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For updating printed manuals, page numbers indicating portions of the manual that are to be removed and replaced are shown below.

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Chapter 15  Abutments, Retaining Walls, and Reinforced Slopes

15-1  Introduction and Design Standards

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

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

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

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

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

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

Overview of Wall Classifications and Design Process for Walls

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

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

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

A number of proprietary wall systems have been extensively reviewed by the Bridge and Structures Office and the HQ Geotechnical Office. This review has resulted in WSDOT preapproving some proprietary wall systems. The design procedures and wall details for these preapproved wall systems shall be in accordance with this manual and other manuals specifically referenced herein as applicable to the type of wall being designed, unless alternate design procedures have been agreed upon between WSDOT and the proprietary wall manufacturer. These preapproved design procedures and details allow the manufacturers to competitively bid a particular project without having a detailed wall design provided in the contract plans. Note that proprietary wall manufacturers may produce several retaining wall options, and not all options from a given manufacturer have been preapproved. The Bridge and Structures Office shall be contacted to obtain the current listing of preapproved proprietary systems prior to including such systems in WSDOT projects. A listing of the preapproved wall systems, as of the current publication date for this manual, is provided in Appendix 15-D. Specific preapproved details and system specific design requirements for each wall system are also included as appendices to Chapter 15. Incorporation of non-preapproved systems requires the wall supplier to completely design the wall prior to advertisement for construction.
All of the manufacturer’s plans and details would need to be incorporated into the contract documents. Several manufacturers may need to be contacted to maintain competitive bidding. More information is available in chapters 610 and 730 of the Design Manual M 22-01.

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

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

15-3  Required Information

15-3.1  Site Data and Permits

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

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

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

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

• Identify engineering properties and parameters required for these analyses.
• Identify the number of tests/samples needed to estimate engineering properties.

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

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

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<td>• subsurface profile (soil, ground water, rock) • horizontal earth pressure coefficients • interface shear strengths • foundation soil/wall fill shear strengths? • compressibility parameters? (including consolidation, shrink/swell potential, and elastic modulus) • chemical composition of fill/foundation soils? • hydraulic conductivity of soils directly behind wall? • time-rate consolidation parameters? • geologic mapping including orientation and characteristics of rock discontinuities? • design flood elevations • seismicity</td>
<td>• SPT • CPT • dilatometer • vane shear • piezometers • test fill? • nuclear density? • pullout test (MSEW/RSS) • rock coring (RQD) • geophysical testing</td>
<td>• 1-D Oedometer • triaxial tests • unconfined compression • direct shear tests • grain size distribution • Atterberg limits • specific gravity • pH, resistivity, chloride, and sulfate tests? • moisture content? • organic content • moisture-density relationships • hydraulic conductivity</td>
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### Table 15-1  Summary of Information Needs and Testing Considerations

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<td>• dewatering</td>
<td>• geologic mapping including orientation and characteristics of rock discontinuities</td>
<td>• vane shear</td>
<td>• specific gravity</td>
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<td></td>
<td>• chemical compatibility</td>
<td>• seismicity</td>
<td>• dilatometer</td>
<td>• organic content</td>
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<td></td>
<td>of wall/soil</td>
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<td>• pullout tests (anchors, nails)</td>
<td>• hydraulic conductivity</td>
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<td></td>
<td>• lateral earth pressure</td>
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<td>• geophysical testing</td>
<td>• moisture content</td>
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<td>• down-drag on wall</td>
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<td>• unit weight</td>
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<td>• pore pressures behind</td>
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<td>• obstructions in</td>
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<td>retained soil</td>
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<td>• liquefaction</td>
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<td>• potential for</td>
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<td>subsidence (karst,</td>
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<td>mining, etc.)</td>
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<td>• constructability</td>
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#### 15-3.3 Site Reconnaissance

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

15-3.4 Field Exploration Requirements

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

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

![Figure 15-1](image-url) Exposed Height (H) for a Retaining Wall or Slope

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

- Geological formation(s).
- Location and thickness of soil and rock units.
- Engineering properties of soil and rock units, such as unit weight, shear strength and compressibility.
• Ground water conditions.
• Ground surface topography.
• Local considerations (e.g., liquefiable, expansive or dispersive soil deposits, underground voids from solution weathering or mining activity, or slope instability potential).

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

15-3.4.1 Exploration Type, Depth, and Spacing

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

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

For retaining walls and reinforced slopes less than 100 feet in length, the exploration should occur approximately midpoint along the alignment or where the maximum height occurs. Explorations should be completed on the alignment of the wall face or approximately midpoint along the reinforced slope, i.e., where the height, as defined in Figure 15-1, is 0.5H. Additional borings to investigate the toe slope for walls or the toe catch for reinforced slopes may be required to assess overall stability issues.
For retaining walls and slopes more than 100 feet in length, exploration points should in general be spaced at 100 to 200 feet, but may be spaced at up to 500 feet in uniform, dense soil conditions. Even closer spacing than 100 to 200 feet should be used in highly variable and potentially unstable soil conditions. Where possible, locate at least one boring where the maximum height occurs. Explorations should be completed on the alignment of the wall face or approximately midpoint along the reinforced slope, i.e., where the height is 0.5H. Additional borings to investigate the toe slope for walls or the toe catch for reinforced slopes may be required to assess overall stability issues. Exploration locations may be adjusted if geophysical testing conducted in accordance with Chapter 5 is done, provided enough borings are available to properly interpret the geophysical test results.

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

15-3.4.2 Walls and Slopes Requiring Additional Exploration

15-3.4.2.1 Soil Nail Walls

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

In addition, each soil nail wall should have at least one test pit excavated to evaluate stand-up time of the excavation face. The test pit shall be completed outside the nail pattern, but as close as practical to the wall face to investigate the stand-up time of the soils that will be exposed at the wall face during construction. The test pit shall remain open at least 24 hours and shall be monitored for sloughing, caving, and groundwater seepage. A test pit log shall be prepared and photographs should be taken immediately after excavation and at 24 hours. If variable soil conditions are present along the wall face, a test pit in each soil type should be completed. The depth of the test pits should be at least twice the vertical nail spacing and the length along the trench bottom should be at least one and a half times the excavation depth to minimize soil-arching effects. For example, a wall with a vertical nail spacing of 4 feet would have a test pit 8 feet deep and at least 12 feet in length at the bottom of the pit.
**15-3.4.2.2 Walls With Ground Anchors or Deadman Anchors**

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

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

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

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

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

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

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

In geotechnical borings, soil samples shall be taken during the Standard Penetration Test (SPT). Fine grained soils or peat shall be sampled with 3-in Shelby tubes or WSDOT Undisturbed Samplers if the soils are too stiff to push 3-in Shelby tubes. All samples in geotechnical borings shall be in accordance with chapters 2 and 3.
In hand holes, sack soil samples shall be taken of each soil type encountered, and WSDOT Portable Penetrometer tests shall be taken in lieu of SPT tests. The maximum vertical spacing between portable penetrometer tests should be 5 feet.

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

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

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

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

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

15-3.6 Groundwater

One of the principal goals of a good field reconnaissance and field exploration is to accurately characterize the groundwater in the project area. Groundwater affects the design, performance, and constructability of project elements. Installation of piezometer(s) and monitoring is usually necessary to define groundwater elevations. Groundwater measurements shall be conducted in accordance with Chapter 2, and shall be assessed for each wall. In general, this will require at least one groundwater measurement point for each wall. If groundwater has the potential to affect wall performance or to require special measures to address drainage to be implemented, more than one measurement point per wall will be required.
15-3.7  Wall Backfill Testing and Design Properties

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

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

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

15-4  General Design Requirements  

15-4.1  Design Methods

The AASHTO LRFD Bridge Design Specifications shall be used for all abutments and retaining walls addressed therein. The walls shall be designed to address all applicable limit states (strength, service, and extreme event). Rock walls, reinforced slopes, and soil nail walls are not specifically addressed in the AASHTO specifications, and shall be designed in accordance with this manual. Many of the FHWA manuals used as WSDOT design references were not developed for LRFD design. For those wall types (and including reinforced slopes) for which LRFD procedures are not available, allowable stress design procedures included in this manual, either in full or by reference, shall be used, again addressing all applicable limit states.
Figure 15-2  Permeability and Capillarity of Drainage Materials (Department of Defense 2005)
The load and resistance factors provided in the AASHTO LRFD Specifications have been developed in consideration of the inherent uncertainty and bias of the specified design methods and material properties, and the level of safety used to successfully construct thousands of walls over many years. These load and resistance factors shall only be applied to the design methods and material resistance estimation methods for which they are intended, if an option is provided in this manual or the AASHTO LRFD specifications to use methods other than those specified herein or in the AASHTO LRFD specifications. For estimation of soil reinforcement pullout in reinforced soil (MSE) walls, the resistance factors provided are to be used only for the default pullout methods provided in the AASHTO LRFD specifications. If wall system specific pullout resistance estimation methods are used, resistance factors shall be developed statistically using reliability theory to produce a probability of failure Pf of approximately 1 in 100 or smaller. Note that in some cases, Section 11 of the AASHTO LRFD Bridge Design Specifications refers to AASHTO LRFD Section 10 for wall foundation design and the resistance factors for foundation design. In such cases, the design methodology and resistance factors provided in the Chapter 8 shall be used instead of the resistance factors in AASHTO LRFD Section 10, where the GDM and the AASHTO Specifications differ.

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

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

15-4.2 Tiered Walls

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

For tiered walls, the FHWA manual entitled “Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes” by Berg, et al. (2009), shall be used as the basis for design for those aspects of the design not covered in the AASHTO LRFD Bridge Design Specifications and the GDM.
15-4.3 Back-to-Back Walls

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

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

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

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

15-4.4 Walls on Slopes

Standard Plan walls founded on slopes shall meet the requirements in the Standard Plans. Additionally, all walls shall have a near horizontal bench at the wall face at least 4 feet wide to provide access for maintenance. Bearing resistance for footings in slopes and overall stability requirements in the AASHTO LRFD Bridge Design Specifications shall be met. Table C11.10.2.2-1 in the AASHTO LRFD Bridge Design Specifications should be used as a starting point for determining the minimum wall face embedment when the wall is located on a slope. Use of a smaller embedment must be justified based on slope geometry, potential for removal of soil in front of the wall due to erosion, future construction activity, etc., and external and global wall stability considerations.
15-4.5 **Minimum Embedment**

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

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

15-4.6 **Wall Height Limitations**

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

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

15-4.7 **Serviceability Requirements**

Walls shall be designed to structurally withstand the effects of total and differential settlement estimated for the project site, both longitudinally and in cross-section, as prescribed in the AASHTO LRFD Specifications. In addition to the requirements for serviceability provided above, the following criteria (tables 15-2, 15-3, and 15-4) shall be used to establish acceptable settlement criteria (includes settlement that occurs during and after wall construction):
Table 15-2  Settlement Criteria for Reinforced Concrete Walls, Nongravity Cantilever Walls, Anchored/Braced Walls, and MSE Walls With Full Height Precast Concrete Panels (Soil is Place Directly Against Panel)

<table>
<thead>
<tr>
<th>Total Settlement</th>
<th>Differential Settlement Over 100 Feet</th>
<th>Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>ΔH ≤ 1 in</td>
<td>ΔH&lt;sub&gt;100&lt;/sub&gt; ≤ 0.75 in</td>
<td>Design and Construct</td>
</tr>
<tr>
<td>1 in &lt; ΔH ≤ 2.5 in</td>
<td>0.75 in &lt; ΔH&lt;sub&gt;100&lt;/sub&gt; ≤ 2 in</td>
<td>Ensure structure can tolerate settlement</td>
</tr>
<tr>
<td>ΔH &gt; 2.5 in</td>
<td>ΔH&lt;sub&gt;100&lt;/sub&gt; &gt; 2 in</td>
<td>Obtain Approval&lt;sup&gt;1&lt;/sup&gt; prior to proceeding with design and Construction</td>
</tr>
</tbody>
</table>

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

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

<table>
<thead>
<tr>
<th>Total Settlement</th>
<th>Differential Settlement Over 100 Feet</th>
<th>Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>ΔH ≤ 2 in</td>
<td>ΔH&lt;sub&gt;100&lt;/sub&gt; ≤ 1.5 in</td>
<td>Design and Construct</td>
</tr>
<tr>
<td>2 in &lt; ΔH ≤ 4 in</td>
<td>1.5 in &lt; ΔH&lt;sub&gt;100&lt;/sub&gt; ≤ 3 in</td>
<td>Ensure structure can tolerate settlement</td>
</tr>
<tr>
<td>ΔH &gt; 4 in</td>
<td>ΔH&lt;sub&gt;100&lt;/sub&gt; &gt; 3 in</td>
<td>Obtain Approval&lt;sup&gt;1&lt;/sup&gt; prior to proceeding with design and Construction</td>
</tr>
</tbody>
</table>

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

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

<table>
<thead>
<tr>
<th>Total Settlement</th>
<th>Differential Settlement Over 50 Feet</th>
<th>Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>ΔH ≤ 4 in</td>
<td>ΔH&lt;sub&gt;50&lt;/sub&gt; ≤ 3 in</td>
<td>Design and Construct</td>
</tr>
<tr>
<td>4 in &lt; ΔH ≤ 12 in</td>
<td>3 in &lt; ΔH&lt;sub&gt;50&lt;/sub&gt; ≤ 9 in</td>
<td>Ensure structure can tolerate settlement</td>
</tr>
<tr>
<td>ΔH &gt; 12 in</td>
<td>ΔH&lt;sub&gt;50&lt;/sub&gt; &gt; 9 in</td>
<td>Obtain Approval&lt;sup&gt;1&lt;/sup&gt; prior to proceeding with design and Construction</td>
</tr>
</tbody>
</table>

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

For two-stage walls, Table 15-4 settlement limits apply to the first stage. In that case, the effect of that settlement on installation of the second stage facing shall be addressed.

