DRAWINGS SHALL BE COMPLETED PRIOR TO THE SETTING OF THE CONCRETE. 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 SHOULD BE THE STARTING POINT. IF NEITHER OF THESE ARE THE CASE, THEN BEGIN THE ERECTION ON THE FACING WITH THE LOWEST ELEVATION.

B) THE SECOND STEP IS TO ESTABLISH A FACE OF WALL LINE. THIS LINE SHOULD BE ESTABLISHED BY THE CONTRACTOR. THE CONTRACTOR SHALL BE RESPONSIBLE FOR ESTABLISHING REFERENCE POINTS FOR THIS WHICH 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 5/8" LEVEL AND BATTERED TO THE FILL SIDE APPROXIMATELY 5/8" IN 5 FEET. THIS BATTER SHALL BE MAINTAINED THROUGHOUT THE ERECTION OF THE WALL. THE BATTER 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 JOINTS. THE SPACING SHOULD BE BETWEEN 5/8" AND 3/4". THE TONGUE AND GROOVE JOINTS SHALL BE MEASURED TO INSURE ADEQUATE TIGHTNESS BETWEEN THE PANELS. THICKNESS OF THE PANELS, LEVEL AND CHECK THE HEIGHT BETWEEN THE PANELS.

E) ADJUST THE PANELS WITH WEDGES OR SPACERS TO ACHIEVE 43/8" 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 BRACING SHALL BE PLACED TO THE TOP OF THE PANELS. THE BRACING SHALL BE PLACED 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. THE BACKFILL SHALL BE PLACED TO 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 EMBOS IN THE PANEL. INSERT THE PIN CONNECTOR ROD THROUGH THE EMBOS, WINDOW AND INTO LOOPS. THE 3 EMBOS. REPEAT THIS STEP FOR THE OTHER 3 EMBOS.

K) THE NEXT LISTS OF BACKFILL ARE THEN PLACED (SEE 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 6" FROM THE BACK FACE IF NECESSARY TO PREVENT EXCESSIVE OVERTHROW WHEN COMPACTING THE BACKFILL.

E) INSTALL END AND TOP PANELS AS REQUIRED.

F) CONTINUE THE ABOVE PROCEDURES UNTIL ALL FACING PANELS ARE INSTALLED.

ERECTOR'S RESPONSIBILITY

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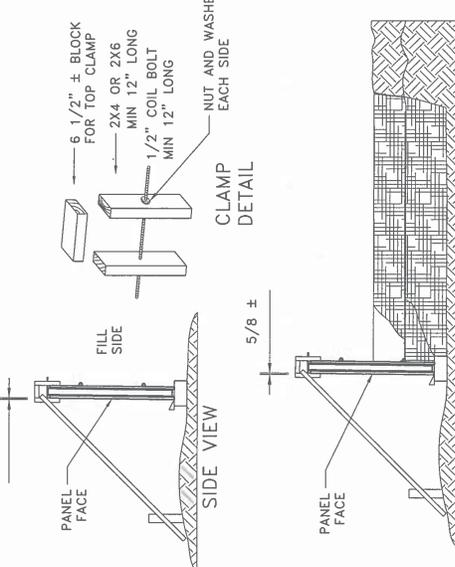


FIGURE 1

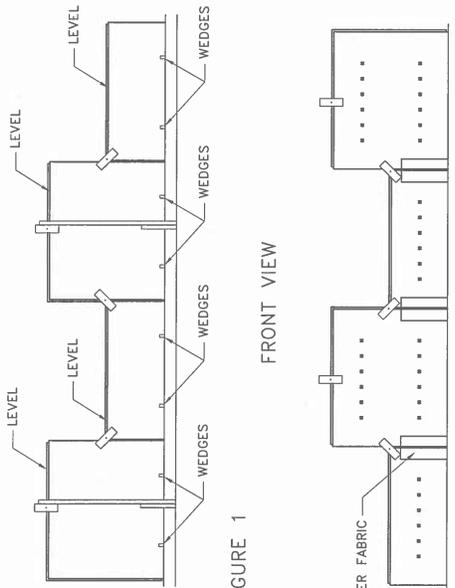


FIGURE 2

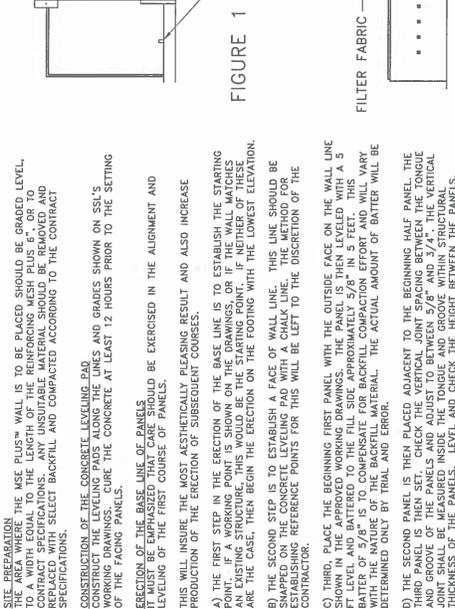


FIGURE 3

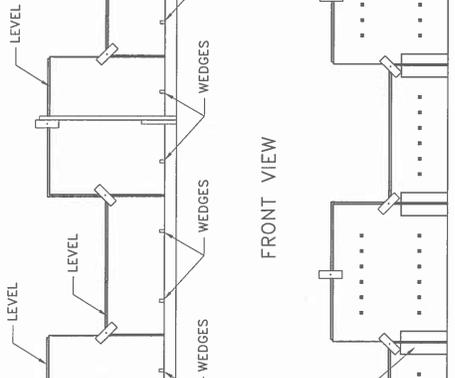


FIGURE 4

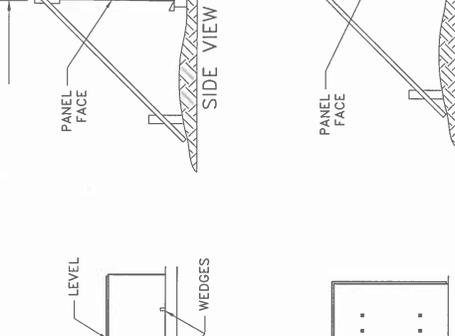


FIGURE 5

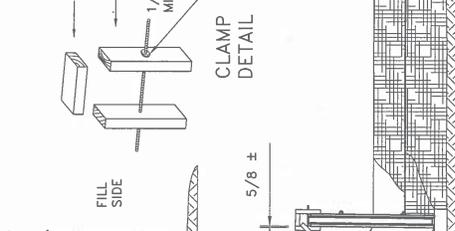


FIGURE 6



FIGURE 7

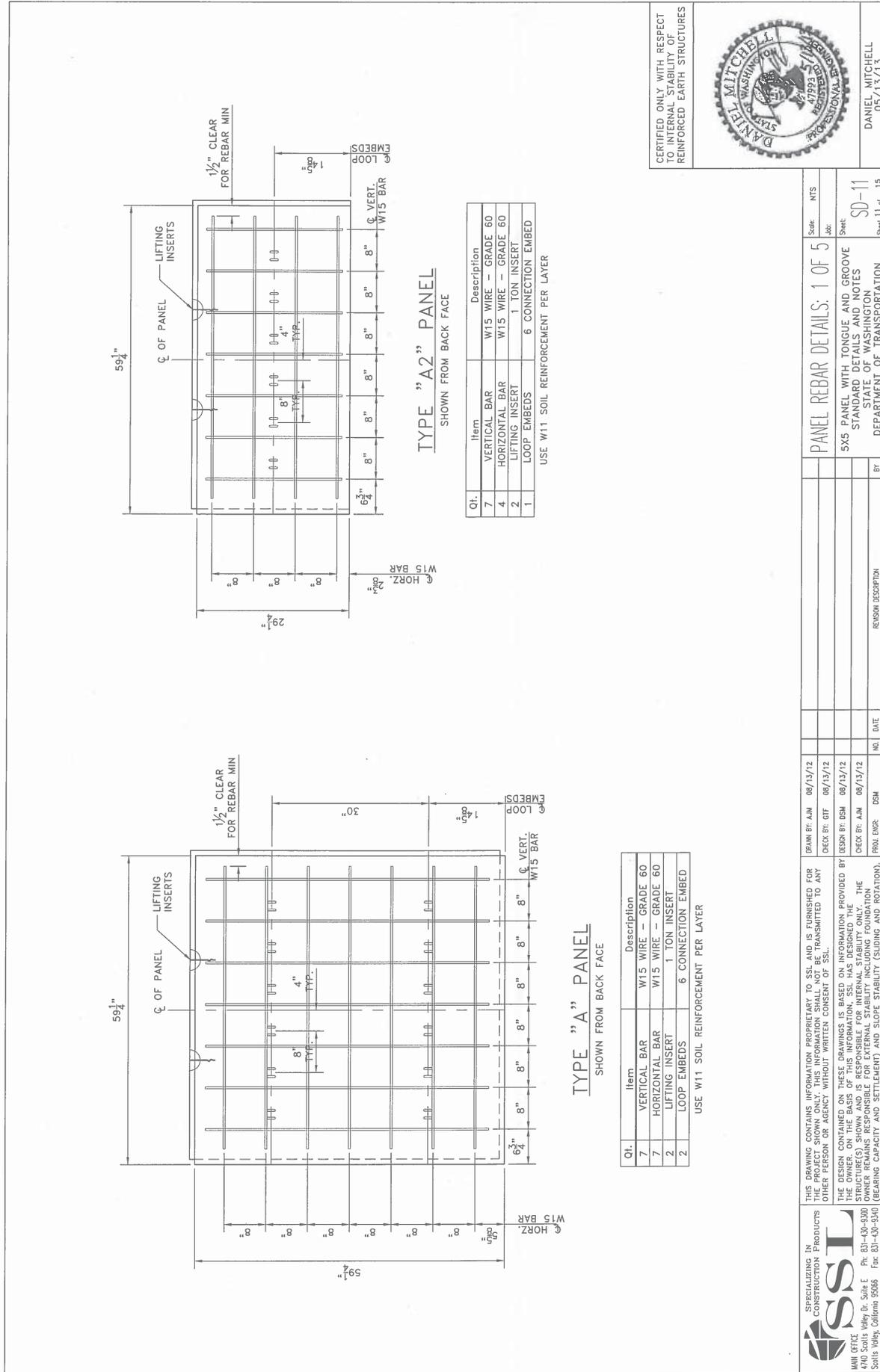
CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES

DANIEL MITCHELL  
 05/13/13

Scale:	NTS
Job:	
Sheet:	50-10
Sheet 10 of 15	

ERECTOR'S RESPONSIBILITY				
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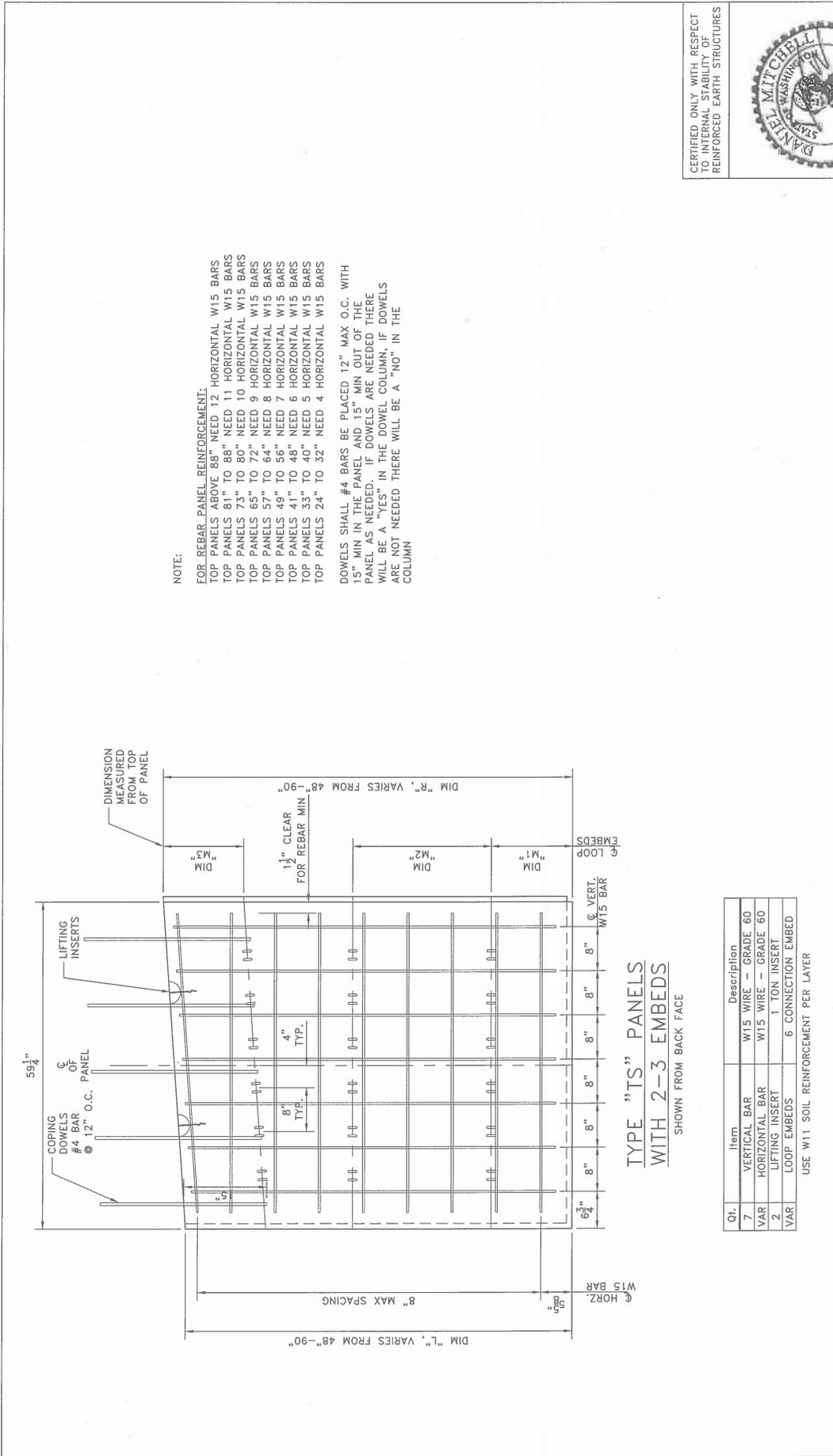
PANEL REBAR DETAILS: 1 OF 5  
Scale: NTS  
Job: SD-11  
Sheet: SD-11 of 15  
DEPARTMENT OF TRANSPORTATION

NO.	DATE	BY	REVISION DESCRIPTION

DESIGN BY: AM 08/13/12  
CHECK BY: GTF 08/13/12  
DESIGN BY: DSM 08/13/12  
CHECK BY: AM 08/13/12  
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OPERATIONS IN CONSTRUCTION PRODUCTS  
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**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

**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

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**SSL**  
 4740 Scotts Valley Dr., Suite E  
 Scotts Valley, California 95066  
 Ph: 831-430-9300  
 Fax: 831-430-9340

DESIGN BY: A.M.	08/13/12
CHECK BY: GTF	08/13/12
DESIGN BY: DSM	08/13/12
CHECK BY: A.M.	08/13/12
PROJ. ENGR: DSM	

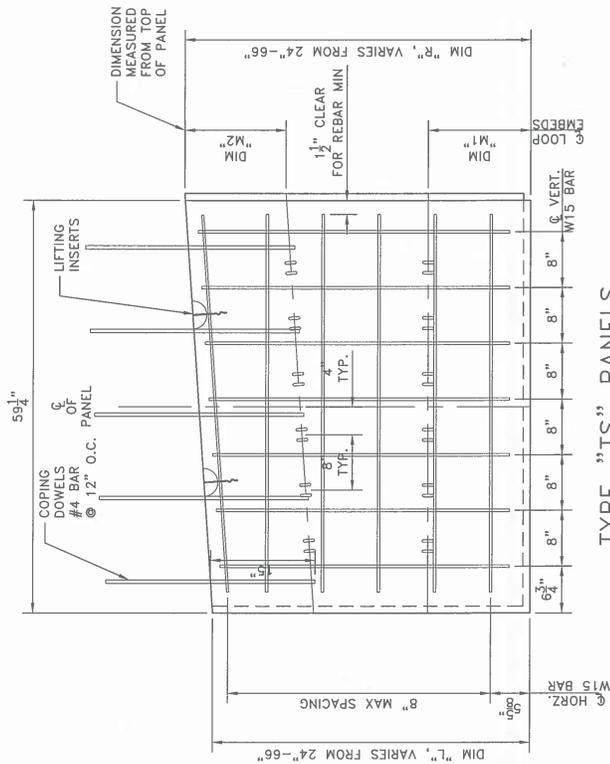
NO.	DATE	REVISION DESCRIPTION	BY

**PANEL REBAR DETAILS: 2 OF 5**  
 Scale: NTS  
 Job: SD-12  
 Sheet: SD-12  
 Sheet 12 of 15  
 DEPARTMENT OF TRANSPORTATION  
 STATE OF WASHINGTON  
 STANDARD DETAILS AND NOTES

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DANIEL MITCHELL  
 05/13/13



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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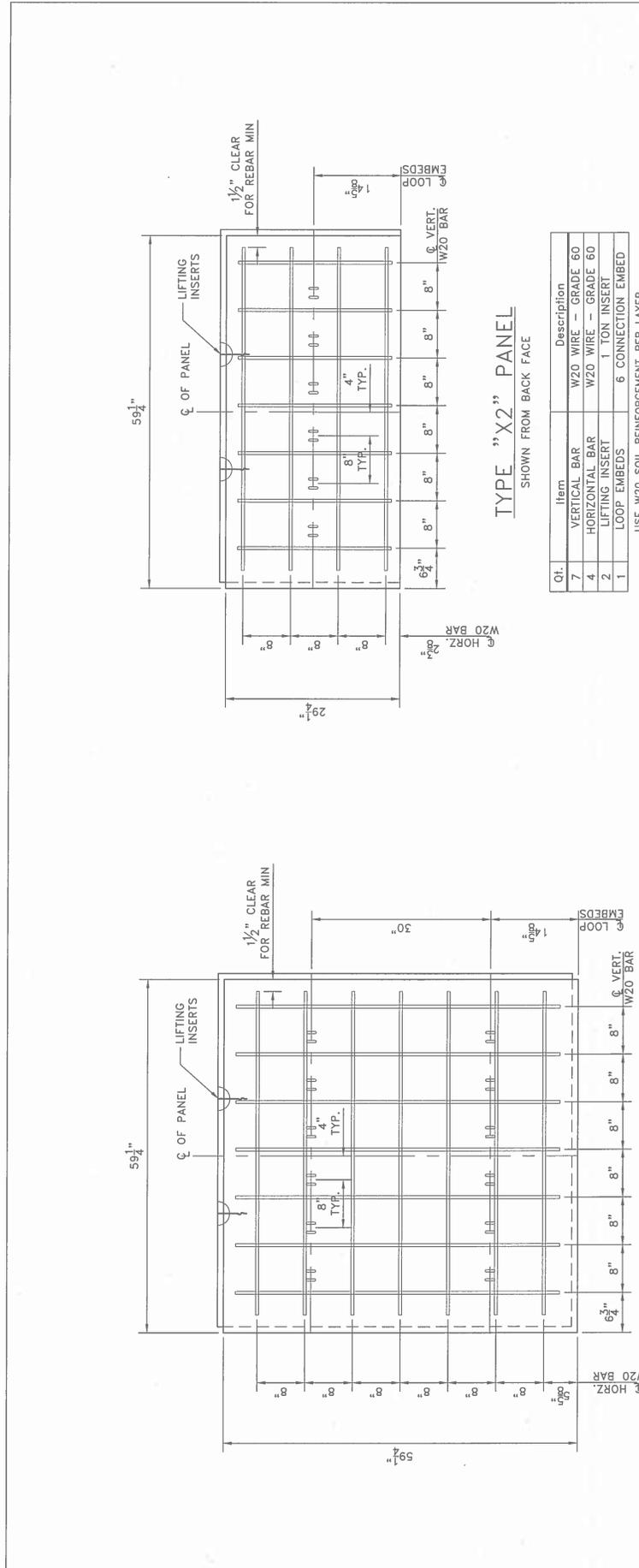
DANIEL MITCHELL  
05/13/15

Scale: NTS  
Job: SD-13  
Sheet: 13 of 15  
PANEL REBAR DETAILS: 3 OF 5  
5X5 PANEL WITH TONGUE AND GROOVE STAIRCASE WITH NOTES STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION

NO.	DATE	REVISION DESCRIPTION	BY

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 MINI OFFICE
   
 4740 Scotts Valley Dr. Suite E
   
 Scotts Valley, California 95066
   
 Pk: 831-432-9300
   
 Fax: 831-432-9340



TYPE "X" 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 "X2" 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

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05/13/15

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Scale: NTS  
Job: SD-14  
Sheet: 14 of 15

REVISION DESCRIPTION	DATE	NO.	BY

**PANEL REBAR DETAILS: 4 OF 5**

5X5 PANEL WITH TONGUE AND GROOVE STAIRCASE WASHINGTON STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION

SPECIALTIES IN CONSTRUCTION PRODUCTS

SSL  
4740 Scotts Valley Dr. Suite E  
Scotts Valley, California 95066  
Ph: 831-430-9300  
Fax: 831-430-9340





**MAIN OFFICE**  
 4740 Scotts Valley Drive, Suite "E"  
 Scotts Valley, California 95066  
 Telephone 831/430-9300  
 FAX 831/430-9340

**INDEX OF DRAWINGS**

SHEET	DESCRIPTION
DT-01	TITLE & INDEX
DT-02	MAXIMUM BATTER DETAILS
DT-03	CORNER DETAILS
DT-04	BURRED LAP JOINT LINE DETAILS
DT-05	TRUSS LARGER SYSTEMS
DT-06	OBSTRUCTION DETAILS
DT-07	CULVERT PENETRATION DETAILS
DT-08	LEVELING PAD DETAILS
DT-09	COPING SAMPLE DETAILS
DT-10	SLIP JOINT DETAILS



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 PROJ. ENGR:

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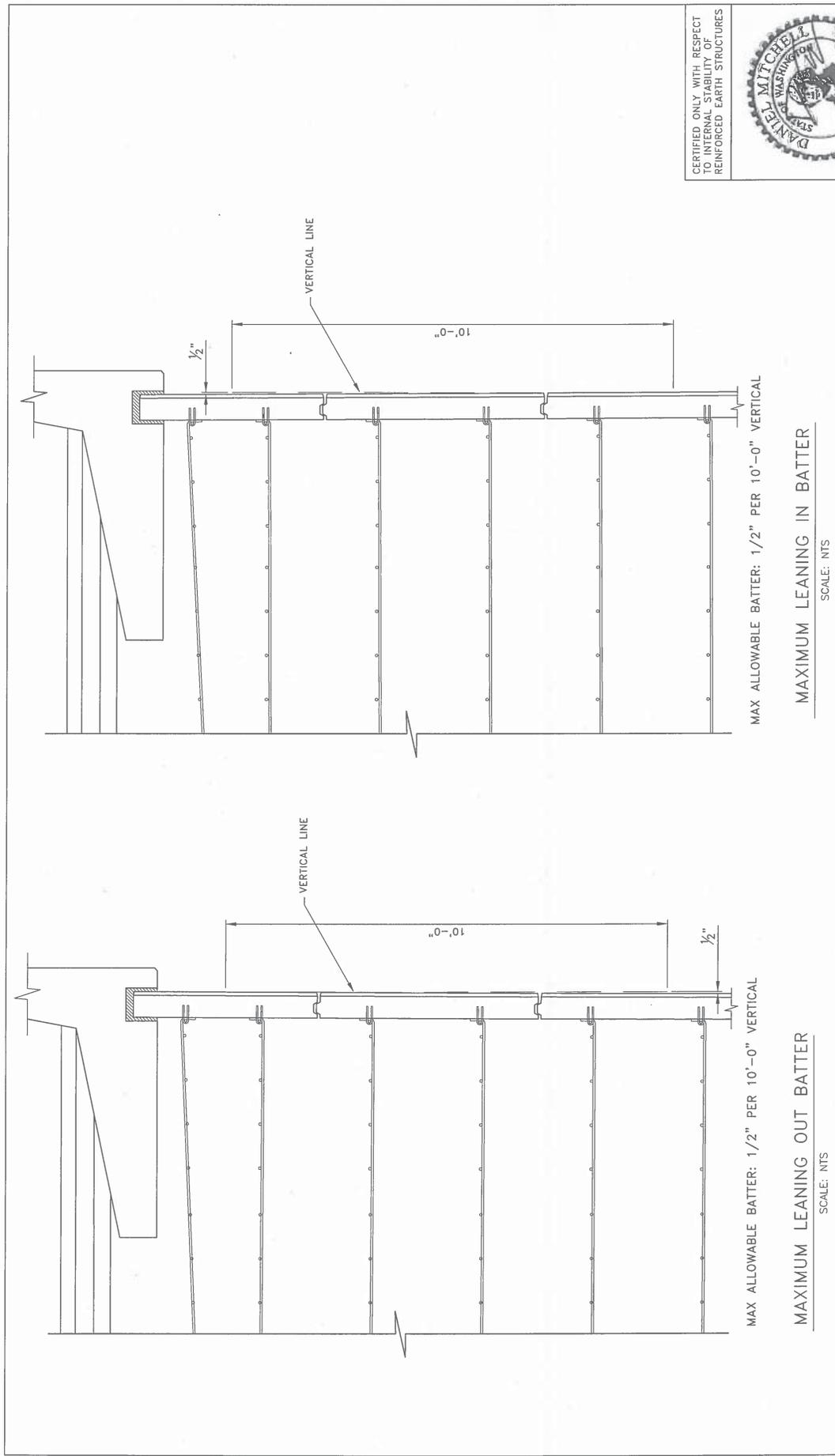
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 STATE OF WASHINGTON  
 DEPARTMENT OF TRANSPORTATION  
 SSL, LLC, USE PLUS  
 PANEL WALL SYSTEM

Scale: NTS  
 Job: ---  
 Sheet: DT-01  
 Sheet 01 of 10

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5/13/13

Scale: NTS	Job: ---	Sheet: DT-02	Sheet 02 of 10
<b>MAXIMUM BATTER DETAILS</b>			
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION SSL, LLC, MSE PLUS PANEL WALL SYSTEM			
DESIGN BY: AM	8/17/12	NO.	DATE
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**OBTUSE OUTSIDE CORNER**  
SCALE: NTS

**ORTHOGONAL OUTSIDE CORNER**  
SCALE: NTS

**ACUTE OUTSIDE CORNER**  
SCALE: NTS

**OBTUSE INSIDE CORNER**  
SCALE: NTS

**ORTHOGONAL INSIDE CORNER**  
SCALE: NTS

**ACUTE INSIDE CORNER**  
SCALE: NTS

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DATE: \_\_\_\_\_ NO. \_\_\_\_\_ BY \_\_\_\_\_

PROJ. ENGR: \_\_\_\_\_

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DESIGN BY: \_\_\_\_\_

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Job: \_\_\_\_\_

Sheet: DT-03

Sheet 03 of 10

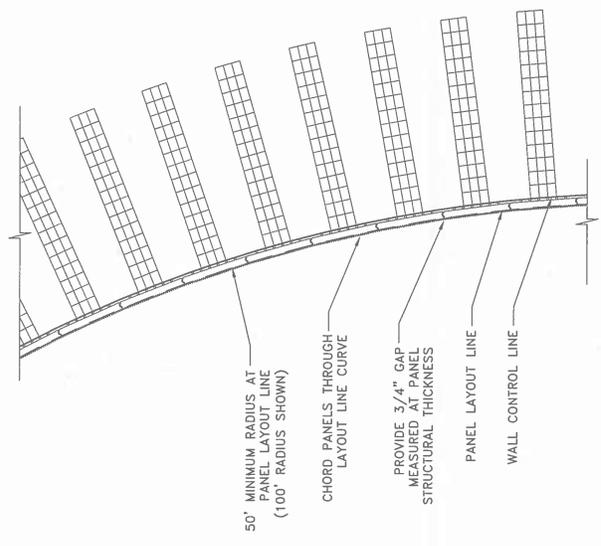
**CORNER DETAILS**

STATE OF WASHINGTON  
DEPARTMENT OF TRANSPORTATION  
SSL LLC, MSE PLUS  
PANEL WALL SYSTEM

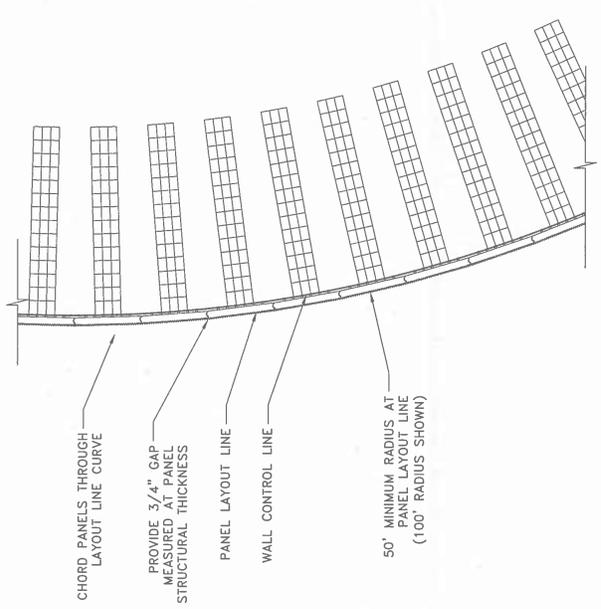
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5/13/13

- GENERAL NOTES:  
 1. ALL CURVED LOOPS SHOWN ARE APPROXIMATE. REFER TO WALL ELEVATIONS FOR MESH LENGTH AND TYPE. REFER TO PANEL ELEVATIONS FOR MESH NUMBER AND SPACING.  
 2. SEE OBSTRUCTION DETAIL FOR ADDITIONAL INFORMATION.  
 3. VERIFY LOCATIONS OF OBSTRUCTIONS.  
 4. VERIFY REINFORCEMENT MAY NOT BE SHOWN FOR CLARITY.  
 5. REFER TO WALL ELEVATIONS FOR ACTUAL FINISH TYPE USED.



OUTSIDE RADIUS LAYOUT DETAIL  
 SCALE: NTS



INSIDE RADIUS LAYOUT DETAIL  
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Scale: NTS  
 Job: ---  
 Sheet: DT-04  
 Sheet 04 of 10

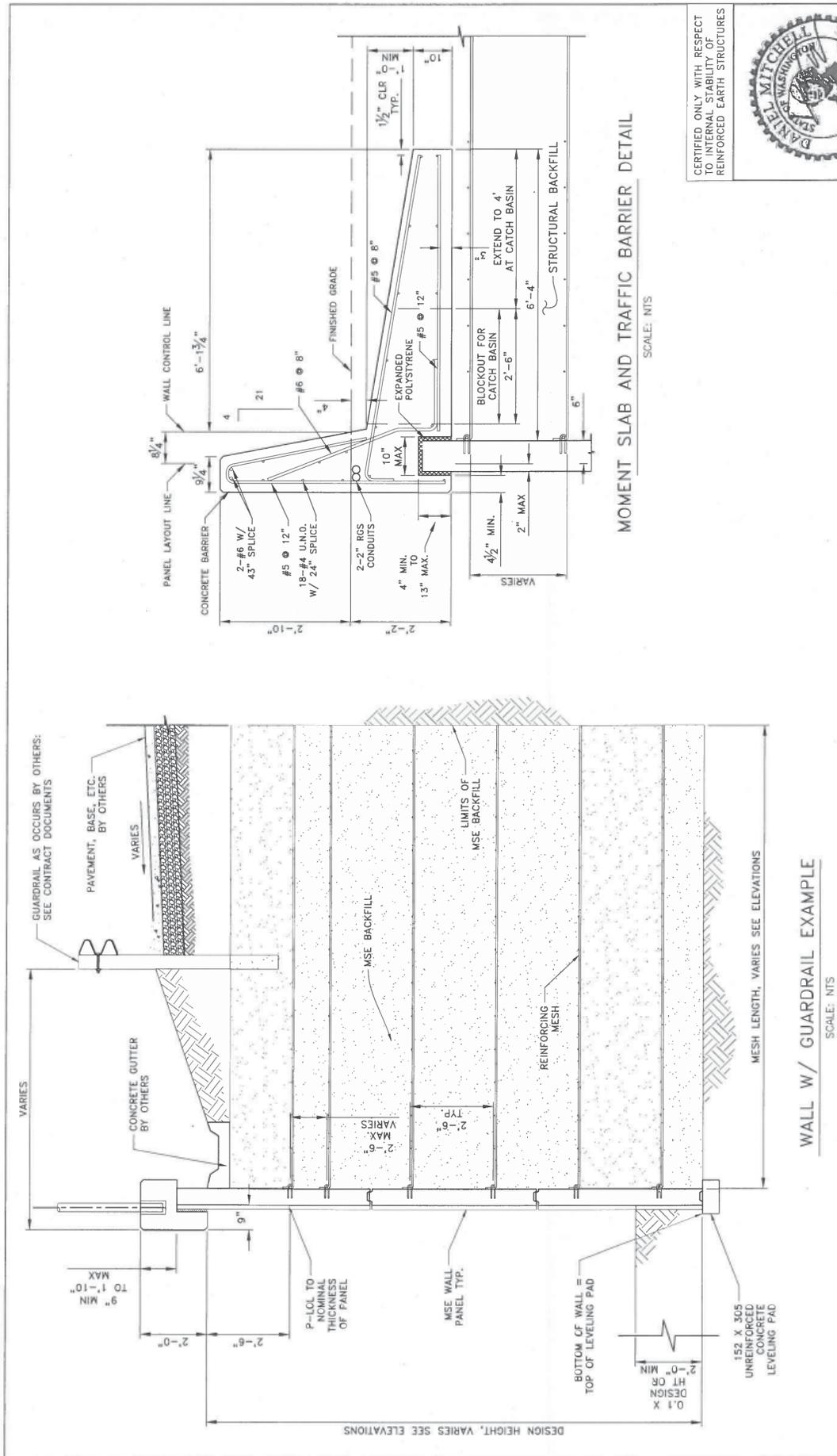
CURVED LAYOUT LINE DETAILS  
 STATE OF WASHINGTON  
 DEPARTMENT OF TRANSPORTATION  
 SSL LLC, MSE PLUS  
 PANEL WALL SYSTEM

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**SSL**  
 MAIN OFFICE: Valley Dr., Suite E, 4740  
 Scotts Valley, California 95066  
 Ph: 831-430-9300  
 Fax: 831-430-9340