For the second stage facing, long-term settlement shall be limited to the values shown in tables 15-2 and 15-3.

For MSE walls with precast panel facings up to 75 feet<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 than indicated in tables 15-2 to 15-4 may be necessary to meet aesthetic requirements for the walls.
15-4.8  **Active, Passive, At-Rest Earth Pressures**

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

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

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

15-4.9  **Surcharge Loads**

Article 3.11.6 in the AASHTO LRFD Bridge Design Specifications shall be used for surcharge loads acting on all retaining walls and abutments for walls in which the ground surface behind the wall is 4H:IV or flatter, the wall shall be designed for the possible presence of construction equipment loads immediately behind the wall. These construction loads shall be taken into account by applying a 250 psf live load surcharge to the ground surface immediately behind the wall. Since this is a temporary construction load, seismic loads should not be considered for this load case.
15-4.10 Seismic Earth Pressures

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

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

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

These maximum allowed displacements do not apply to walls that support other structures, unless it is determined that the supported structures have the ability to tolerate the design displacement without compromising the required performance of the supported structure. These maximum allowed displacements also do not apply to walls that support utilities that cannot tolerate such movements and must function after the design seismic event or that support utilities that could pose a significant danger to the public 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.

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

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

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

Numerical, two dimensional effective stress methods may also be used to assess the earth pressure on retaining walls due to retained soil that contains liquefiable layers. The geotechnical designer shall provide documentation that their numerical model has been validated and calibrated with field data, centrifuge data, and/or extensive sensitivity analyses. Due to the highly specialized nature of these more sophisticated liquefaction assessment approaches, approval by the State Geotechnical Engineer is required to use nonlinear effective stress methods for liquefaction evaluation, and independent peer review as described in Section 6-3.3 shall be conducted.
15-4.12 Overall Stability

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

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

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

15-4.13 Wall Drainage

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

- Gabion walls and rock walls are generally considered permeable and do not typically require wall drains, provided construction geotextile is placed against the native soil or fill.
- Soil nail walls shall use composite drainage material centered between each column of nails. The drainage material shall be connected to weep holes using a drain gate or shall be wrapped around an underdrain.
- Cantilever and Anchored wall systems using lagging shall have composite drainage material attached to the lagging face prior to casting the permanent facing. Walls without facing or walls using precast panels are not required to use composite drainage material provided the water can pass through the lagging unhindered.
- For walls subject to periodic inundation due to tides or frequent flooding, additional drainage features shall be included with the wall to prevent or at least minimize the potential for rapid draw-down conditions, such as additional weep holes, chimney drains, etc., plus rapidly draining backfill as described in Section 15-3.7 below the level of inundation, if wall backfill is needed.
15-4.14 Utilities

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

15-4.15 Guardrail and Barrier

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

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

(a) Pressure distribution for reinforcement pullout

(b) Pressure distribution for reinforcement rupture.
15-5  Wall Type Specific Design Requirements

15-5.1  Abutments

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

15-5.2  Nongravity Cantilever and Anchored Walls

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

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

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

15-5.2.1  Nongravity Cantilever Walls

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

In general, the drilled hole for the soldier piles for nongravity cantilever walls will be filled with a relatively low strength flowable material such as controlled density fill (CDF), provided that water is not present in the drilled hole. Since CDF has a relatively low cement content, the cementitious material in the CDF has a tendency to wash out when placed through water. If the CDF becomes too weak because of this, the design assumption that the full width of the drilled hole, rather than the width of the soldier pile by itself, governs the development of the passive resistance in front of the wall will become invalid. The presence of groundwater will affect the choice of material specified by the structural designer to backfill the soldier pile holes, e.g., CDF if the hole is not wet, or higher strength concrete designed for tremie applications. Therefore, it is important that the geotechnical designer identify the potential for ground water in the drilled holes during design, as the geotechnical stability of a nongravity cantilever soldier pile wall is governed by the passive resistance available in front of the wall.
Typically, when discrete vertical elements are used to form the wall, it is assumed that due to soil arching, the passive resistance in front of the wall acts over three pile/shaft diameters. For typical site conditions, this assumption is reasonable. However, in very soft soils, that degree of soil arching may not occur, and a smaller number of pile diameters (e.g., 1 to 2 diameters) should be assumed for this passive resistance arching effect. For soldier piles placed in very dense soils, such as glacially consolidated till, when CDF is used, the strength of the CDF may be similar enough to the soil that the full shaft diameter may not be effective in mobilizing passive resistance. In that case, either full strength concrete should be used to fill the drilled hole, or only the width of the soldier pile should be considered effective in mobilizing passive resistance.

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

15-5.2.2 Anchored/Braced Walls

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

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

15-5.2.3 Permanent Ground Anchors

The geotechnical designer shall define the no-load zone for anchors in accordance with the AASHTO LRFD Bridge Design Specifications. If the ground anchors are installed through landslide material or material that could potentially be unstable, the no load zone shall include the entire unstable zone as defined by the actual or potential failure surface plus 5 feet minimum. The contract documents should require the drill hole in the no load zone to be backfilled with a non-structural filler. Contractors may request to fill the drill hole in the no load zone with grout prior to testing and acceptance of the anchor. This is usually acceptable provided bond breakers are present on the strands, the anchor unbonded length is increased by 8 feet minimum, and the grout in the unbonded zone is not placed by pressure grouting methods.
The geotechnical designer shall determine the factored anchor pullout resistance that can be reasonably used in the structural design given the soil conditions. The ground anchors used on the projects shall be designed by the Contractor. Compression anchors (see Sabatini, et al., 1999) may be used, but conventional anchors are preferred by WSDOT.

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

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

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

The AASHTO LRFD Bridge Design Specifications specifically addresses anchor testing. The AASHTO specifications recommend that the test loads used in past allowable stress design practice be reduced by the load factor applicable to the limit state that controls the maximum factored design load for the anchor. For the strength limit state, a load factor $\gamma_{Eh}$ 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:

<table>
<thead>
<tr>
<th>Strength Limit State Controls</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load</td>
</tr>
<tr>
<td>AL</td>
</tr>
<tr>
<td>0.25FDL</td>
</tr>
<tr>
<td>0.50FDL</td>
</tr>
<tr>
<td>0.75FDL</td>
</tr>
<tr>
<td>1.00FDL</td>
</tr>
<tr>
<td>1.15FDL</td>
</tr>
<tr>
<td>1.25FDL</td>
</tr>
<tr>
<td>1.50FDL</td>
</tr>
<tr>
<td>AL</td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th>Strength Limit State Controls</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load</td>
</tr>
<tr>
<td>AL</td>
</tr>
<tr>
<td>0.25FDL</td>
</tr>
<tr>
<td>0.50FDL</td>
</tr>
<tr>
<td>0.75FDL</td>
</tr>
<tr>
<td>1.00FDL</td>
</tr>
<tr>
<td>1.15FDL</td>
</tr>
<tr>
<td>AL</td>
</tr>
</tbody>
</table>
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:

<table>
<thead>
<tr>
<th>Strength Limit State Controls</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load</td>
</tr>
<tr>
<td>AL</td>
</tr>
<tr>
<td>0.25FDL</td>
</tr>
<tr>
<td>0.50FDL</td>
</tr>
<tr>
<td>0.75FDL</td>
</tr>
<tr>
<td><strong>1.00FDL</strong></td>
</tr>
<tr>
<td>AL</td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5*</th>
<th>Cycle 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>AL</td>
<td>AL</td>
<td>AL</td>
<td>AL</td>
<td>AL</td>
<td>AL</td>
</tr>
<tr>
<td>0.25FDL</td>
<td>0.25FDL</td>
<td>0.25FDL</td>
<td>0.25FDL</td>
<td>0.25FDL</td>
<td>Lock-off</td>
</tr>
<tr>
<td>0.50FDL</td>
<td>0.50FDL</td>
<td>0.50FDL</td>
<td>0.50FDL</td>
<td>0.50FDL</td>
<td></td>
</tr>
<tr>
<td>0.75FDL</td>
<td>0.75FDL</td>
<td>0.75FDL</td>
<td>0.75FDL</td>
<td>1.00FDL</td>
<td></td>
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<td></td>
<td>1.00FDL</td>
<td></td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>1.15FDL</td>
<td></td>
</tr>
</tbody>
</table>

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

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

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

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

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

15-5.2.4 **Deadmen**

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

**Effect of Anchor Location Relative to the Wall**

- Anchor block left of \( a_c \) provides no resistance.
- Anchor block right of \( a_f \) provides full resistance with no load transferred to wall.
- Anchor block between \( a_c \) and \( a_f \) provides partial resistance and transfers load \( A_1 \) to base of wall.

**Vector Diagram for Free Body**

- Where \( P_a \) = active force on back of \( d_e \) at anchor block.

**Continuous Anchor Wall Located Between Rupture Surface and Slope at Friction Angle**

- Anchor wall right of \( c_c \)
- Anchor wall left of \( c_c \)

**Effect of Depth and Spacing of Anchor Blocks**

- Anchor resistance for \( h_1 \) and \( h_2 \)
- Ultimate \( A_{pc}/d = P_a/2 \), where \( A_{pc}/d \) is anchor resistance, and \( P_a \), \( P_b \) taken per linear foot of wall.
- Individual anchors:
  - If \( d < h_b \), ultimate \( A_{pc}/d = (d-h_b) \) tan \( \phi \), where \( d \) is resultant force of soil at rest on vertical area \( c_d \) or \( c_x \).
  - If \( d > h_b \), ultimate \( A_{pc}/d = A_{pc}/d = (3 A_{pc}/d) \) tan \( \phi \), where \( h_b \) is bearing capacity of strip footing of width \( h_b \) and surcharge load \( y \) (Chapter 4).

**General Requirements:**

1. Allowable value of \( A_p \) and \( A_{pc} \) is ultimate value / 2, factor of safety of 2 against failure.
2. Values of \( k_a \) and \( k_b \) are for cohesionless materials. If backfill has both \( k_a \) and \( k_b \) strengths, compute active and passive forces according to Figures 7 and 9. Fine-grained soils of medium to high plasticity should not be used at the anchorage.
3. Soils within passive wedge of anchorage shall be compacted to no less than 90% of maximum unit weight (ASTM D698 test).
4. Tie rod is designed for allowable \( A_p \) or \( A_{pc} \). The rod connections to wall and anchorage are designed for 1.2 (allowable \( A_p \) or \( A_{pc} \)).
5. Tie rod connection to anchorage is made at the location of the resultant earth pressures acting on the vertical face of the anchorage.
15-5.3 **Mechanically Stabilized Earth Walls**

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

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

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

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

These MSE wall design procedures assume that inextensible reinforcements are not mixed with extensible reinforcements within the same wall. Therefore, MSE walls shall not contain a mixture of inextensible and extensible reinforcements.
### 15-5.3.1 Soil Reinforcement Spacing Considerations

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

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

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

Horizontal spacings as large as 3.3 ft have been used in typical design and construction practice for MSE walls. Back-analysis of instrumented MSE walls indicates that reinforcement load prediction accuracy is not adversely compromised with horizontal spacing of this magnitude when the reinforcement elements are directly attached to rigid facings such as precast concrete panels. However, for flexible facings such as welded wire, large horizontal spacing of the reinforcement has been shown to cause poor wall performance and therefore should not be used for walls with flexible facing. For flexibly faced walls, even a gap of 1.5 ft between reinforcement elements can result in excessive deformation of the facing elements. Therefore, if horizontal gaps of this magnitude are used, the effect of the gaps on the facing panel deformation should be investigated.
15-5.3.2  **Live Load Considerations for MSE Walls**

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

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

15-5.3.3  **Backfill Considerations for MSE Walls**

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

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

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

- **For steel reinforced wall systems, the effect of the higher fines content on corrosion rate of the steel reinforcement** – General practice nationally is that use of backfill with up to 15 percent silt content is acceptable for steel reinforced systems (AASHTO, 2010; Berg, et al., 2009). If higher silt content soils are used, elevated corrosion rates for the steel reinforcement should be considered (see Elias, et al., 2009).

- **Prevention of water or moisture build-up in the wall reinforced backfill** – When the fines content is greater than 5 percent, the material should not be considered to be free draining (see Section 15-3.7). In such cases where the fines content is greater than that allowed in the WSDOT gravel borrow specification (i.e., greater than 7 percent), special measures to prevent water from entering the reinforced backfill shall be implemented. This includes placement of under-drains at the back of the reinforced soil zone, sheet drains to intercept possible ground and rainwater infiltration flow, and use of some type impermeable barrier over the top of the reinforced soil zone.

- **Potential for long-term lateral and vertical deformation of the wall due to soil creep, or in general as cohesive soil shear strength is lost over the life of the wall** – Strain and load increase with time in a steel reinforced soil wall was observed for a large wall in California, a likely consequence of using a backfill soil with a significant cohesion component (Allen, et al., 2001). The Stiffness Method (see Section 15-5.3.10.1, especially Table 15-E-2 in Appendix 15-E) may be used to estimate the reinforcement strain increase caused by loss of cohesive shear strength over time (i.e., estimate the reinforcement strain using the $c-\phi$ 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.

### 15-5.3.4 Compound Stability Assessment for MSE Walls

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

Limit equilibrium analyses (LEA) shall be used to evaluate compound stability. The long-term strength of each backfill reinforcement layer intersected by the failure surface should be considered as resisting forces in the limit equilibrium slope stability analysis.
To perform a LEA for compound stability, three analysis steps are conducted, which are as follows:

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

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

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

Regarding pullout, the length of reinforcement needed behind the critical compound stability failure surface may vary significantly depending on the reinforcement coverage ratio anticipated and the frictional characteristics of the soil reinforcement. Therefore, several scenarios for these two key variables may need to be investigated to assure it is feasible to obtain the desired level of compound stability for all wall/reinforcement types that are to be considered for the selected width “B” of the box. For convenience, to define the box width “B” required for the pullout length, an average active and resistant zone length should be defined for the box. This concept is illustrated in Figure 15-6. In this figure “H” is the total wall height, “T” is the load required in each reinforcement layer that must be resisted to achieve the desired level of safety in the wall for compound stability (Section 15-4.12 applies for compound stability with regard to the slope stability safety factor needed), and $T_{\text{total}}$ 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, $T_{\text{alf}}$, of the reinforcement layers within the defined “box” and the total pullout resistance available for the reinforcement contained within the box, considering factored loads and resistance values. The engineer needs to select the value of “B” that meets this pullout length requirement. However, the value of “B” selected should be minimized to keep the wall base width required to a minimum, to keep excavation needs as small as possible.
From the wall supplier's view, the contract would specify a specific value of "B" that is long enough such that the desired minimum pullout resistance can be obtained but that provides a consistent basis for bidding purposes with regard to the amount of excavation and shoring needed to build the wall.

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

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

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

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

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

- The minimum base width is 0.4H or 6 feet, whichever is greater, where H is the total height of the new wall. Note that for soil reinforcement lengths that are less than 8 feet, the weight and size of construction equipment used to place and compact the soil backfill will need to be limited in accordance with the AASHTO LRFD Bridge Design Specifications Article C11.10.2.1.
• A minimum of two reinforcement layers, or whatever is necessary for stability, shall extend over the top of the existing structure or steep rock face an adequate distance to insure adequate pullout resistance. The minimum length of these upper two reinforcement layers should be 0.7H, 5 feet behind the face of the existing structure or rock face, or the minimum length required to resist the pullout forces applied to those layers, whichever results in the greatest reinforcement length. Note that to accomplish this, it may be necessary to remove some of the top of the existing structure or rock face if the existing structure is nearly the same height as the new wall. The minimum clearance between the top of the existing structure or rock face and the first reinforcement layer extended beyond the top of the existing structure should be 6 in to prevent stress concentrations.

• The MSE wall reinforcements that are truncated by the presence of the existing structure or rock face shall not be directly connected to that existing face, due to the risk of the development of downdrag forces at that interface and the potential to develop bin pressures and higher reinforcement forces (i.e., $T_{\text{max}}$).

• For internal stability design of MSE walls in this situation, see Morrison, et al. (2006). Global and compound stability, both for static (strength limit state) and seismic loading, shall be evaluated, especially to determine the strength and pullout resistance needed for the upper layers that extend over the top of the existing feature. At least one surface that is located at the face of the existing structure but that goes through the upper reinforcement layers shall be checked for both static and seismic loading conditions. That surface will likely be critical for sizing the upper reinforcement layers.

• For new walls with a height over 30 feet, a lateral deformation analysis should be conducted (e.g., using a properly calibrated numerical model). Approval from the State Geotechnical and Bridge Design Engineers is required in this case.

• This type of MSE wall design should not be used to support high volume mainline transportation facilities if the vertical junction between the existing wall or rock face and the back of the new wall is within the traffic lane, especially if there is potential for cracking in the pavement surface to occur due to differential vertical movement at that location unless approved by the State Geotechnical and State Pavement engineers.

Figure 15-7  Example of Steep Shored MSE Wall
15-5.3.6  **MSE Wall Supported Abutments**

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

1. This Geotechnical Design Manual
2. The Bridge Design Manual (Section 7.5).
3. AASHTO LRFD Bridge Design Specifications.

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

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

- Two stage geosynthetic wrapped face geosynthetic walls (i.e., similar to the Standard Plan D-3 wall) with cast-in-place (CIP) or precast concrete full height panels, or shotcrete depending on aesthetic needs,

- Single stage dry-cast concrete modular block faced walls using WSDOT preapproved concrete block – geosynthetic reinforcement combinations (see Appendix 15-D), and

- WSDOT preapproved proprietary MSE walls identified as such (see Appendix 15-D), but only those that are concrete faced. Welded wire faced preapproved MSE walls may be used for temporary bridge abutment applications. However, MSE walls identified in Appendix 15-D as preapproved proprietary walls shall not be considered preapproved for the MSE wall supported bridge abutment application (i.e., a special design is required).

Figures 15-8, 15-9, and 15-10 provide typical sections that should be used in the design of MSE wall bridge abutments. The base of the wall may be truncated to reduce excavation needs subject to the limitations provided in Section 11 of the AASHTO LRFD Bridge Design Specifications. Figure 15-9 is similar to the Standard Plan geosynthetic wall (Standard Plan D-3), except as modified in this figure for this application. This figure does not show all the details needed for the facing design. For the additional facing details needed, see Standard Plans D-3-10 and D-3-11. The minimum tensile strength of the geotextile or geogrid used as bridge approach soil reinforcement in figures 15-8 and 15-9 shall be 2.4 kips/ft in accordance with ASTM D4595 for geotextiles or ASTM D6637 for geogrids. The soil reinforcement and facing design is project specific and shall be completed in accordance with manuals and design policy documents cited at the beginning of this section.
**Figure 15-8** Typical Section for MSE Wall Supported Abutment – Flat Slab Superstructure With no Footing and Dry-Cast Modular Block Wall Facing

**Figure 15-9** Typical Section for MSE Wall Supported Abutment – Flat Slab Superstructure With no Footing and Precast or CIP Concrete Wall Facing (Two Stage Wall Construction)
Figure 15-10  Typical Section Showing External Dimensions for Bridge With Spread Footing Supported Directly on an MSE Wall Semi-Integral Abutment (L-Abutment Similar; Wing/Curtain Wall Not Shown)

A = 4 feet min for SE Walls (precast concrete panel face or cast-in-place concrete face), 2 feet min for special designed geosynthetic retaining walls with wrapped face
B = 3 feet min for I-girder bridges, and 5 feet min for non-I-girder, slab, and box girder bridges
C = 30 feet max

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

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

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

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

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

The potential lateral and vertical deformation of the wall, considering the affect of the footing load on the wall, should be evaluated. Measures shall be taken to minimize potential deformation of the reinforced soil, such as use of high quality backfill such as Gravel Borrow compacted to 95 percent of maximum density. The settlement and lateral deformation of the soil below the wall shall also be included in this deformation analysis. If there is significant uncertainty in the amount of vertical deformation in or below the wall anticipated, the ability to jack the abutment to accommodate unanticipated abutment settlement should also be considered in the abutment design.
15-5.3.7 Full Height Propped Precast Concrete Panel MSE Walls

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

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

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

15-5.3.8 Flexible Faced MSE Walls With Vegetation

If a vegetated face is to be used with an MSE wall, the exposed (i.e., above ground wall height shall be limited to 20 feet or less, and the wall face batter shall be no steeper than 1H:6V, unless the facing is battered at 1H:2V or flatter, in which case the maximum height could be extended to 30 feet). A flatter facing batter may be needed depending on the wall system – see appendices to this GDM chapter for specific requirements.

For the vegetated facing, if the facing batter is steeper, or if the height is greater than specified here, the compressibility of the facing topsoil could create excessive stresses, settlement, and/or bulging in the facing, any of which could lead to facing stability or deformation problems.

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

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

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

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

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

- Total settlement is limited to 2 in and differential settlement is limited to 1.5 inch as identified in Table 15-3. Since this is specified in Table 15-3, this also applies to shorter walls.
- A concrete leveling pad is placed below the first lift of blocks to provide a uniform flat surface for the blocks. Note that this should be done for all preapproved block faced walls regardless of height.
• A moderately compressible bearing material is placed between each course of blocks, such as a geosynthetic reinforcement layer. The layer must provide an even bearing surface (many polyester geogrids or multi-filament woven geotextiles provide an adequately even bearing surface with sufficient thickness and compressibility to distribute the bearing load between blocks evenly). The bearing material needs to extend from near the front edge of the blocks (without protruding beyond the face) to at least the back of the blocks or a little beyond. As a minimum, this should be done for all block lifts that are 25 feet or more below the wall top, but doing this for block lifts at depths of less than 25 feet as well is desirable.

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

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

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

15-5.3.10 **Internal Stability Using the Stiffness Method**

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

15-5.3.10.1 **Determination of T\text{max} Using the Stiffness Method**

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

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

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

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

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

$$T_{\text{max}} = S_v [H \gamma_r D_{\text{max}} + \gamma_r (H_{\text{ref}} / H)S]k_{avh} \Phi$$

where,

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

$D_{\text{max}}$ shall be determined as follows:

For $z < z_b$:

$$D_{\text{max}} = D_{\text{max}0} + (z/z_b)(1 - D_{\text{max}0})$$

(15-2)

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

$$z_b = C_h (H)^{1.2}$$

(15-3)

where,

- $z$ = depth of reinforcement layer below top of wall at wall face (ft)
- $z_b$ = depth below top of wall at wall face where $D_{\text{max}}$ becomes equal to 1.0 (and below which $D_{\text{max}}$ equals 1.0) (ft)
- $D_{\text{max}0}$ = $T_{\text{max}}$ distribution factor magnitude at top of wall at wall face, equal to 0.12 (dim)
- $C_h$ = coefficient equal to 0.32 when $H$ is in ft and 0.40 when $H$ is in meters
Determination of the $T_{\text{max}}$ distribution factor $D_{\text{tmax}}$ is illustrated in Figure 15-12. In the figure, depths below the wall top have been normalized by the wall height, $H$. $T_{\text{mxmx}}$ is the maximum value of $T_{\text{max}}$ in the wall section where the soil backfill failure surface crosses the reinforcement layers.

**Figure 15-12** Illustration of $D_{\text{tmax}}$ factor for the Stiffness Method

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

$$\Phi = \Phi_g \Phi_{fs}$$  \hspace{1cm} (15-4)

where,

$\Phi_g$ = global stiffness factor (dim)

$\Phi_{fs}$ = facing stiffness factor (dim)

The global stiffness factor $\Phi_g$ shall be determined as follows:

$$\Phi_g = \alpha \left( \frac{S_{global}}{P_a} \right)^\beta$$  \hspace{1cm} (15-5)

where,

$\alpha$ = empirical coefficient = 0.16

$\beta$ = empirical exponent = 0.26

$S_{global}$ = global reinforcement stiffness (ksf)

$P_a$ = atmospheric pressure at sea level (equals 2.11 ksf if $S_{global}$ is in ksf, or 101 kPa if $S_{global}$ is in kPa)

and,
\[ S_{\text{global}} = \frac{J_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} R_c J_i}{H} \]  

(15-6)

where,
- \( J_{\text{ave}} \) = average secant tensile creep stiffness corrected for the coverage ratio, i.e., \( R_c J_i \) of all \( n \) reinforcement layers (kips/ft)
- \( J_i \) = secant tensile creep stiffness of reinforcement layer \( i \) per unit of reinforcement width (kips/ft)
- \( R_c \) = reinforcement coverage ratio (dim)
- \( n \) = number of reinforcement layers in wall section (dim)

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

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

The facing stiffness factor \( \Phi_{\text{fs}} \) shall be determined as follows:

\[ \Phi_{\text{fs}} = \eta \left( \frac{S_{\text{global}}}{P_a} \right)^{F_f} \]  

(15-7)

where,
- \( \eta \) = empirical coefficient = 0.57
- \( \kappa \) = empirical exponent = 0.15
- \( F_f \) = facing stiffness parameter as calculated using Equation 15-8 (dim)

\[ F_f = \frac{1.5H^3 P_a}{E b^3 (h_{\text{eff}} / H)} \]  

(15-8)

where,
- \( E \) = elastic modulus of the "equivalent elastic beam" representing the wall face (ksf)
- \( b \) = thickness of the facing column (ft)
- \( h_{\text{eff}} \) = equivalent height of an un-jointed facing column that is approximately 100% efficient in transmitting moment through the height of the facing column (ft)

All other variables are as defined previously.