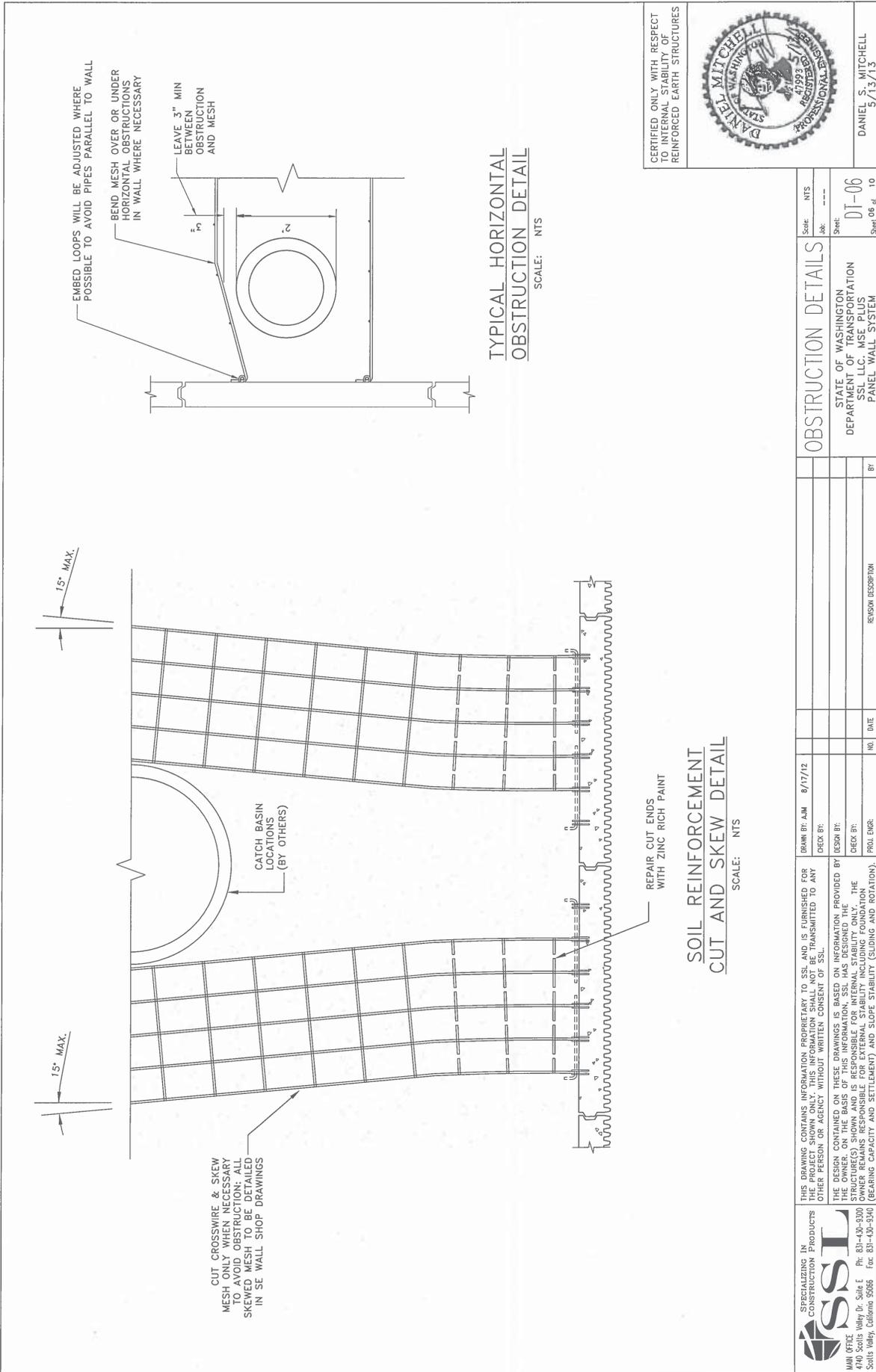


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TRAFFIC BARRIER SYSTEMS		Scale: NTS	Job: ---
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION SSL LLC, MSE PLUS PANEL WALL SYSTEM		Sheet: DT-05	Sheet 05 of 10
DESIGNED BY: AJM	8/17/12	CHECKED BY:	
DESIGNED BY:		CHECKED BY:	
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<p>SPECIALIZING IN CONSTRUCTION PRODUCTS</p> <p>4140 South Valley Dr., Suite E South Valley, California 95566 Ph: 831-430-3300 Fax: 831-430-3140</p>			

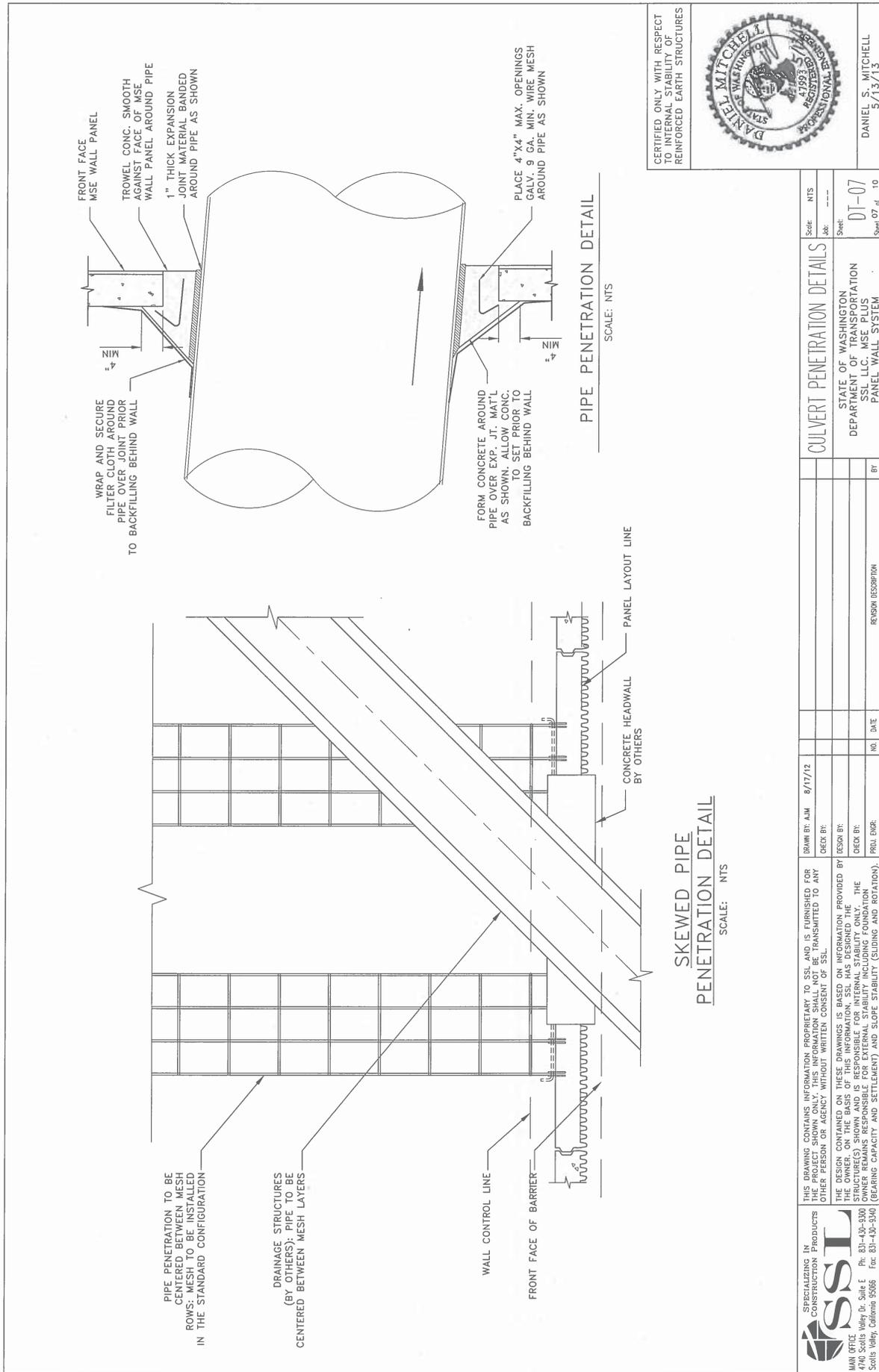


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<p>DESIGNED BY: AM</p> <p>CHECK BY:</p>	<p>DATE: 8/17/12</p> <p>NO.:</p>	<p>NO.:</p> <p>DATE:</p>
<p>DESIGNED BY:</p> <p>CHECK BY:</p> <p>PROJ. ENGR:</p>	<p>NO.:</p> <p>DATE:</p>	<p>NO.:</p> <p>DATE:</p>
<p>REVISION DESCRIPTION</p>		
<p>BY:</p>		
<p>OBSTRUCTION DETAILS</p>		
<p>STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION SSL LLC, MSE PLUS PANEL WALL SYSTEM</p>		
<p>Scale: NTS</p> <p>Job: ---</p>	<p>Sheet: DT-06</p> <p>Sheet 06 of 10</p>	