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

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

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

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

$$\Phi = \Phi_g \Phi_{fs} \Phi_{fb} \Phi_{local} \Phi_c$$

(15-9)

where,

- $\Phi_g$ = global stiffness factor (dim)
- $\Phi_{fs}$ = facing stiffness factor (dim)
- $\Phi_{fb}$ = facing batter factor (dim)
- $\Phi_{local}$ = local stiffness factor (dim)
- $\Phi_c$ = soil cohesion factor (dim)

$\Phi_g$ and $\Phi_{fs}$ are determined as shown in equations 15-5 and 15-7. $\Phi_{fb}$ shall be determined as follows:

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

(15-10)

where,

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

For both determinations of the coefficient of active lateral earth pressure, wall friction is assumed to be zero.
The local stiffness factor, $F_{local}$, shall be determined as follows:

$$F_{local} = \left( \frac{S_{local}}{S_{localave}} \right)^a$$  \hspace{1cm} (15-11)

where,

- $a = \text{empirical exponent} = 0.50$ for extensible reinforcement (e.g., geotextiles, geogrids, polymer straps)
- $S_{local} = \text{local reinforcement stiffness determined as follows:}$

$$S_{local} = \frac{R_C J}{S_v}$$  \hspace{1cm} (15-12)

where,

- $R_C, J$, and $S_v$ are as defined previously

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

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

where,

- all variables are as defined previously.

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

The soil cohesion factor, $F_{c}$, shall be determined as follows:

$$F_c = e^{\lambda (c/(\gamma_T H))}$$  \hspace{1cm} (15-13)

where,

- $e = \text{base for the natural logarithm, equal to approximately 2.718...}$
- $\lambda = \text{empirical coefficient within exponent} = -16$
- $c = \text{cohesion of MSE wall backfill (psf)}$

All other variables are as defined previously.

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

Conceptually, the Stiffness Method was developed by starting with the Simplified Method, but modifying that method empirically to improve its accuracy, considering the stiffness of the wall components, and improving the distribution of $T_{max}$ as a function of depth in the wall to more accurately reflect full scale wall measurements. Figure 15-13 illustrates the relationship between the Simplified Method and the Stiffness Method.
### 15-5.3.10.2 Load and Resistance Factors for the Stiffness Method

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

<table>
<thead>
<tr>
<th>Limit State(^1)</th>
<th>Reinforcement Type</th>
<th>Load Factor, ( Y_{p-EV} ) and ( Y_{p-EVc} )</th>
<th>Live Load(^2), ( Y_{LL} )</th>
<th>Resistance Factor, ( \Phi_{rr} ), ( \Phi_{cr} ), ( \Phi_{po} ), and ( \Phi_{sf} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement rupture, ( Y_{p-EV} ) and connection failure, ( Y_{p-con} ) (strength limit)</td>
<td>Geogrids and geotextiles</td>
<td>1.35</td>
<td>1.75</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>4Polymer straps</td>
<td>1.35</td>
<td>1.75</td>
<td>0.55</td>
</tr>
<tr>
<td>Soil failure, ( Y_{p-EVsf} ) (service limit)</td>
<td>All geosynthetics</td>
<td>1.20</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Pullout, ( Y_{p-EV} ) (strength limit - default model in AASHTO 2020)(^3)</td>
<td>All geosynthetics</td>
<td>1.35</td>
<td>N/A</td>
<td>0.70</td>
</tr>
</tbody>
</table>

Notes:

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

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

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

4. Also termed geostraps.
15-5.3.10.3 **Design for Internal Stability Limit States Using the Stiffness Method**

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

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

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

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

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

$$\varepsilon_{\text{rein}} = \frac{\gamma_{P\text{-EVsf}} T_{\text{max}}}{\phi_{sf} R_c J_i} \leq \varepsilon_{\text{mmax}}$$  \hspace{1cm} (15-14)

where,
- $\varepsilon_{\text{rein}}$ = the reinforcement strain in any individual reinforcement layer corresponding to $T_{\text{max}}$ (%)
- $\gamma_{P\text{-EVsf}}$ = load factor for prediction of $T_{\text{max}}$ for the soil failure limit state in Table 15-5 (dim)
- $T_{\text{max}}$ = the maximum load in the reinforcement at each reinforcement level, as specified in Section 15-5.3.10.1 (kips/ft)
- $\phi_{sf}$ = resistance factor that accounts for uncertainty in the measurement of the reinforcement stiffness at the specified strain, as specified in Table 15-5 (dim)
- $R_c$ = reinforcement coverage ratio (dim)
- $J_i$ = secant tensile stiffness of reinforcement layer i per unit of reinforcement width (kips/ft)
- $\varepsilon_{\text{mmax}}$ = maximum acceptable strain in the wall cross-section corresponding to $T_{\text{max}}$ in any reinforcement layer (%)
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If multiple load sources are acting on the reinforced soil backfill, they shall be added to $T_{\text{max}}$ as determined using Equation 15-1 by using superposition.

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

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

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

15.5.3.10.3.2 Pullout Limit State (Strength I)

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

$$L_e \geq \frac{\gamma_{p-EV} T_{\text{max}}}{\phi_{po} F^* \alpha \sigma_v C R_c}$$  \hspace{1cm} (15-15)

where,

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

$$T_{\text{totalf}} = \gamma_{p-EV}T_{\text{max}} + \gamma_{p-ES}S_v(k_a\Delta\sigma_v + \Delta\sigma_H)$$

where,

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

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

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

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

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

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

where,

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

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

$$T_{\text{totalf}} = \gamma_{p-EV}T_{\text{max}} + (\gamma_{LS})\gamma T_{\text{eq}} < \phi T_{al}R_c$$

where,

- $T_{\text{totalf}}$ = total factored load for each reinforcement layer (lbs/ft)
- $\gamma_{LS}$ = load factor for live load surcharge, LS, as specified in the AASHTO LRFD Bridge Design Manual, Table 3.4.1-1 (dim.)
- $\gamma$ = unit weight of soil used to calculate live load surcharge, LS (lbs/ft$^3$)
- $T_{\text{eq}}$ = equivalent height of soil for live load surcharge (ft)
If multiple load sources other than traffic live load are present, use Equation 15-16 to determine $T_{\text{total}}$. It follows that if these additional load sources are added by superposition for the Strength limit state design, that these additional load sources should also be added by superposition to the Service limit state value of $T_{\text{max}}$, in Equation 15-14. However, doing so is likely to be excessively conservative, especially for typical loads used for bridge footings. If such foundation loads are present above the reinforced soil portion of the wall, it may be best to design the geosynthetic wall using the Simplified Method or using limit equilibrium as included in the AASHTO LRFD Bridge Design Specifications.

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

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

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

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

### 15-5.4 Prefabricated Modular Walls

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

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

### 15-5.5 Rock Walls

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

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

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

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

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

### 15-5.6 Reinforced Slopes

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

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


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

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

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

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

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

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

Walls underpinning structures such as bridges and retaining walls shall have double corrosion protected (encapsulated) nails within the zone of influence of the structure being retained or supported.
Furthermore, nails installed in soils with strong corrosion potential, defined as:

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

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

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


15-6 Standard Plan Walls

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

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

- $A_s = 0.51g$ for Wall Types 1 through 4, and 0.20g for Wall Types 5 through 8. For sliding stability, the wall is allowed to slide 4 in to calculate $k_h$ from $A_s$ using a Newmark deformation analysis, or a simplified version of it.
- For the wall Backfill, $\varphi = 36^\circ$ and $\gamma = 130 \text{ pcf}$.
- For the foundation soil, for sliding stability analysis, $\varphi = 32^\circ$.
- 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, $\varphi = 38^\circ$ and $\gamma = 130$ pcf.
• For the foundation soil, for sliding stability analysis, $\varphi = 36^\circ$, and interface friction angle of $0.7 \times 36^\circ = 25^\circ$.
• For the retained soil behind the soil reinforcement, for external stability analysis, $\varphi = 36^\circ$ and $\gamma = 130$ pcf.
• Wall settlement criteria are as specified in Table 15-2, unless the settlement of the first stage wall (i.e., the geosynthetic wall without the final concrete fascia) is complete before the final concrete fascia is installed, in which case the settlement criteria in Table 15-4 may be used).

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

15-7 Temporary Cut Slopes and Shoring

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

15-7.1 Overview

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

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

Examples of instances where temporary slopes may be necessary include:

• Situations where there is adequate room to construct a stable temporary slope in lieu of shoring.
• Excavations behind temporary or permanent retaining walls.
• Situations where a combination of shoring and temporary excavation slopes can be used.
• Removal of unsuitable soil adjacent to an existing roadway or structure;
• Shear key construction for slide stabilization.
• Culvert, drainage trench, and utility construction, including those where trench boxes are used.
The primary difference between temporary shoring/cut slopes/cofferdams, hereinafter referred to as temporary shoring, and their permanent counterparts is their design life. Typically, the design life of temporary shoring is the length of time that the shoring or cut slope are required to construct the adjacent, permanent facility. Because of the short design life, temporary shoring is typically not designed for seismic loading, and corrosion protection is generally not necessary. Additionally, more options for temporary shoring are available due to limited requirements for aesthetics. Temporary shoring is typically designed by the contractor unless the contract plans include a detailed shoring design. For contractor designed shoring, the contractor is responsible for internal and external stability, as well as global slope stability, soil bearing capacity, and settlement of temporary shoring walls.

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

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

15-7.2 Geotechnical Data Needed for Design

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

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

15-7.3 General Design Requirements

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

15-7.3.1 Design Procedures

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

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

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

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

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

If temporary excavation slopes are required to install the shoring system, the stability of the temporary excavation slope shall be assessed and stabilized.
15-7.3.2 Safety Factors/Resistance Factors

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

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

15-7.3.3 Design Loads

The active, passive, and at-rest earth pressures used to design temporary shoring shall be determined in accordance with the procedures outlined in Article 3.11.5 of the AASHTO LRFD Bridge Design Specifications or Section 5 of the AASHTO Standard Specifications for Highway Bridges (2002) for wall types in which LRFD procedures are not available.

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

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

The design of temporary shoring must also take into account the loading and destabilizing effect caused by excavation dewatering.
15-7.3.4 **Design Property Selection**

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

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

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

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

15-7.4 **Special Requirements for Temporary Cut Slopes**

Temporary cuts slopes are used extensively in construction due to the ease of construction and low costs. Since the contractor has control of the construction operations, the contractor is responsible for the stability of cut slopes, as well as the safety of the excavations, unless otherwise specifically stated in the contact documents. Because excavations are recognized as one of the most hazardous construction operations, temporary cut slopes must be designed to meet Federal and State regulations in addition to the requirements stated in the GDM. Federal regulations regarding temporary cut slopes are presented in Code of Federal Regulations (CFR) Part 29, Sections 1926. The State of Washington regulations regarding temporary cut slopes are presented in Part N of WAC 296-155. Key aspects of the WAC with regard to temporary slopes are summarized below for convenience. To assure obtaining the most up to date requirements regarding temporary slopes, the WAC should be reviewed.
**WAC 296-155** presents maximum allowable temporary cut slope inclinations based on soil or rock type, as shown in Table 15-7. **WAC 296-155** also presents typical sections for compound slopes and slopes combined with trench boxes. The allowable slopes presented in the WAC are applicable to cuts 20 feet or less in height. The WAC requires that slope inclinations steeper than those specified by the WAC or for slope heights greater than 20 feet, as well as slopes in soils or rock not meeting the requirements to be classified as stable rock, or Type A, B, or C soil, shall be designed by a registered professional engineer. As a minimum, the design by or under the supervision of the registered professional engineer shall include a geotechnical slope stability analysis (i.e., Chapter 7) that is based on a knowledge of the subsurface conditions present, including soil and rock stratigraphy, engineering data that can be used to estimate soil and rock properties, and ground water conditions, and with consideration to the loading conditions on or above the slope that could affect its stability. The design shall be conducted in accordance with the requirements in this GDM and referenced documents. Engineering recommendations based upon field observations alone shall not be considered to be an engineering design as defined in the WAC and this GDM.

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

<table>
<thead>
<tr>
<th>Soil or Rock Type</th>
<th>Maximum Allowable Temporary Cut Slopes (20 Feet Maximum Height)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stable Rock</td>
<td>Vertical</td>
</tr>
<tr>
<td>Type A Soil</td>
<td>¾H:1V</td>
</tr>
<tr>
<td>Type B Soil</td>
<td>1H:1V</td>
</tr>
<tr>
<td>Type C Soil</td>
<td>1⅛H:1V</td>
</tr>
</tbody>
</table>

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

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

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

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

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

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

15-7.5 Performance Requirements for Temporary Shoring and Cut Slopes

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

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

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

In the case of cantilever walls, the resistance factor of 0.75 applied to the passive resistance accounts for variability in properties and other sources of variability, as well as the prevention of excess deformation to fully mobilize the passive resistance. The amount of deformation required to mobilize the full passive resistance typically varies from 2 to 6 percent of the exposed wall height, depending on soil type in the passive zone (AASHTO 2017).
15-7.6 Special Design Requirements for Temporary Retaining Systems

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

15-7.6.1 Fill Applications

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

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

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

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

15-7.6.1.1 MSE Walls

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

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

Alternatively, for geosynthetic reinforcement, a default combined reduction factor for creep, durability, and installation damage in accordance with the AASHTO specifications (LRFD or Standard Specifications) may be used, ranging from a combined reduction factor RF of 4.0 for walls with a life of up to three years, to 3.0 for walls with a one-year life, to 2.5 for walls with a six month life. If steel reinforcement is used for temporary MSE walls, the reinforcement is not required to be galvanized, and the loss of steel due to corrosion is estimated in consideration of the anticipated wall design life.
15-7.6.1.2  Prefabricated Modular Block Walls

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

15-7.6.2  Cut Applications

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

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

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

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

15-7.6.2.1  Trench Boxes

In accordance with the WSDOT Standard Specifications, trench boxes are not considered to be structural shoring, as they generally do not provide full lateral support to the excavation sides. Trench boxes are not appropriate for excavations that are deeper than the trench box. Generally, detailed analysis is not required for design of the system; however, the contractor should be aware of the trench box’s maximum loading conditions for situations where surcharge loading may be present, and should demonstrate that the maximum anticipated lateral earth pressures will not exceed the structural capacity of the trench box. Geotechnical information required to determine whether trench boxes are appropriate for an excavation include the soil type, density, and groundwater conditions. Also, where existing improvements are located near the excavation, the soil should exhibit adequate standup time to minimize the risk of damage as a result of caving soil conditions against the outside of the trench box. In accordance with sections 15-7.3 and 15-7.4, the excavation slopes outside of the trench box shall be designed to be stable.
15-7.6.2.2  **Sheet Piling, with or without Ground Anchors**

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

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

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

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

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

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

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

The ground anchors for temporary walls do not need special corrosion protection if the wall design life is three years or less. However, the anchor bar or steel strand section shall be designed for the anticipated corrosion loss that could occur during the wall design life. Easements may be required if the ground anchors, if used, extend outside the right of way/property boundary.
15-7.6.2.4  **Prefabricated Modular Block Walls**

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

15-7.6.2.5  **Braced Cuts**

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

15-7.6.2.6  **Soil Nail Walls**

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

15-7.6.3  **Uncommon Shoring Systems for Cut Applications**

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

- Diaphragm/slurry walls
- Secant pile walls
- Cellular 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-F.

15-7.7  **Shoring and Excavation Design Submittal Review Guidelines**

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

1.  **Shoring System Geometry**
   a.  Has the shoring geometry been correctly developed, and all pertinent dimensions shown?
   b.  Are the slope angle and height above and below the shoring wall shown?
   c.  Is the correct location of adjacent structures, utilities, etc., if any are present, shown?
2. Performance Objectives for the Shoring System
   a. Is the anticipated design life of the shoring system identified?
   b. Are objectives regarding what the shoring system is to protect, and how to protect it, clearly identified?
   c. Does the shoring system stay within the constraints at the site, such as the right of way limits, boundaries for temporary easements, etc?

3. Subsurface conditions
   a. Is the soil/rock stratigraphy consistent with the subsurface geotechnical data provided in the contract boring logs?
   b. Did the contractor/shoring designer obtain the additional subsurface data needed to meet the geotechnical exploration requirements for slopes and walls as identified in chapters 10 and 15, respectively, and Appendix 15-F for unusual shoring systems?
   c. Was justification for the soil, rock, and other material properties used for the design of the shoring system provided, and is that justification, and the final values selected, consistent with Chapter 5 and the subsurface field and lab data obtained at the shoring site?
   d. Were ground water conditions adequately assessed through field measurements combined with the site stratigraphy to identify zones of ground water, aquitards and aquicludes, artesian conditions, and perched zones of ground water?

4. Shoring system loading
   a. Have the anticipated loads on the shoring system been correctly identified, considering all applicable limit states?
   b. If construction or public traffic is near or directly above the shoring system, has a minimum traffic live load surcharge of 250 psf been applied?
   c. If larger construction equipment such as cranes will be placed above the shoring system, have the loads from that equipment been correctly determined and included in the shoring system design?
   d. If the shoring system is to be in place longer than three years, have seismic and other extreme event loads been included in the shoring system design?

5. Shoring system design
   a. Have the correct design procedures been used (i.e., the GDM and referenced design specifications and manuals)?
   b. Have all appropriate limit states been considered (e.g., global stability of slopes above and below wall, global stability of wall/slope combination, internal wall stability, external wall stability, bearing capacity, settlement, lateral deformation, piping or heaving due to differential water head)?
6. Are all safety factors, or load and resistance factors for LRFD shoring design, identified, properly justified in a manner that is consistent with the GDM, and meet or exceed the minimum requirements of the GDM?

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

8. Shoring System Monitoring/Testing
   a. Is a monitoring/testing plan provided to verify that the performance of the shoring system is acceptable throughout the design life of the system?
   b. Have appropriate displacement or other performance triggers been provided that are consistent with the performance objectives of the shoring system?

9. Shoring System Removal
   a. Have any elements of the shoring system to be left in place after construction of the permanent structure is complete been identified?
   b. Has a plan been provided regarding how to prevent the remaining elements of the shoring system from interfering with future construction and performance of the finished work (e.g., will the shoring system impede flow of ground water, create a hard spot, create a surface of weakness regarding slope stability)?

15-8 References


US Department of Defense, 2005, Soil Mechanics, Unified Facilities Criteria (UFC), UFC 3-220-10N.


15-9 Appendices

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

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

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

Appendix 15-D Preapproved Proprietary Wall Systems

Appendix 15-E MSE Wall Design Using the Stiffness Method

Appendix 15-F Description of Typical Temporary Shoring Systems and Selection Considerations

Appendix 15-G Testing and Acceptance Protocols for Tiebacks in Clay

Appendix 15-H Preapproved Wall Appendix: Specific Requirements and Details for Hilfiker Welded Wire Faced Walls

Appendix 15-I Preapproved Wall Appendix: Specific Requirements and Details for Eureka Reinforced Soil Concrete Panel Walls

Appendix 15-J Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

Appendix 15-K Preapproved Wall Appendix: Specific Requirements and Details for Tensar ARES Walls

Appendix 15-L Preapproved Wall Appendix: Specific Requirements and Details for Tensar MESA Walls

Appendix 15-M Preapproved Wall Appendix: Specific Requirements and Details for Tensar Welded Wire Form Walls

Appendix 15-N Preapproved Wall Appendix: Specific Requirements and Details for SSL Concrete Panel Walls

Appendix 15-O Preapproved Wall Appendix: Specific Requirements and Details for Landmark Reinforced Soil Wall

Appendix 15-P Preapproved Wall Appendix: Specific Requirements and Details for Allan Block Walls

Appendix 15-Q Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls

Appendix 15-R Preapproved Wall Appendix: Specific Requirements and Details for Lock and Load Walls

Appendix 15-S Preapproved Wall Appendix: Specific Requirements and Details for KeyGrid Walls

Appendix 15-T Preapproved Wall Appendix: Specific Requirements and Details for Basalite GEOWALL
Appendix 15-A  Preapproved Proprietary Wall and Reinforced Slope General Design Requirements and Responsibilities

15-A-1  Design Requirements

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

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

15-A-2  Design Responsibilities

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

- **Soil pH** (determined by AASHTO T289) = 4.5 to 9 for permanent applications and 3 to 10 for temporary applications.
• Maximum soil particle size ≤ 1.25 inches, unless full scale installation damage tests are conducted in accordance with AASHTO R 69 so that the design can take into account the potential greater degree of damage.

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

15-A-4 MSE Wall Facing Tolerances

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

1. Deviation from the design batter and horizontal alignment, when measured along a 10 feet straight edge, shall not exceed the following:
   a. Welded wire faced structural earth wall: 2 inches
   b. Precast concrete panel and concrete block faced structural earth wall: ¾ inch

2. Deviation from the overall design batter of the wall shall not exceed the following per 10 feet of wall height:
   a. Welded wire faced structural earth wall: 1.5 inches
   b. Precast concrete panel and concrete block faced structural earth wall: ½ inch

3. The maximum outward bulge of the face between welded wire faced structural earth wall reinforcement layers shall not exceed 2 inches. The maximum allowable offset in any precast concrete facing panel joint shall be ¾ inch. The maximum allowable offset in any concrete block joint shall be ⅜ inch.

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

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

15-A-5 References


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

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

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

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

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

1. External stability design
   a. Are the structure dimensions, and design cross-sections, in the wall/reinforced slope supplier’s plan consistent with the contract requirements and geotechnical design? As a minimum, check wall/slope base width, embedment depth, and face batter in comparison to the geotechnical external stability design. (GT, ST).
   b. Have the design documents and plan details been certified in accordance with this manual? (GT, ST)
2. **Internal stability design**
   a. Has the correct, and agreed upon, design procedure been used (i.e., as specified in the GDM, BDM, and AASHTO LRFD Specifications), including the correct earth pressures and earth pressure coefficients? (GT)
   b. Has appropriate load group for each limit state been selected? (GT, ST)
      i. In general, with the exception of the Stiffness Method described in Section 15.5.3.10.3.1 the service limit state is not specifically checked for internal stability.
      ii. Strength I should be used for the strength limit state, unless an owner specified vehicle is to be used, in which case Strength II should also be checked.
      iii. Extreme Event I should be used for seismic design.
      iv. Extreme Event II should be used for scour design.
   c. Have the correct load factors been selected (see GDM, BDM and the AASHTO LRFD Specifications)? Note that for reinforced slopes, since LRFD procedures are currently not available, load factors are not applicable to reinforced slope design. (GT, ST)
   d. Has live load been treated correctly regarding magnitude (in general, approximated as 2 feet of soil surcharge load) and location (over reinforced zone for bearing, behind reinforced zone for sliding and overturning)? (GT, ST)
   e. Have the effects of any external surcharge loads, including traffic barrier impact loads, been taken into account in the calculation of load applied internally to the wall reinforcement and other elements? (GT, ST)
   f. Has the correct PGA been used for seismic design for internal stability? (GT)
   g. Have the correct resistance factors been selected for design for each limit state? For reinforced slopes, since LRFD design procedures are currently not available, check to make sure that the correct safety factors have been selected. (GT)
   h. Have the correct reinforcement and connector properties been used?
      i. For steel reinforcement, have the steel reinforcement dimensions and spacing been identified? (GT, ST)
      ii. For steel reinforcement, has it been designed for corrosion using the correct corrosion rates, correct design life (75 years, unless specified otherwise in the contract documents)? (GT, ST)
      iii. Have the steel reinforcement connections to the facing been designed for corrosion, and has appropriate separation between the soil reinforcement and the facing concrete reinforcement been done so that a corrosion cell cannot occur, per the AASHTO LRFD Specifications? (GT, ST)
iv. For geosynthetic reinforcement products selected, are the long-term design nominal strengths, $T_{al}$, used for design consistent with the values of $T_{al}$ provided in the Qualified Products List (QPL) and consistent with the products approved for the particular wall system in this GDM. (GT)

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

vi. If a coverage ratio, $R_c$, 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 the Stiffness Method is used, soil failure at the strength limit state (GT)

j. If obstructions such as small structure foundations, culverts, utilities, etc., must be placed within the reinforced backfill zone (primarily applies to MSE walls and reinforced slopes), has the design of the reinforcement placement, density and strength, and the facing configuration and details, to accommodate the obstruction been accomplished in accordance with the GDM, BDM, and AASHTO LRFD Specifications? (GT, ST)
k. Has the computer output for internal stability been hand checked to verify the accuracy of the computer program calculations (compare hand calculations to the computer output; also, a spot check calculation by the reviewer may also be needed if the calculations do not look correct for some reason)? (GT)