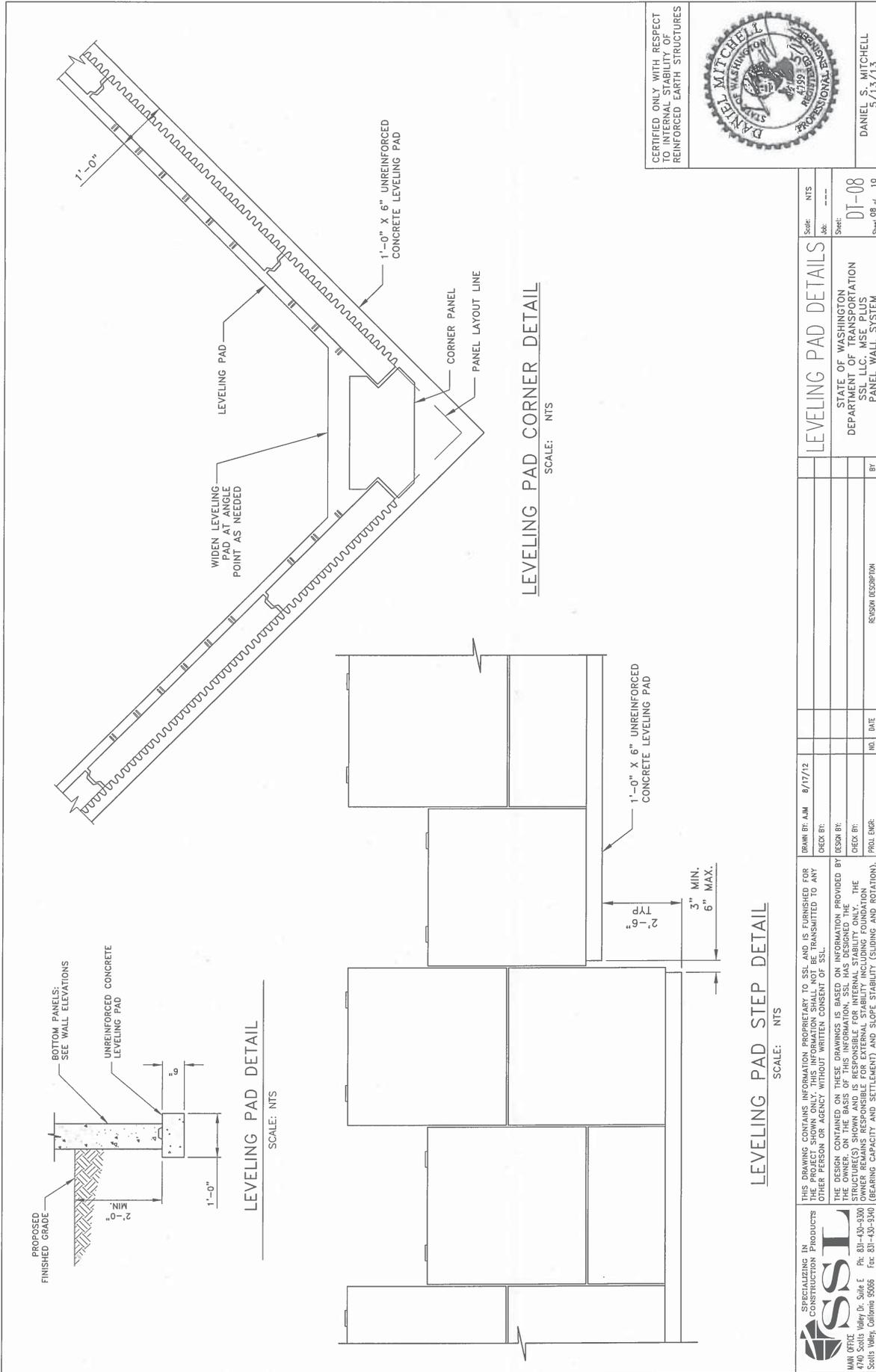


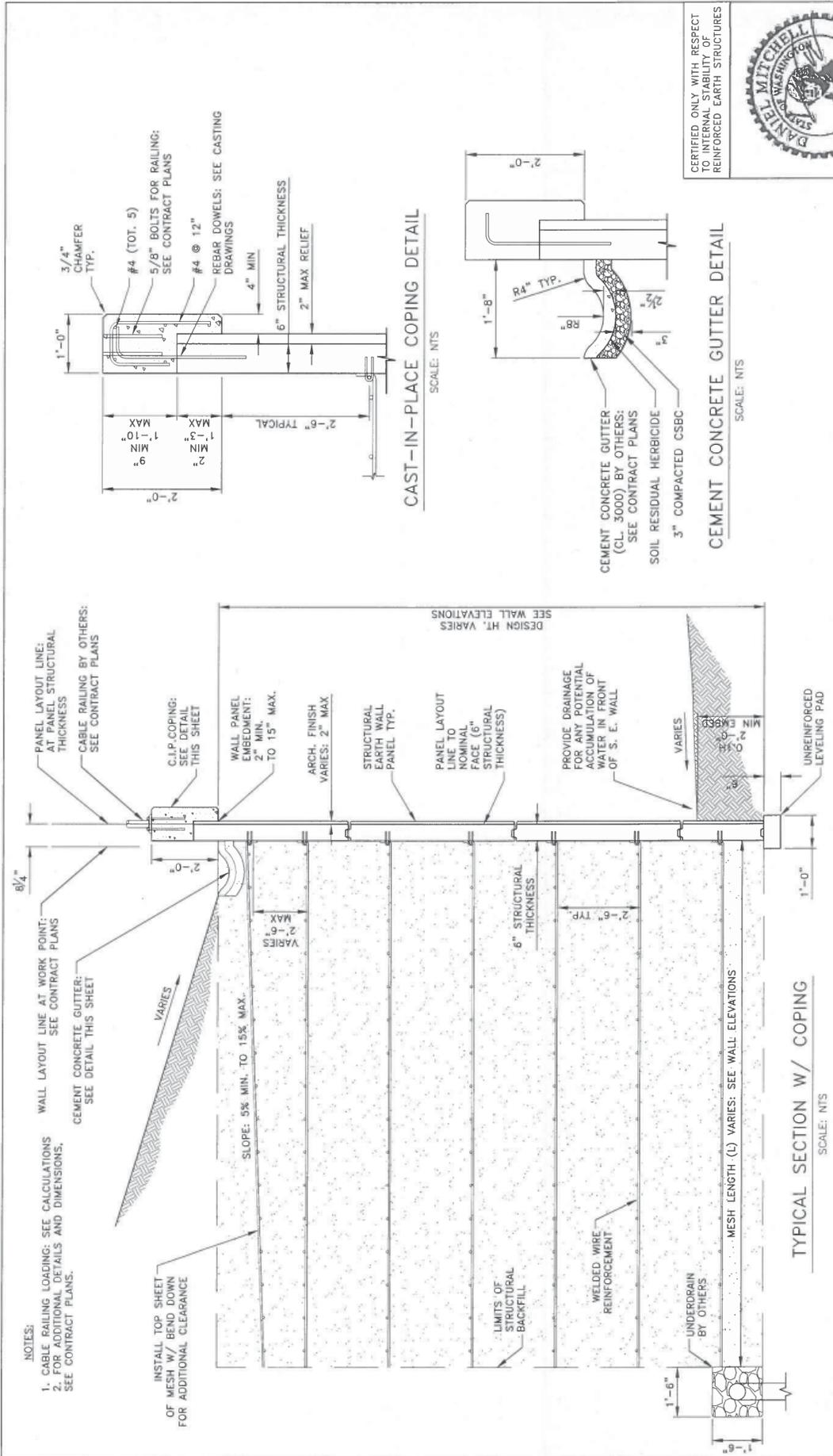
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<p>SPECIALIZING IN CONSTRUCTION PRODUCTS</p> <p><b>SSL</b></p> <p>MAIN OFFICE 4740 Scotts Valley Dr., Suite E Scotts Valley, California 95666 Ph: 831-430-9300 Fax: 831-430-9340</p>		<p>DESIGNED BY: _____</p> <p>CHECK BY: _____</p>	<p>DATE: _____</p>
<p>STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION SSL LLC MSE PLUS PANEL WALL SYSTEM</p>		<p>NO. _____</p> <p>DATE: _____</p>	<p>BY: _____</p>
<p>CULVERT PENETRATION DETAILS</p>		<p>Scale: NTS</p> <p>Job: _____</p> <p>Sheet: DT-07</p> <p>Sheet 07 of 10</p>	

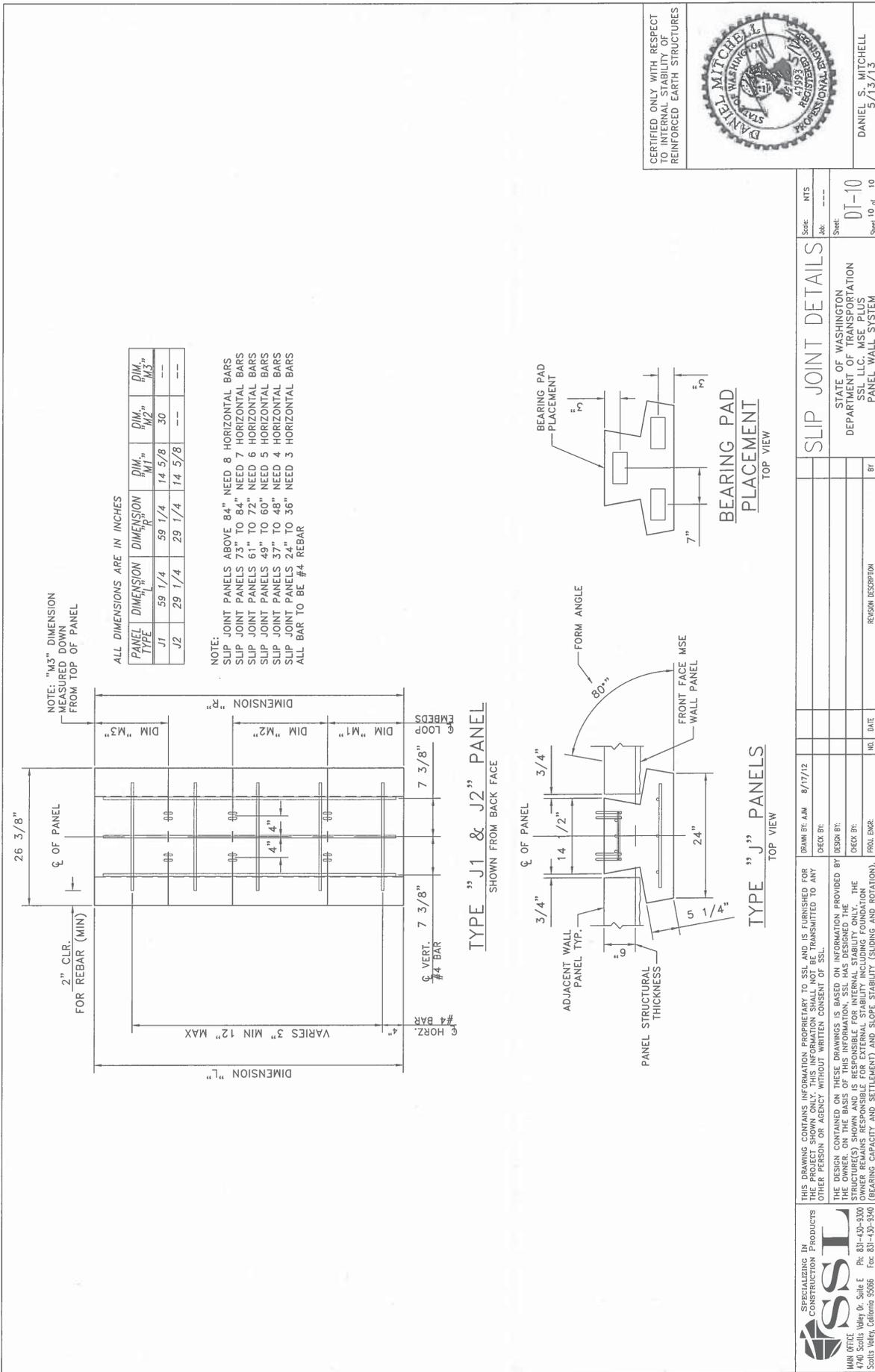




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<p>INSTALL TOP SHEET OF MESH W/ BEND DOWN FOR ADDITIONAL CLEARANCE</p> <p>WALL LAYOUT LINE AT WORK POINT: SEE CONTRACT PLANS</p> <p>CEMENT CONCRETE GUTTER: SEE DETAIL THIS SHEET</p> <p>WALL LAYOUT LINE: AT PANEL STRUCTURAL THICKNESS</p> <p>CABLE RAILING BY OTHERS: SEE CONTRACT PLANS</p> <p>CLIP COPING: SEE DETAIL THIS SHEET</p> <p>WALL PANEL EMBEDMENT: 2" MIN. TO 15" MAX.</p> <p>ARCH. FINISH VARIES: 2" MAX</p> <p>STRUCTURAL EARTH WALL PANEL TYP.</p> <p>PANEL LAYOUT LINE TO NOMINAL FACE (6" STRUCTURAL THICKNESS)</p> <p>PROVIDE DRAINAGE FOR ANY POTENTIAL ACCUMULATION OF WATER IN FRONT OF S. E. WALL</p> <p>VARIES</p> <p>UNREINFORCED LEVELING PAD</p>		<p>DRUM BY: AM</p> <p>DATE: 8/17/12</p> <p>DESIGNED BY: [ ]</p> <p>CHECKED BY: [ ]</p> <p>PROJ. ENGR: [ ]</p>	<p>SCALE: NTS</p> <p>Sheet: DT-09</p> <p>Sheet 09 of 10</p>
<p>NOTES:</p> <ol style="list-style-type: none"> <li>CABLE RAILING LOADING: SEE CALCULATIONS FOR ADDITIONAL DETAILS AND DIMENSIONS. SEE CONTRACT PLANS.</li> </ol>		<p>STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION SSL CONTRACT USE ONLY PANEL WALL SYSTEM</p>	<p>SCALE: NTS</p> <p>Sheet: DT-09</p> <p>Sheet 09 of 10</p>

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DANIEL S. MITCHELL  
5/13/13



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	CHECK BY: DESIGN BY: CHECK BY: PROJ. ENGR:	SHEET: DI-10 SHEET 10 OF 10
REVISION DESCRIPTION NO. DATE BY		



## ***Preapproved Wall Appendix: Specific Requirements and Appendix 15-R Details for Landmark Reinforced Soil Wall***

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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.

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	$3948+N*\tan 16^\circ$	9456	$T_{ultconn}/9456$	1.2
Tapered Block	Miragrid 5XT	$2837 - N*\tan 7^\circ$	3844	$T_{ultconn}/3844$	1.45*
	Miragrid 8XT	$4250 - N*\tan 5^\circ$	6564	$T_{ultconn}/6564$	1.45*
	Miragrid 10XT	$3770+N*\tan 30^\circ$ 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  $RF_{CR}$  reported in the QPL, Appendix D for these geogrid products.

### Approved Connection Strength Design Values for Landmark Walls

Table 15-R-1

$T_{ac}$ , the long-term connection strength, shall be calculated as follows:

$$T_{ac} = \frac{T_{MARV} \times CR_u}{RF_{CR} \times RF_D}$$

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 1/8 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 be 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.





**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**

**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.



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CERTIFICATE OF AUTHORIZATION: 2535	
DESIGN ENGINEER:	WASHINGTON REGISTRATION: 47447
DEAN SANDRI	

Scale: NOT TO SCALE

VERIFY THAT THE BAR ABOVE IS "1" IF NOT, ADJUST THE SCALE ACCORDINGLY.