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

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

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

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

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

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

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

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

   b. Make sure any wall/slope corner or angle point details are consistent with the preapproved details and the contract requirements, both regarding the facing and the soil reinforcement. This also applies to overlap of reinforcement for back-to-back walls (GT, ST)
4. Wall and reinforced slope construction quality assurance
   a. Discuss all aspects of the wall/slope construction and quality assurance activities at the wall/reinforced preconstruction meeting. The preconstruction meeting should include representatives from the wall supplier and related materials suppliers, the earthwork contractor, the wall constructor, the prime contractor, the project inspection and construction administration staff, and the geotechnical and structural reviewers/designers. (GT, ST, and region project office)
   b. Check to make sure that the correct wall or reinforced slope elements, including specific soil reinforcement products, connectors, facing blocks, etc., are being used to construct the wall (visually check identification on the wall elements). For steel systems, make sure that reinforcement dimensions are correct, and that they have been properly galvanized. (region project office)
   c. Make sure that all wall elements are not damaged or otherwise defective. (region project office)
   d. Make sure that all materials certifications reflect what has been shipped to the project and that the certified properties meet the contract/design requirements. Also make sure that the identification on the wall elements shipped to the site match the certifications. Determine if the date of manufacture, production lot number, and piece mark on the rear face of each panel match the identification of the panels shown on the shop drawings (if req. by special prov.) (region project office)
   e. Obtain samples of materials to be tested, and compare test results to project minimum requirements. Also check dimensional tolerances of each wall element. (region project office)
   f. Make sure that the wall backfill meets the design/contract requirements regarding gradation, ability to compact, and aggregate durability. (region project office)
   g. Check the bearing pad elevation, thickness, and material to make sure that it meets the specifications, and that its location relative to the ground line is as assumed in the design. Also check to make sure that the base of the wall excavation is properly located, and that the wall base is firm. (region project office)
   h. As the wall is being constructed, make sure that the right product is being used in the right place. For soil reinforcement, make sure that the product is the right length, spaced vertically and horizontally correctly per the plans, and that it is placed and pulled tight to remove any slack or distortion, both in the backfill and at the facing connection. Make sure that the facing connections are properly and uniformly engaged so that uneven loading of the soil reinforcement at the facing connection is prevented. (region project office)
   i. Make sure that facing panels or blocks are properly seated on one another as shown in the wall details. (region project office)
j. Check to make sure that the correct soil lift thickness is used, and that backfill compaction is meeting the contract requirements. (region project office)

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

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

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

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

o. For reinforced slopes, in addition to what is listed above as applicable, check to make sure that the slope facing material is properly connected to the soil reinforcement. Also check that secondary reinforcement is properly placed, and that compaction out to the slope surface is accomplished. (region project office)
Appendix 15-C  Wall/Reinforced Slope Systems
Evaluation: Submittal Requirements

15-C-1 Instructions

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

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

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

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

15-C-2 Part One: Wall System Overview

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

15-C-3 Part Two: Plan Details

As a minimum, provide the following plan sheet details:

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

6. Traffic barrier systems
   • Guardrail
   • Moment slab barrier

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

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

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

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

11. Coping and gutter details.

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

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

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

For materials not currently covered by WSDOT specifications, provide material specifications describing the material type, quality, certifications, lab and field testing, acceptance and rejection criteria along with support information for each material items. Include representative test results (lab and/or field) clearly referencing the date, source and method of test, and, where required, the method of interpretation and/or extrapolation. Along with the source of the supplied information, include a listing of facilities normally used for testing (i.e., in-house and independent).
All geosynthetic reinforced wall systems shall use a soil reinforcement product listed in the WSDOT Qualified Product List (QPL). Inclusion of geosynthetic reinforcement products on the QPL will be a necessary prerequisite to wall system approval.

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

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

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

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

4. For the coping, provide the following information:
   • Dimensions and tolerances
   • Material used (including any reinforcement)
   • Method/details to attach coping to wall top
5. For the traffic railing/barrier, provide the following information:
   • Dimensions of precast and cast-in-place barriers and reaction slabs
   • How barrier/railing is placed on/in and/or attached to wall top
   • How guard railing is placed on/in and/or attached to wall top

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

15-C-5 Part Four: Design

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

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

For internal stability design, provide design procedures, assumptions, and any deviations from the design standards identified above required to design the wall or reinforced system for each of the design issues: listed below. Note that some of these design issues are specific to structural earth wall or reinforced slope design and may not be applicable to other wall types.
1. Assumed failure surface used for design
2. Distribution of horizontal stress
3. How surcharge loads are handled in design
   - Concentrated dead load
   - Sloped surcharge
   - Broken-back surcharge
   - Live load
   - Traffic impact
4. Determination of the long-term tensile strength of reinforcement
5. Pullout design of soil reinforcement or facing components that protrude into wall backfill
6. Determination of vertical and horizontal spacing of soil reinforcements (including traffic impact requirements)
7. Facing design
   - Connections between facing units and components
   - Facing unit strength requirements
   - Interface shear between facing units
   - Connections between facing and soil reinforcement/reinforced soil mass
   - How facing batter is taken into account for the range of facing batters available for the system
   - Facing compressibility/deformation, if a flexible facing is used
8. Seismic design considerations
9. Design assumptions/parameters for assessing mobilization of backfill weight internal to wall system (primarily applies to prefabricated modular walls as defined in the AASHTO LRFD Bridge Design Specifications)

List all wall/slope system design limitations, including:
- Seismic loading
- Environmental constraints
- Wall height
- External loading
- Horizontal and vertical deflection limits
- Tolerance to total and differential settlement
- Facing batter
- Other
Computer Support:

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

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

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

15-C-6 Part Five: Construction

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

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

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

3. Quality Control/Quality Assurance of Construction:

Describe the quality control and quality assurance measurements required during construction to assure consistency in meeting performance requirements.
15-C-7 Part Six: Performance

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

1. Provide a copy of any system warranties.

2. Identify the designated Responsible Party for:
   - System performance
   - Material performance
   - Project-specific design (in-house, consultant)

3. List insurance coverage types (e.g., professional liability, product liability, performance) limits, basis (i.e., per occurrence, claims made) provided by each responsible party

4. Provide a well documented history of performance (with photos, where available), including:
   - Oldest
   - Highest
   - Projects experiencing maximum measure settlement (total and differential)
   - Measurements of lateral movement/tilt
   - Demonstrated aesthetics
   - Project photos
   - Maintenance history

5. Provide the following types of field test results, if available:
   - Case histories of instrumented structures
   - Construction testing
   - Pullout testing

6. Regarding construction/in-service structure problems, provide case histories of structures where problems have been encountered, including an explanation of the problems and methods of repair.

7. Provide a list of state DOT's that have used this wall system, including contact persons, addresses and telephone numbers.
## Appendix 15-D  Preapproved Proprietary Wall Systems

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

<table>
<thead>
<tr>
<th>Wall Supplier</th>
<th>System Name and Appendix Location</th>
<th>System Description and Appendix Location</th>
<th>ASD/ LFD or LRFD?</th>
<th>Height, or Other Limitations</th>
<th>Year Initially Approved</th>
<th>Last Approved Update</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hilfiker Retaining Walls</td>
<td>Welded Wire Retaining Wall Appendix 15-H</td>
<td>Welded wire facing that is continuous with welded wire soil reinforcement</td>
<td>ASD/LFD</td>
<td>33 feet</td>
<td>Approved 11/9/04 (submitted 9/15/03)</td>
<td></td>
</tr>
<tr>
<td>Hilfiker Retaining Walls</td>
<td>Eureka Reinforced Soil Wall Appendix 15-I</td>
<td>Precast concrete 5’×5’ facing panels and welded wire mat soil reinforcement</td>
<td>ASD/LFD</td>
<td>33 feet</td>
<td>Approved 11/9/04 (submitted 10/5/04)</td>
<td></td>
</tr>
<tr>
<td>The Reinforced Earth Co.</td>
<td>Reinforced Earth Wall Appendix 15-J</td>
<td>Precast concrete 5’×5’ facing panels and steel strip soil reinforcement</td>
<td>LRFD</td>
<td>33 feet</td>
<td>Approved 11/9/04 (submitted 3/29/04)</td>
<td></td>
</tr>
<tr>
<td>Tensar Earth Technologies, Inc.</td>
<td>ARES Wall Appendix 15-K</td>
<td>Precast concrete 5’×5’ facing panels and Tensar geogrid soil reinforcement</td>
<td>ASD/LFD</td>
<td>33 feet</td>
<td>Approved 11/9/04 (submitted 8/6/04)</td>
<td></td>
</tr>
<tr>
<td>Tensar Earth Technologies, Inc.</td>
<td>MESA Wall Appendix 15-L</td>
<td>Modular dry cast concrete block facing with Tensar geogrid soil reinforcement</td>
<td>ASD/LFD</td>
<td>33 feet</td>
<td>Approved 11/9/04 (submitted 4/19/04 and 9/22/04)</td>
<td></td>
</tr>
<tr>
<td>Tensar Earth Technologies, Inc.</td>
<td>Welded Wire Form Wall Appendix 15-M</td>
<td>Tensar geogrid wrapped face wall with welded wire facing form</td>
<td>ASD/LFD</td>
<td>33 feet*</td>
<td>Approved 3/3/06 (submitted 11/26/05)</td>
<td></td>
</tr>
<tr>
<td>SSL, LLC</td>
<td>MSEPlus Wall Appendix 15-N</td>
<td>Precast concrete 5’×5’ facing panels and steel welded wire strip soil reinforcement</td>
<td>LRFD</td>
<td>33 feet</td>
<td>Approved 8/5/13 (submitted 5/28/13)</td>
<td></td>
</tr>
</tbody>
</table>

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

1 For those systems still identified as ASD/LFD, use of the current AASHTO LRFD Bridge Design Specifications is preferred.
# Table 15-D-1  Preapproved Proprietary Walls

<table>
<thead>
<tr>
<th>Wall Supplier</th>
<th>System Name and Appendix Location</th>
<th>System Description and Appendix Location</th>
<th>ASD/LFD or LRFD? ¹</th>
<th>Height, or Other Limitations</th>
<th>Year Initially Approved</th>
<th>Last Approved Update</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Wall Systems, Inc.</td>
<td>Landmark Appendix 15-O</td>
<td>Modular dry cast concrete block facing with Miragrid geogrid soil reinforcement</td>
<td>LRFD</td>
<td>33 feet</td>
<td>2012</td>
<td>Approved 4/2/12 (submitted 10/21/11)</td>
</tr>
<tr>
<td>Allan Block Corporation</td>
<td>Allan Block Wall (battered face) Appendix 15-P</td>
<td>Modular dry cast concrete block facing with Miragrid or Stratagrid geogrid soil reinforcement</td>
<td>LRFD</td>
<td>33 feet</td>
<td>2009</td>
<td>Approved 7/15/09 (submitted 1/15/08)</td>
</tr>
<tr>
<td>Lock and Load Retaining Walls LTD</td>
<td>Lock and Load Wall Appendix 15-R</td>
<td>Precast concrete panel facing attached to wrapped face geogrid wall</td>
<td>LRFD</td>
<td>33 feet</td>
<td>2013</td>
<td>Approved 7/10/13 (submitted 5/3/13)</td>
</tr>
<tr>
<td>Basalite Concrete Products, LLC</td>
<td>GEOWALL Structural Earth Retaining Wall Appendix 15-T</td>
<td>Modular dry cast concrete block facing with Miragrid or Stratagrid geogrid soil reinforcement</td>
<td>LRFD</td>
<td>33 feet</td>
<td>2018</td>
<td>Approved 1/2/18 (submitted 3/4/17)</td>
</tr>
</tbody>
</table>

¹ 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.