0 1

Prof. No: AVE  
 Date: 04/20/11  
 Reviewed by: AVE  
 Drawn By: AVE

PROPOSED SEGMENTAL RETAINING WALLS		Sheet or W3	10
0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM			
WSDOT SUBMITTAL DETAILS		GENERAL NOTES	
REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):			

**ISOMETRIC VIEWS - FULL SIZE UNITS**  
(X) (WX) (NOT TO SCALE)

**ISOMETRIC VIEWS - HALF SIZE UNITS**  
(X) (WX) (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.

**STRAIGHT UNIT - LARGE**  
**STRAIGHT UNIT - SMALL**

**TAPERED UNIT - LARGE**  
**TAPERED UNIT - SMALL**

**FOUNDATION UNIT**

**STRAIGHT UNIT - LARGE**  
**STRAIGHT UNIT - SMALL**

**TAPERED UNIT - LARGE**  
**TAPERED UNIT - SMALL**

**3-WAY BLOCK VIEWS - FULL SIZE UNITS**  
(X) (WX) (NOT TO SCALE)

**3-WAY BLOCK VIEWS - HALF SIZE UNITS**  
(X) (WX) (NOT TO SCALE)

**3-WAY BLOCK VIEWS - CORNER UNITS**  
(X) (WX) (NOT TO SCALE)

**CAP UNIT**

**MISCELLANEOUS UNITS**  
(X) (WX) (NOT TO SCALE)

**LEFT CORNER UNIT**

**RIGHT CORNER UNIT**

**CERTIFICATE OF AUTHORIZATION: 2535**

DESIGN ENGINEER: DEAN SANDRI	WASHINGTON REGISTRATION: 47447
---------------------------------	-----------------------------------

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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

PROPOSED SEGMENTAL RETAINING WALLS  
0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM  
WSDOT SUBMITTAL DETAILS

REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):	TYPICAL DETAILS	Sheet of W4 10
---	-----------------	----------------

**ANCHOR WALL SYSTEMS, INC.**

**REINFORCEMENT CONNECTION**  
 (X) WX (NOT TO SCALE)  
 ANCHOR LOCKING BAR (ON REINFORCEMENT LAYERS ONLY)  
 EXTEND GEOSYNTHETIC REINFORCEMENT TO WITHIN 1' OF THE FACE OF THE LOWER BLOCK

**CAP BLOCK DESIGN DETAILS**  
 (X) WX (NOT TO SCALE)  
 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 RADIUS 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.

**POTENTIAL GROUNDWATER BEHIND WALL**  
 (X) WX (NOT TO SCALE)  
 CAP BLOCK  
 LANDMARK BLOCK  
 0"-4" (VARIES)  
 FINISHED GRADE  
 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 4" MINIMUM GROUNDWATER RISE)  
 FILTER FABRIC  
 6" NON-REINFORCED CONCRETE BASE LEVELING PAD

**TYPICAL STEP-UP DETAIL**  
 (X) WX (NOT TO SCALE)  
 FULL-HIGH LANDMARK BLOCK  
 LANDMARK FOUNDATION BLOCK  
 FOUNDATION SOIL  
 6" NON-REINFORCED CONCRETE BASE LEVELING PAD  
 HALF-HIGH LANDMARK BLOCK

**WALL FACE WITH RAILING/FENCE**  
 (X) WX (NOT TO SCALE)  
 SPLIT RAIL, CHAIN LINK, PIPE OR OTHER FENCE/RAILING TYPE  
 SLEEVE FOR POST/PIPE (PVC OR SIMILAR)

**DAYLIGHT DRAIN TILE THROUGH WALL**  
 (X) WX (NOT TO SCALE)  
 REMOVE PORTION OF ADJACENT UNITS TO ALLOW SUBDRAIN THRU FACE  
 4" MIN. DIA. WALL SUBDRAIN (EL. VARIES)  
 4" TEE  
 4" CUT  
 2" CUT  
 DAYLIGHT SUBDRAIN THROUGH WALL FACE  
 FINISHED GRADE  
 1ST COURSE ABOVE GRADE  
 1ST COURSE BELOW GRADE  
 FOUNDATION COURSE  
 SPACING VARIES DEPENDING ON SITE CONDITIONS (90 FT. MAXIMUM)

**WALL ABUTTING EXISTING STRUCTURE**  
 (X) WX (NOT TO SCALE)  
 EXISTING STRUCTURE  
 FILTER FABRIC BEHIND TWO BLOCKS, EXTENDING BACK 3 FT NEXT TO THE EXISTING STRUCTURE  
 FIRST COURSE  
 EXISTING STRUCTURE  
 FILTER FABRIC BEHIND TWO BLOCKS, EXTENDING BACK 3 FT NEXT TO THE EXISTING STRUCTURE  
 SECOND COURSE

**TYPICAL BASE PREPARATION**  
 (X) WX (NOT TO SCALE)  
 APPROXIMATE LIMITS OF EXCAVATION  
 LEVELING TRENCH  
 6" NON-REINFORCED CONCRETE BASE LEVELING PAD  
 NATIVE SOIL  
 2 FT. MIN.

**90° CORNERS - LEVELING PAD**  
 (X) WX (NOT TO SCALE)  
 6" NON-REINFORCED CONCRETE BASE LEVELING PAD  
 90° OUTSIDE CORNER  
 90° INSIDE CORNER

**PROPOSED SEGMENTAL RETAINING WALLS**  
 0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM  
 WSDOT SUBMITTAL DETAILS

**CERTIFICATE OF AUTHORIZATION: 2535**  
 DESIGN ENGINEER: DEAN SANDRI  
 WASHINGTON REGISTRATION: 47447

**SCALE:** 3/16" = 1'-0"  
 VERIFY THAT THE BAR ABOVE IS 1" IF NOT, ADJUST THE SCALE ACCORDINGLY.

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**REVISION DATES (SEE SHEET WX FOR REVISION HISTORY):**

Prof. No:	AWM	Reviewed By:	AWM
Date:	04/20/11	Drawn By:	AWM

Sheet of 10  
 TYPICAL DETAILS  
 W5



**FOR ANGLES LESS THAN 55° THIS UNIT WILL REQUIRE MODIFICATION**

RETURN ANGLE (X)

PIECE A, B, C, D, E, F

COURSE A, COURSE B

ANGLE X

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<sub>1</sub>.
2. CREATE PIECE B BY MEASURING DISTANCE D<sub>1</sub> FROM THE CORNER AND CUTTING 1/2 ANGLE X. THE FLAT SIDE FROM THE CORNER IS DISTANCE D<sub>2</sub>.
3. CREATE PIECE C BY MEASURING DISTANCE D<sub>2</sub> FROM THE CORNER AND CUTTING ANGLE X TOWARD THE FACE OF THE BLOCK. MARK PIECE D AS THE MIRROR OF PIECE B.
4. PIECE D IS THE MIRROR OF PIECE B.
5. PIECE E IS THE MIRROR OF PIECE A.
6. PIECE F IS THE MIRROR OF PIECE C.
7. AFFIX PIECES A, B, C, D, E & F WITH CONSTRUCTION GRADE ADHESIVE.
8. ALTERNATE COURSES A & B UNTIL FULL HEIGHT OF CORNER HAS BEEN REACHED.

**ACUTE OUTSIDE CORNER (NOT TO SCALE)**

COURSE A, COURSE B

PIECE A, PIECE B

CUT CORNER TO FIT

NOTE:

1. ALTERNATE COURSES A AND B UNTIL FULL HEIGHT OF CORNER HAS BEEN REACHED.

**90° OUTSIDE CORNER W/OUT CORNER UNIT (NOT TO SCALE)**

COURSE A, COURSE B

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)**

COURSE A, COURSE B

PIECE A, PIECE B, PIECE C

CUT CORNER TO FIT

NOTE:

1. ALTERNATE COURSES A AND B UNTIL FULL HEIGHT OF CORNER HAS BEEN REACHED.

**ACUTE INSIDE CORNER (NOT TO SCALE)**

COURSE A, COURSE B

PIECE A, PIECE B

CUT CORNER TO FIT

NOTE:

1. ALTERNATE COURSES A AND B UNTIL FULL HEIGHT OF CORNER HAS BEEN REACHED.

**OBTUSE INSIDE CORNER (NOT TO SCALE)**

COURSE A, COURSE B

PIECE A, PIECE B, PIECE C

CUT CORNER TO FIT

NOTE:

1. ALTERNATE COURSES A AND B UNTIL FULL HEIGHT OF CORNER HAS BEEN REACHED.

**0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM WSDOT SUBMITTAL DETAILS**

PIECE A, PIECE B, PIECE C

FABRICATED CORNER UNIT

COURSE A, COURSE B

CUT CORNER TO FIT

NOTE:

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.

**PROPOSED SEGMENTAL RETAINING WALLS**

PIECE A, PIECE B, PIECE C

FABRICATED CORNER UNIT

COURSE A, COURSE B

CUT CORNER TO FIT

NOTE:

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.

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CERTIFICATE OF AUTHORIZATION: 2535  
DESIGN ENGINEER: DEAN SANDRI  
WASHINGTON REGISTRATION: 47447

Scale: 3/16" = 1'-0"  
VERIFY THAT THE BAR ABOVE IS 1". IF NOT, ADJUST THE SCALE ACCORDINGLY.

REVISION DATES (SEE SHEET W7 FOR REVISION HISTORY):  
Reviewed By: AJWE  
Drawn By: AJWE  
Date: 04/20/11



**SLOPE ABOVE WALL (NOT TO SCALE)**

**SLOPE BELOW WALL (NOT TO SCALE)**

**FENCE BEHIND WALL (NOT TO SCALE)**

**TYPICAL CONCRETE JERSEY BARRIER (NOT TO SCALE)**

**CABLE RAILING POST FOUNDATION DETAILS (NOT TO SCALE)**

**SHORELINE/CHANNEL WALL (NOT TO SCALE)**

**DRAINAGE SWALE (NOT TO SCALE)**

**NOTES:**  
 REINFORCEMENT NOT SHOWN FOR CLARITY. REFERENCE THE ANCA DESIGN MANUAL FOR ADDITIONAL DETAILS.

**NOTE:**  
 BARRIER SHOWN IS CONCEPTUAL ONLY. DESIGN OF THE BARRIER AND REINFORCING STEEL IS THE RESPONSIBILITY OF THE PROJECT STRUCTURAL ENGINEER.

**NOTE:**  
 CONCRETE FILLED TUBE OR FORM TO BE SET DURING THE WALL CONSTRUCTION, NOT DRILLED THROUGH GEOGRID AFTERWARDS WHEN DIRECTLY BEHIND UNITS.

**CERTIFICATE OF AUTHORIZATION: 2535**  
 DESIGN ENGINEER: DEAN SANDRI  
 WASHINGTON REGISTRATION: 47447

**PROPOSED SEGMENTAL RETAINING WALLS**  
**0 & 4 DEGREE BATTERED ANCHOR LANDMARK WALL SYSTEM**  
**WSDOT SUBMITTAL DETAILS**

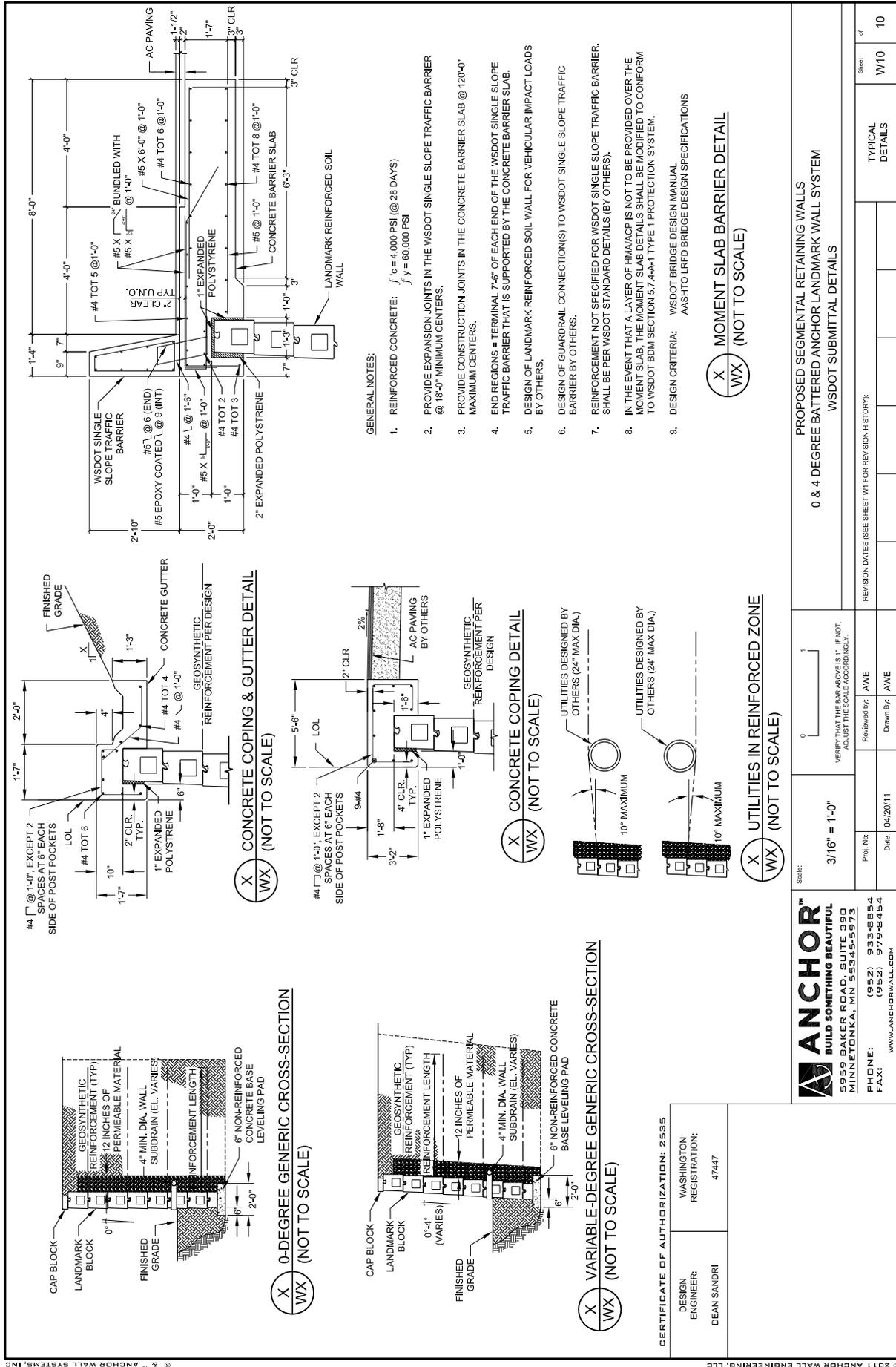
Scale: 3/16" = 1'-0"  
 VERIFY THAT THE BAR ABOVE IS 1" IF NOT, ADJUST THE SCALE ACCORDINGLY.

Revised By: AWE  
 Drawn By: AWE  
 Date: 04/20/11

REVISION DATES (SEE SHEET W1 FOR REVISION HISTORY):

Sheet	W9	of	10
TYPICAL DETAILS			

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# **Preapproved Wall Appendix: Specific Requirements and Details for Allan Block Walls With Face Batter of 3 Degrees or More**

## **Appendix 15-S**

---

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 Stones, AB Classic, AB Three, and AB Rocks. These blocks are for a facing batter of 3 degrees or more. Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2 feet.

**Soil Reinforcement** – Only geosynthetic reinforcement listed in the 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 3 degrees or more (i.e., facing blocks AB Stones, AB Classic, AB Three, and AB Rocks), this includes the following specific products that are approved for use with this wall system:

Miragrid 2XT	Miragrid 7XT	Stratagrid SG150
Miragrid 3XT	Miragrid 8XT	Stratagrid SG200
Miragrid 5XT	Miragrid 10XT	Stratagrid SG350
		Stratagrid SG500

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 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. Connection testing was done for the range of blocks and geosynthetic reinforcements preapproved for this wall system. The design facing/reinforcement connection strength shall be as specified in the following table:

Applicable Facing Blocks	Geogrid Product	T <sub>u</sub> (lbs/ft)
AB Stones, AB Classic, AB Three, and AB Rocks	Miragrid 2XT	125.6 + N*Tan 58.5° below N = 915 lbs/ft, 1624 lbs/ft above N = 915 lbs/ft
	Miragrid 3XT	1420 + N*Tan 11°
	Miragrid 5XT	1191 + N*Tan 18°
	Miragrid 7XT	1065 + N*Tan 25.6°
	Miragrid 8XT	1063 + N*Tan 40° below N = 2155 lbs/ft, 2872 lbs/ft above N = 2155 lbs/ft
	Miragrid 10XT	513 + N*Tan 52° below N = 1000 lbs/ft, 1426 + N*Tan 23° above N = 1000 lbs/ft
	Stratagrid SG150	930 + N*Tan 24°
	Stratagrid SG200	951 + N*Tan 24°
	Stratagrid SG350	929 + N*Tan 25°
	Stratagrid SG500	848 + N*Tan 30°

N = normal load at reinforcement layer at facing, in lbs/ft of width parallel to face.

### Approved Connection Strength Design Values for Allan Block Walls

Table 15-S-1

T<sub>ac</sub>, the long-term connection strength, shall be calculated as follows:

$$T_{ac} = \frac{T_u}{RF_{CR} \times RF_D}$$

Where:

T<sub>u</sub> = the ultimate connection strength from the product specific connection strength tests, the results of which are provided in the previous table,

RF<sub>CR</sub> = creep reduction factor for the geosynthetic, and

RF<sub>D</sub> = the durability reduction factor for the geosynthetic.

RF<sub>CR</sub> and RF<sub>D</sub> shall be as provided in the QPL Appendix D.

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.

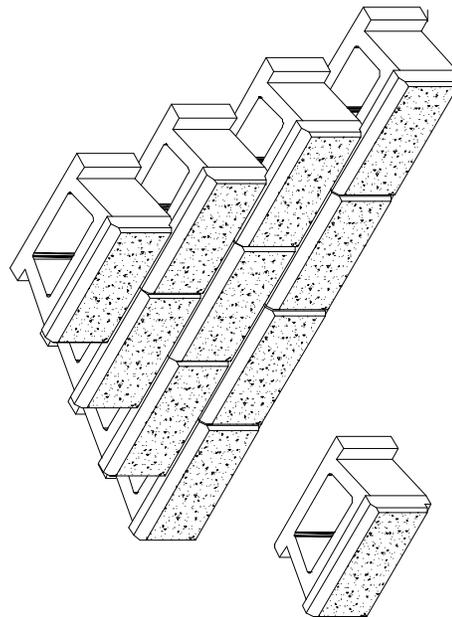
It is noted in ASTM C1372 that a dimensional tolerance for the height of the block of 1/8 inch is allowed, but that Section 15.5.3.8 recommends a tighter dimensional tolerance of 1/16 inch. Based on WSDOT experience, for walls greater than 25 feet in height, some cracking of facing blocks due to differential 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 Allan Block 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 be 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.

CONSTRUCTION DRAWINGS  
Prepared for  
STATE OF WASHINGTON  
DEPARTMENT OF TRANSPORTATION

ALLAN BLOCK DETAIL AND CONSTRUCTION NOTES



SHEET INDEX

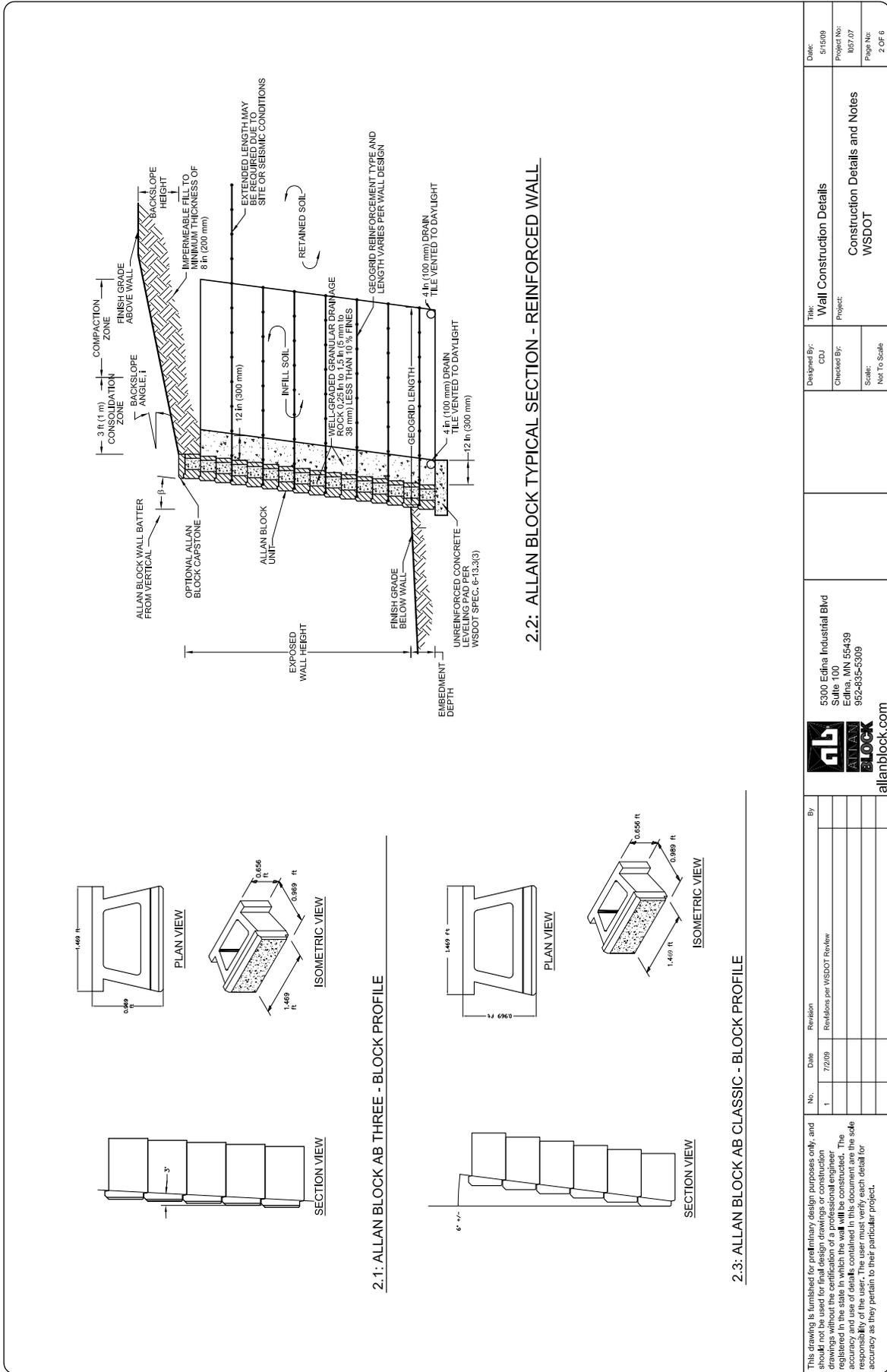
SHEET NO.	SHEET TITLE
1	TITLE SHEET
2	WALL CONSTRUCTION DETAILS
3	GEOGRID CONSTRUCTION DETAILS
4	WALL PENETRATIONS DETAILS
5	RAILING AND BARRIER DETAILS
6	LARGE GRID OBSTRUCTION DETAILS

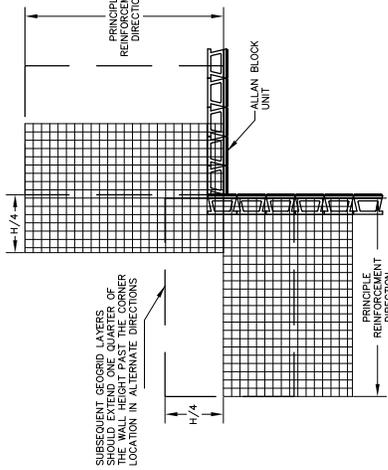
This drawing is furnished for preliminary design purposes only, and drawings without the certification of a professional engineer registered in the state in which the wall will be constructed. The accuracy and use of details contained in this document are the sole responsibility of the user. The user must verify each detail for accuracy as they pertain to their particular project.