² For those systems still identified as ASD/LFD, use of the current AASHTO LRFD Bridge Design Specifications is preferred.
Appendix 15-E  MSE Wall Design Using the Stiffness Method

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

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

\[ T_{\text{max}} = S_v \left[ H \gamma_r D_{\text{max}} + \left( \frac{H_{\text{ref}}}{H} \right) S \gamma_f \right] K_{\text{avh}} \Phi_g \Phi_{fb} \Phi_{local} \Phi_c \]  

(15-E-1)

where,

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

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

Cohesive shear strength of the MSE wall backfill shall not be used for final design (other than as illustrated in Example 5 at the end of this appendix), and MSE wall backfill that has significant soil cohesion should be avoided, as soil cohesion can be lost over time after wall construction and can also significantly reduce the ability of the wall backfill to drain as water percolates into it. This potential post-construction loss of cohesion over time as well as increase in the amount of water stored in the backfill can cause post-construction reinforcement load and deformation increases. The Stiffness Method can be used to estimate the reinforcement load and deformation increases that could occur post-construction as soil cohesion is lost. See Example 5 at the end of this appendix for an illustration of the effect of lost cohesion after wall construction on reinforcement strains.
Table 15-E-1  Summary of equations, parameters, and coefficients for the Stiffness Method
(Allen and Bathurst 2018)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Name</th>
<th>Equation</th>
<th>Coefficient</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{\text{tmax}}$</td>
<td>$T_{\text{max}}$ distribution factor</td>
<td>$z_b = C_h \times (H)^{\gamma} \times \Phi_{fb}$ For $z &lt; z_b$: $D_{\text{tmax}} = D_{\text{tmax0}} + (z/z_b) \times (1 - D_{\text{tmax0}})$ For $z \geq z_b$: $D_{\text{tmax}} = 1.0$</td>
<td>$C_h$ (for $H$ in m) $C_h$ (for $H$ in ft) $\gamma$ $D_{\text{tmax0}}$</td>
<td>0.40 0.32 1.2 0.12</td>
</tr>
<tr>
<td>$\Phi_g$</td>
<td>Global stiffness factor</td>
<td>$\Phi_g = \alpha \left( \frac{S_{\text{global}}}{P_a} \right)^{\beta}$</td>
<td>$\alpha$ $\beta$</td>
<td>0.16 0.26</td>
</tr>
<tr>
<td>$S_{\text{global}}$</td>
<td>Global reinforcement stiffness</td>
<td>$S_{\text{global}} = \frac{J_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} R_c J_i}{H}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Phi_{\text{local}}$</td>
<td>Local stiffness factor</td>
<td>$\Phi_{\text{local}} = \left( \frac{S_{\text{local}}}{S_{\text{local ave}}} \right)^{\alpha}$</td>
<td>“a” for steel “a” for geosynthetic and extensible steel grids $^b$</td>
<td>0 0.5</td>
</tr>
<tr>
<td>$S_{\text{local}}$</td>
<td>Local reinforcement stiffness</td>
<td>$S_{\text{local}} = R_c J_i / S_v$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$S_{\text{local ave}}$</td>
<td>Average local reinforcement stiffness</td>
<td>$S_{\text{local ave}} = \frac{\sum_{i=1}^{n} (R_c J_i / S_v)}{n}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Phi_{fb}$</td>
<td>Facing batter factor</td>
<td>$\Phi_{fb} = \left( \frac{K_{\text{abh}}}{K_{\text{ahb}}} \right)^{d}$</td>
<td>$d$</td>
<td>0.4</td>
</tr>
<tr>
<td>Coefficient of active earth pressure</td>
<td></td>
<td>$K_{\text{abh}} = \frac{\cos^2(\phi_f + \omega)}{\cos^3 \omega \left( 1 + \frac{\sin \phi_f}{\cos \omega} \right)^2}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Phi_{fs}$</td>
<td>Facing stiffness factor</td>
<td>$\Phi_{fs} = \eta \left( \frac{S_{\text{global}}}{P_a} \right)^{\kappa} F_f$</td>
<td>$\eta$ $\kappa$</td>
<td>0.57 0.15</td>
</tr>
<tr>
<td>$F_f$</td>
<td>Facing stiffness parameter</td>
<td>$F_f = \frac{1.5 H^3 P_a}{E b^3 (h_{\text{eff}}/H)}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Phi_c$</td>
<td>Soil cohesion factor</td>
<td>$\Phi_c = \exp(\lambda(c/(\gamma r H)))$</td>
<td>$\lambda$</td>
<td>-16</td>
</tr>
</tbody>
</table>

Notes:
$^a$ see Allen and Bathurst (2015)
$^b$ e.g., crimped longitudinal steel wire
Other Notation in Table 15-E-1:

- $T_{max}$ = the maximum load in the reinforcement (force/unit running length of wall – e.g. (lbs/ft))
- $n$ = number of reinforcement layers
- $H$ = height of wall (ft)
- $H_{ref}$ = reference wall height = 20 ft
- $S_v$ = tributary vertical spacing of the reinforcement layer (ft)
- $b$ = thickness of the facing column (ft)
- $E$ = elastic modulus of the “equivalent elastic beam” representing the wall face (ksf)
- $p_a$ = atmospheric pressure (101 kPa or 2.11 ksf)
- $h_{eff}$ = equivalent height of an un-jointed facing column that is approximately 100% efficient in transmitting moment through the height of the facing column (ft)
- $K_{ahb}$ = horizontal component of active earth pressure coefficient accounting for wall face batter
- $K_{avh}$ = horizontal component of active earth pressure coefficient assuming the wall is vertical ($\omega = 0$)
- $J_i$ = secant tensile creep stiffness of geosynthetic reinforcement layer i at 2% strain and 1000 hours on a per unit of reinforcement width basis from laboratory testing (kips/ft)
- $J_{ave}$ = average secant tensile creep stiffness corrected for the coverage ratio, i.e., $R_c J_i$, of all “n” geosynthetic reinforcement layers (kips/ft)
- $R_c$ = reinforcement coverage ratio
- $\sigma_v$ = vertical pressure due to gravity forces from self-weight of the reinforced soil and soil above the reinforced wall backfill (ksf)
- $c$ = soil cohesion (ksf)
- $\gamma_r$ = unit weight of the reinforced soil (kcf)
- $\gamma_f$ = unit weight of the soil surcharge (kcf)
- $q$ = $S_{fr} = \frac{S}{H} = \frac{c}{S}$ = average vertical pressure due to soil surcharge on the top of the reinforced soil mass up to a maximum width of 70% of the wall height $H$ (ksf)
- $z$ = depth below wall top measured at the back of the facing (ft)
- $K_a$ = active earth pressure coefficient
- $S$ = average soil surcharge depth above the wall top using a soil surcharge unit weight $\gamma_f$ (ft)
- $\phi_r$ = friction angle of the reinforced soil backfill (degrees)
- $\omega$ = wall face batter in clockwise direction from the vertical (degrees). In AASHTO (2020) the face batter $\theta$ is taken clockwise from the horizontal, hence $\omega = \theta - 90^\circ$
<table>
<thead>
<tr>
<th>Cohesive strength component deduced from failure envelope</th>
<th>Plasticity Index PI</th>
<th>Used to calculate $K_{avh}$ and $K_{abh}$</th>
<th>Value of $c$ used to calculate $\phi_c$</th>
<th>Cohesion factor $\phi_c$</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c = 0$</td>
<td>NA</td>
<td>$\phi_{tx}$ or $\phi_{ds}$</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$c &gt; 0$</td>
<td>$\leq 6$</td>
<td>$\phi_{tx}$ or $\phi_{ds}$</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>(curved Mohr-Coulomb envelope due to particle interlocking)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$c &gt; 0$</td>
<td>$\leq 6$</td>
<td>$\phi_{tx}$ or $\phi_{ds}$</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>(apparent cohesion due to matric suction)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$c &gt; 0$</td>
<td>$&gt; 6$</td>
<td>$\phi_{tx}$ or $\phi_{ds}$</td>
<td>0</td>
<td>&gt; 0</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>(true cohesion)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: PI = Plasticity Index, $\phi_r$ = peak friction angle for reinforced soil backfill, $\phi_{tx}$ = peak friction angle from triaxial test, $\phi_{ds}$ = peak friction angle from direct shear test, $\phi_{sec}$ = peak secant friction angle (determined as shown in Allen and Bathurst (2015, 2018)).
15-E-2 Limit State Equations for Design

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

15-E-2.1 Soil Failure Limit State Design

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

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

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

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

In general, the soil failure limit state should be checked first, as this limit state often controls the amount of reinforcement required. The stiffness values coming out of that limit state analysis should then be used to determine $T_{\text{max}}$ for reinforcement and connection rupture, and pullout. For systems with very poor connection strength, it is possible that connection strength could control design instead. If that is the case, the soil failure limit state may need to be reassessed to make sure that the reinforcement creep stiffness is consistent with the ultimate tensile strength needed. See Allen and Bathurst (2019) for information on the correlation between tensile strength and creep stiffness, as well as AASHTO NTPEP (2019) for product line specific correlations between tensile strength and creep stiffness.
15-E-2.2 Reinforcement Strength Design

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

\[ RF = RF_{ID} \times RF_{CR} \times RF_D \]  \hspace{1cm} (15-E-3)

where,

- \( RF_{ID} \) = installation damage reduction factor,
- \( RF_{CR} \) = creep reduction factor, and
- \( RF_D \) = durability reduction factor.

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

The long-term (nominal) design strength is:

\[ T_{at} = T_{ult} \frac{RF_{ID}RF_{CR}RF_D}{RF} \]  \hspace{1cm} (15-E-4)

where,

- \( T_{ult} \) = ultimate tensile strength of the reinforcement

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

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

\[ \gamma_p \gamma_v T_{max} < \phi_r T_{at} R_c \]  \hspace{1cm} (15-E-5)

where,

- \( \phi_r \) = the resistance factor for reinforcement rupture.

All parameters are as defined previously.

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

\[ T_{at} \text{ (required)} = \left( \frac{\gamma_p \gamma_v}{\gamma_r R_c} \right) T_{max} \left( \frac{\gamma_p \gamma_v}{\phi_r R_c} \right) S_v \left[ H \gamma_v D_{tmax} + \left( \frac{H_{ref}}{H} \right) S_f \right] K_{avh} \phi_{f,b} \phi_g \phi_{f,sl} \phi_{local} \phi_c \]  \hspace{1cm} (15-E-6)

where,

- \( T_{at} \text{ (required)} \) = the required minimum (factored) long-term reinforcement strength to resist the factored loads.
The equivalent expression for the case of an additional live load LL is:

\[ T_{at\ (required)} = \left( \frac{\gamma_{p-EV}}{\phi_{cr}} \right) T_{max} = \left( \frac{\gamma_{p-EV}}{\phi_{cr}} \right) S_v \left( H \gamma_f D_{tmax} + \frac{H_{ref}}{H} S \gamma_f \right) + \left( \frac{\gamma_{LL}}{\gamma_{p-EV}} \right) K_{avh} \phi_{f_b} \phi_{f_g} \phi_{f_s} \phi_{local} \phi_c \]  

(15-E-7)

where,

- \( LL \) = live load (kPa),
- \( \gamma_{LL} \) = live load factor = 1.75.

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

\[ T_{at\ (required)} = \left( \frac{\gamma_{p-EV} T_{max} + \gamma_{p-ES} S_v (K_a \Delta \sigma_v + \Delta \sigma_H)}{\phi_{cr}} \right) S_v \left( H \gamma_f D_{tmax} + \frac{H_{ref}}{H} S \gamma_f \right) + \left( \frac{\gamma_{LL}}{\gamma_{p-EV}} \right) K_{avh} \phi_{f_b} \phi_{f_g} \phi_{f_s} \phi_{local} \phi_c + \left( \frac{\gamma_{p-ES} S_v (K_a \Delta \sigma_v + \Delta \sigma_H)}{\phi_{cr}} \right) \]  

(15-E-8)

where,

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

However, the soil failure limit state would also need to be checked for the combined loading. If superposition principles are used to determine that combined loading, the soil failure limit state will become excessively conservative for footing load that are typical for bridges. Therefore, if designing an MSE bridge abutment, due to the high footing load that is likely, it is best to use the Simplified Method instead to do that design.
15-E-2.3 Connection Strength Design

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

The reinforcement connection strength limit state equation is as follows:

$$\gamma_{p-EVc}T_o = \phi_{cr}T_{ac}R_c$$

(15-E-9)

where,

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

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

$$T_{\text{ultconn}} = c_{\text{conn}} + N \tan \phi_{\text{conn}}$$

$$= c_{\text{conn}} + (b \sigma_n) \tan \phi_{\text{conn}} = c_{\text{conn}} + (b[\gamma_{bk} \times z]) \tan \phi_{\text{conn}}$$

(15-E-10)

where,

- $c_{\text{conn}} = \text{the vertical axis intercept (e.g., units of kips/ft) on a plot of connection capacity versus normal load } N \text{ (e.g., units of kips/ft) or stress } \sigma_n \text{ (in ksf) acting at the connection due to the facing column}$,
- $b = \text{toe to heel dimension of the block}$,
- $\gamma_{bk} = \text{the unit weight of the infilled block}$,
- $z = \text{the depth of the connection below the crest of the wall (assuming the wall is vertical)}$, and
- $\phi_{\text{conn}} = \text{the slope of the failure envelope line segment}$. 
For many systems, the line segment for the normal load of interest may be horizontal (hence, $c_{\text{conn}} > 0$ and $\phi_{\text{conn}} = 0$) (Bathurst and Simac 1993).

$T_{ac}$ is determined as follows:

$$T_{ac} = \frac{T_{ult} \times CR_{cr}}{RF_D} \quad (15\text{-E-11})$$

where,

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

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

$$CR_{cr} = \frac{T_{ultconn}}{(RFCR \times T_{lot})} \quad (15\text{-E-12})$$

where,

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

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

$$T_{ac} (\text{required}) = \frac{T_{ultconn}}{(RF_D \times RFCR)} \geq \left( \frac{Y_{con}}{\phi_{cr} R_c} \right) \left( T_o + LL \left( \frac{Y_{LL}}{Y_{con}} \right) S_v K_{avh} \phi_g \phi_{fb} \phi_{fs} \phi_{local} \phi_c \right) \quad (15\text{-E-13})$$

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

$$T_{ult} (\text{required}) = \frac{T_{ultconn}}{CR_u} \geq \left( \frac{Y_{con}}{\phi_{cr} R_c} \right) \left( RF_D \times RFCR \right) \left( T_o + LL \left( \frac{Y_{LL}}{Y_{con}} \right) S_v K_{avh} \phi_g \phi_{fb} \phi_{fs} \phi_{local} \phi_c \right) \quad (15\text{-E-14})$$

All variables are defined previously. For other types of connections, minor modifications to these equations may be needed; see AASHTO (2020) for guidance on handling other facing connection systems.
15-E-2.4  Pullout Resistance Limit State Design

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

\[ \gamma_{perv} T_{max} = \phi_{po} P_c \]  \hspace{1cm} (15-E-15)

where,

- \( P_c \) = nominal calculated pullout resistance, and
- \( \phi_{po} \) = resistance factor applicable to pullout resistance.