No.	Date	Revision	By
1	7/2/09	Revisions per WSDOT Review	

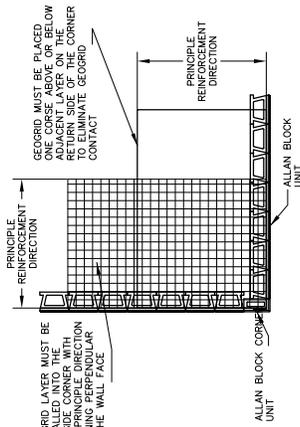
  
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 Edina, MN 55439  
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[allanblock.com](http://allanblock.com)

Designer By: GCI	Title: Title Sheet	Date: 9/15/09
Checked By:	Project: Construction Details and Notes WSDOT	Project No: 1057.07
Scale: Not To Scale		Page No: 1 OF 6

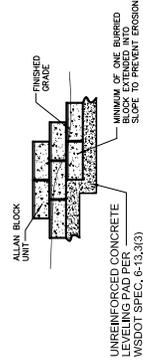




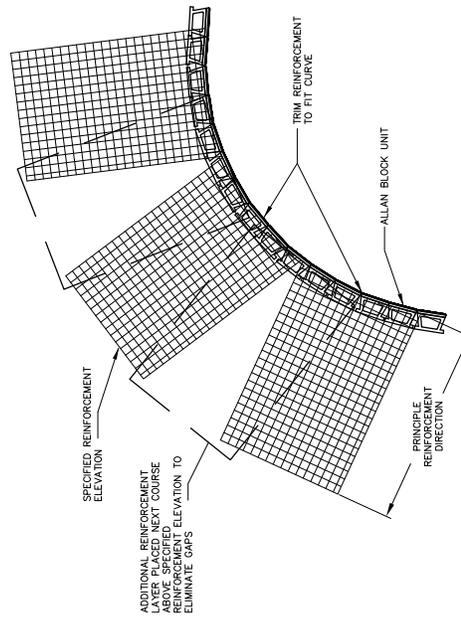
3.1: INSIDE CORNER GEOGRID OVERLAP



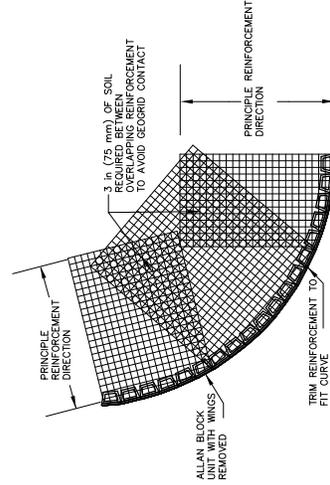
3.2: OUTSIDE CORNER GEOGRID OVERLAP



3.3: STEP UP AT BASE COURSE



3.4: INSIDE CURVE GEOGRID OVERLAP



3.5: OUTSIDE CURVE GEOGRID OVERLAP

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No.	Date	Revision
1	7/2/09	Revisions per WSDOT Review

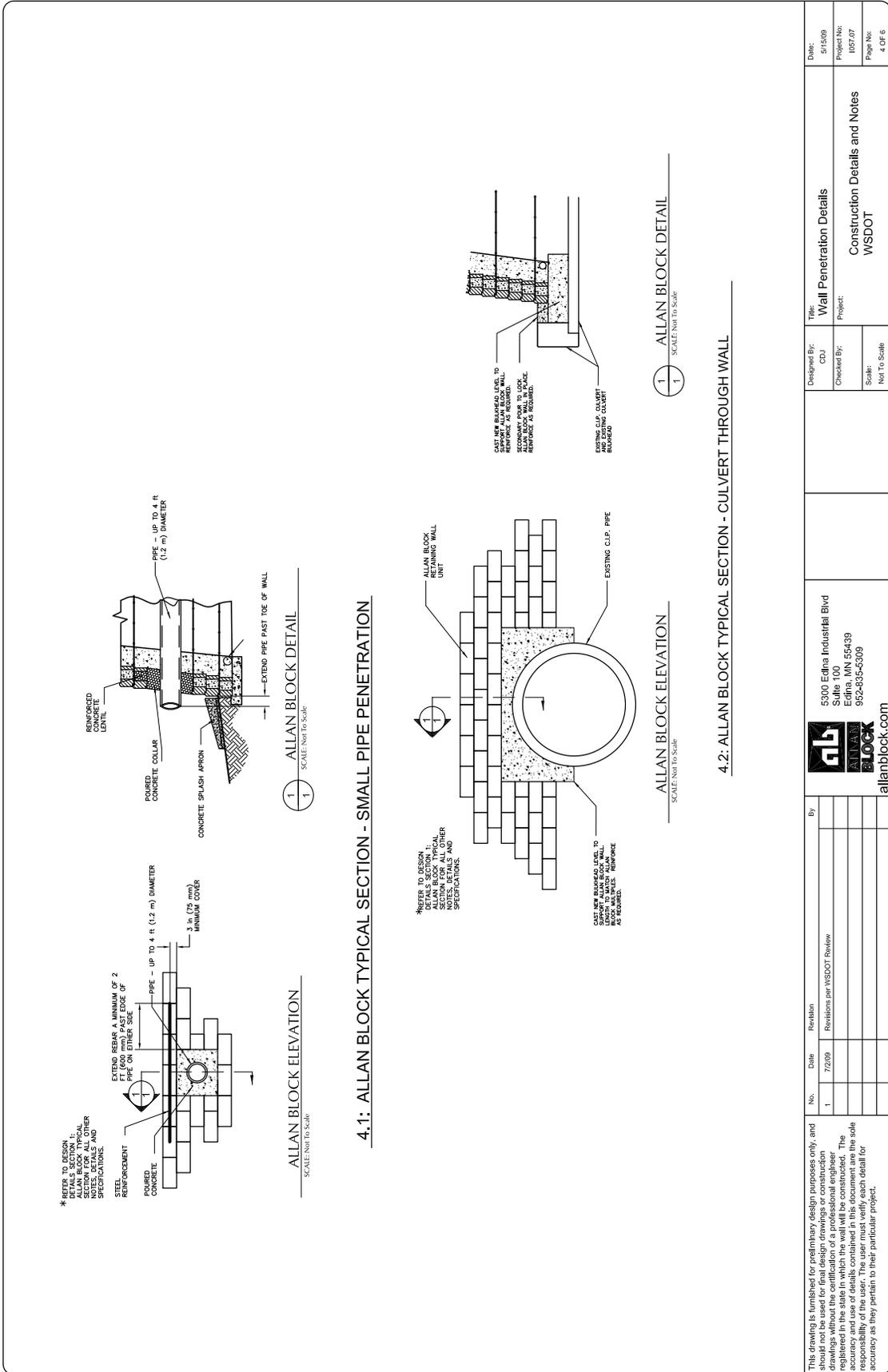
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Designed By: CDU  
Checked By:  
Scale: Not To Scale

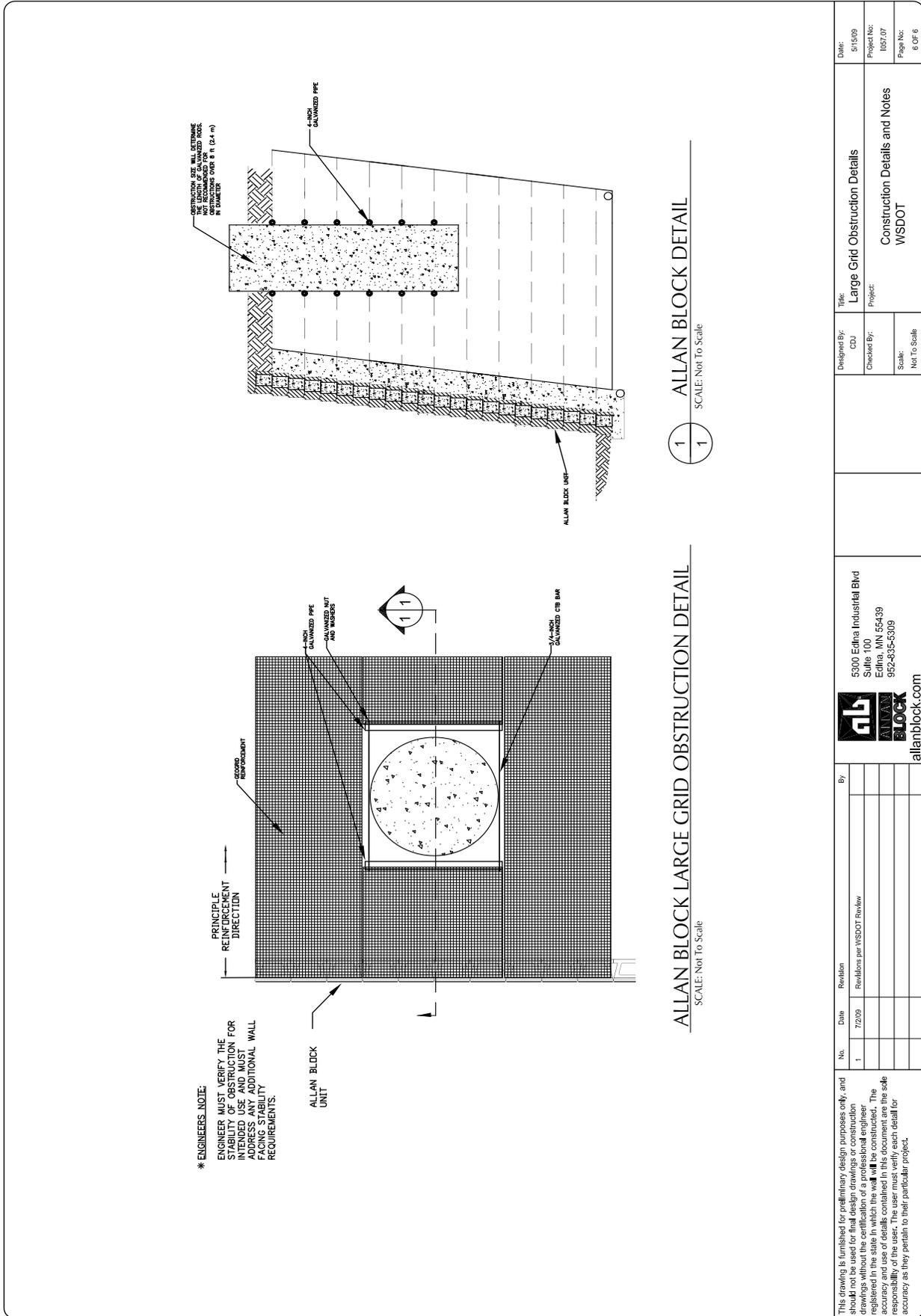
Title: Geogrid Construction Details  
Project: Construction Details and Notes  
WSDOT

Date: 6/16/09  
Project No: 1057.07  
Page No: 3 OF 6



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No. 1 Date: 7/2009 Revision: Revisions per WSDOT Review	By:	Designed By: CDU Checked By:	Scale: Not To Scale	
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	1	7/2/09	Revisions per WSDOT Review	
Title: <b>Large Grid Obstruction Details</b> Project: <b>Construction Details and Notes WSDOT</b>				
Designed By: <b>CDJ</b> Checked By: <b>CDJ</b> Scale: <b>Not To Scale</b>				
Date: <b>5/11/09</b> Project No: <b>1057.07</b> Page No: <b>6 OF 6</b>				
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