Other variables are defined previously.

\( P_c \) is calculated as:

\[ P_c = C(F^* \alpha) \sigma_v L_e R_c \]  \hspace{1cm} (15-E-16)

where,

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

Details how to determine \( \alpha \) and \( F^* \), vertical stress \( \sigma_v \), and anchorage length \( L_e \) behind the active zone are provided in AASHTO (2020), Article 11.10.6.3.2.
15-E-2.5 Design Process for the Stiffness Method

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

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

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

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

$$T_{\text{md}} = \left(\frac{P_i}{n}\right)$$

where,

$P_i$ = internal inertia force due to the weight of backfill within the active zone, i.e., the shaded area in AASHTO (2020) Figure 11.10.7.2-1 (kips/ft)

$n$ = total number of reinforcement layers in the wall at a specific wall section (dim)

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

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

$$k_h = 0.74A_s \left(\frac{A_s}{d}\right)^{0.25}$$

where,

$k_h$ = horizontal seismic acceleration coefficient (dim)

$A_s$ = earthquake ground acceleration coefficient as specified in Equation 3.10.4.2-2 in AASHTO (2020)

$d$ = lateral wall displacement (in.)

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

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

\[
P_i = k_h \left( \gamma_r \times A_{\text{active}} + \gamma_{\text{facing}} \times T_f \times H \right)
\]  

(15-E-19)

where,

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

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

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

\[
T_{\text{total}} = \gamma_{\text{seis}} \left( T_{\text{max}} + T_{\text{md}} \right)
\]  

(15-E-20)

where,

- \( \gamma_{\text{seis}} \) = Extreme Event I load factor for reinforcement load due dead load plus seismically induced reinforcement load (dim)

Note that the reason \( \gamma_{\text{seis}} = 1.0 \) for both the static and dynamic portions of the load is that a significantly higher probability of failure is targeted due to the fact that the load has a small likelihood of occurring and also that some damage is acceptable during seismic loading. The Strength Limit State load factors are significantly greater than 1.0 because the probability of failure targeted is much lower, and significant damage due to the static loading is to be prevented by the design.

For seismic pullout design, \( T_{\text{total}} \) is used. However, for reinforcement rupture and connection rupture under seismic loading, the static and dynamic components of the reinforcement load must be handled separately. The reason for this is that the strength required to resist \( T_{\text{max}} \) must include the effects of creep because it is a sustained load, but the strength required to resist \( T_{\text{md}} \) should not include the effects of creep due to the transient nature of \( T_{\text{md}} \). See AASHTO (2020) for additional details.
15-E-3.1 Reinforcement Rupture (Extreme Event I - Seismic)

The ultimate tensile strength of the reinforcement is determined by summing together the portion needed to resist the static force (i.e., $T_{max}$) and the portion needed to resist the dynamic portion (i.e., $T_{md}$). Therefore,

$$T_{ult} = S_{rs} + S_{rt} \quad (15-E-21)$$

where,

- $S_{rs}$ = ultimate reinforcement tensile resistance required to resist static load component (kips/ft)
- $S_{rt}$ = ultimate reinforcement tensile resistance required to resist dynamic load component (kips/ft)

For the static component,

$$S_{rs} = \frac{\gamma_{seis} T_{max} RF}{\phi R_c} \quad (15-E-22)$$

Again, $T_{max}$ is determined using the Stiffness Method.

For the dynamic component,

$$S_{rt} = \frac{\gamma_{seis} T_{md} RF_{ID} RF_{D}}{\phi R_c} \quad (15-E-23)$$

where,

- $\gamma_{seis}$ = the Extreme Event I load factor for reinforcement load due to dead load plus seismically induced reinforcement load (dim.)
- $\phi = $ resistance factor for combined static/earthquake loading from Article 11.5.8 (dim.)
- $R_c = $ reinforcement coverage ratio specified in Article 11.10.6.4.1 (dim.)
- $RF = $ combined strength reduction factor to account for potential long-term degradation due to installation damage, creep, and chemical aging specified in Article 11.10.6.4.3b (dim.)
- $RF_{ID} = $ strength reduction factor to account for installation damage to reinforcement specified in Article 11.10.6.4.3b (dim.)
- $RF_{D} = $ strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation specified in Article 11.10.6.4.3b (dim.)
15-E-3.2 Connection Rupture (Extreme Event I - Seismic)

Similarly, the approach used for reinforcement rupture during seismic shaking is also used for connection rupture.

\[ T_{\text{ult}} = S_{\text{rsc}} + S_{\text{rtc}} \]  \hspace{1cm} (15-E-24)

where,

\[ S_{\text{rsc}} = \text{ultimate reinforcement tensile resistance required to resist static load component at connection (kips/ft)} \]

\[ S_{\text{rtc}} = \text{ultimate reinforcement tensile resistance required to resist dynamic load component at connection (kips/ft)} \]

\[ S_{\text{rsc}} = \frac{\gamma_{\text{seis}} T_0 R_{\text{FD}}}{F_r \phi C_{R\text{r}} R_c} \]  \hspace{1cm} (15-E-25)

\[ S_{\text{rtc}} = \frac{\gamma_{\text{seis}} T_{\text{md}} R_{\text{FD}}}{F_r \phi C_{R\text{u}} R_c} \]  \hspace{1cm} (15-E-26)

where,

\[ \gamma_{\text{seis}} = \text{the Extreme Event I load factor for reinforcement load due to dead load plus seismically induced reinforcement load (dim)} \]

\[ T_0 = \text{applied load to reinforcement at facing connection (kip/ft)} \]

\[ R_{\text{FD}} = \text{reduction factor to prevent rupture of reinforcement due to chemical and biological degradation specified in Article 11.10.6.4.4b (dim.)} \]

\[ \phi = \text{resistance factor applicable to seismic loading, typically 1.0 (dim.)} \]

\[ C_{R\text{r}} = \text{long-term connection strength reduction factor to account for reduced ultimate strength resulting from connection, equal to } C_{R\text{u}} / RF_{CR} \text{ or } T_{\text{crc}} / T_{\text{lot}}, \text{in which } T_{\text{crc}} \text{ is the creep limited connection strength at the desired design life if the creep limited connection strength is determined directly from the connection creep test data (dim.)} \]

\[ R_c = \text{reinforcement coverage ratio from Article 11.10.6.4.1 (dim.)} \]

\[ T_{\text{md}} = \text{factored incremental dynamic inertia force (kip/ft)} \]

\[ C_{R\text{u}} = \text{short-term reduction factor to account for reduced ultimate strength resulting from connection as specified in AASHTO (2020) Article C11.10.6.4.4b, equal to } T_{\text{ultconn}} / T_{\text{lot}} \text{ (dim.)} \]

\[ F_r = \text{reduction factor to account for reduced friction during shaking between facing blocks and geosynthetic reinforcement (equal to 0.8 if the connection relies primarily on friction, or 1.0 if the connection is structural, i.e., does not rely on friction)} \]

15-E-3.3 Pullout (Extreme Event I - Seismic)

\[ L_e = \frac{\gamma_{\text{seis}} (T_{\text{max}} + T_{\text{md}})}{\phi C (0.8a F^*) \sigma_p R_c} \]  \hspace{1cm} (15-E-27)

where,

\[ \gamma_{\text{seis}} = \text{the Extreme Event I load factor for reinforcement load due to dead load plus seismically induced reinforcement load (dim)} \]
15-E-4 Summary of MSE Wall Internal Stability Design Steps for Geosynthetic Walls Using the Stiffness Method

1. Establish wall geometry (height, surcharge, minimum width of reinforced soil zone to satisfy external stability, face batter), and select facing type

2. Establish soil backfill properties (\(\phi\), c, \(\gamma\))

3. Develop preliminary wall reinforcement layout (Sv and Sh)

4. Calculate active earth pressure coefficient for reinforced zone

5. Calculate reinforcement load for each layer, \(T_{max}\), using the creep stiffness required in each layer to meet Service Limit state requirements (i.e., the soil failure limit state) as a starting point

6. Calculate long-term tensile strength needed for each layer, \(T_{al}\), for Strength Limit state, starting with creep stiffness values determined from Step 5
   a. Reinforcement rupture: \(\phi T_{al} \geq \gamma_p - E_{Tmax}\)
   b. Connection rupture: \(\phi T_{ac} \geq \gamma_p - E_{T0}\)
   c. In both cases, select geosynthetic reinforcement products with consideration to long-term strength reduction factors applicable to each product (i.e., RFID, RFCR, and RFD) and with consideration to the long-term connection strength available considering the block-geosynthetic combinations available

7. Calculate reinforcement length needed, \(L_a + L_e\), for pullout, Strength Limit State

8. Calculate long-term strength needed for Extreme Event I Limit state (seismic design)
   a. \(T_{totalf} = \gamma_{seis}(T_{max} + T_{md})\)
   b. Reinforcement rupture
   c. Connection rupture
   d. In both cases, select geosynthetic reinforcement products with consideration to long-term strength reduction factors applicable to each product (i.e., RFID and RFD, as RFCR not important for seismic loading) and with consideration to the connection strength available considering the block-geosynthetic combinations available
   e. Pullout

9. If the connection strength is low and/or if the seismic acceleration is high and controls the reinforcement design with regard to strength and stiffness required, recalculate \(T_{max}\) using the increased stiffness required and recheck all limit states

10. Check compound stability (\(T_{al}\) and reinforcement length needed, both Strength and Extreme Event I limits)
A series of 20 ft tall wall design examples are provided in the sections that follow to illustrate these design steps for various cases. These design examples include:

1. Flexible face wall, coverage ratio, $R_c$, of 1.0,

2. Modular block face wall, coverage ratio, $R_c$, of 1.0, mechanical facing-reinforcement connection; same as Example 1 but with stiff, rather than flexible, facing,

3. Modular block face wall, coverage ratio, $R_c$, of 0.9, mechanical facing-reinforcement connection; same as Example 2 except coverage ratio is less than 1.0 to illustrate how the coverage ratio is addressed in design,

4. Modular block face wall, coverage ratio, $R_c$, of 0.9, frictional facing-reinforcement connection (proprietary wall system); same as Example 3 except that the facing reinforcement is frictional rather than mechanical, and

5. Flexible face wall, coverage ratio, $R_c$, of 1.0; same as Example 1, partial example to illustrate the effect of backfill cohesion, and how cohesion should, and should not be, handled in wall design.

15-E-5 Stiffness Method Design Example 1: Flexible Faced Geosynthetic Wall

15-E-5.1 General

This first example is a simple design case. Subsequent examples will add features that increase in complexity to illustrate how various scenarios are handled when using the Stiffness Method.

Figure 15-E-2 shows a cross-section of the wall for this design example, and material properties are provided in Table 15-E-3. Assume for this example that polyester (PET) geogrid will be used for the soil reinforcement. Assume a flexible facing will be used (e.g., welded wire baskets, or just a geosynthetic wrap facing). Assume that the connection between the geogrid and the facing is not an issue (i.e., the connection strength is 100% efficient) – while this may not be the case for welded wire baskets as shown, it will be the case for a wrapped face wall. So this assumption is made in this example to focus on the simplest case for illustration purposes. The wall backfill is assumed to be a well-graded gravelly sand, with no cohesion. The scope of this design example is limited to the service (soil failure limit only), strength, and Extreme Event I (seismic) limit states. Furthermore, for simplicity, live loads are not included in the calculations to follow. It is also assumed that the foundation soil has sufficient shear strength to prevent global instability, bearing capacity failure and excessive settlement. For seismic design, assume that the ground acceleration, $A_s$, is 0.50g.
Note that a 2 ft vertical spacing of the reinforcement is used for this example. Normally, for a wrapped face geosynthetic wall, a vertical spacing of 2 ft is too large to keep face bulging, overall lateral deformation, and possibly vertical deformation, under control and is likely marginally too large even for a welded wire flexible faced wall. This spacing is being used in this example to facilitate comparisons with the stiff (dry cast concrete blocks) faced wall examples provided subsequent to this example. Also, a reinforcement coverage ratio, $R_c$, of 1.0 is used for this example. Also note that if a wrapped face geosynthetic wall is part of a two-stage wall system in which a concrete wall facing is added after the post-construction wall movement has ceased (e.g., the WSDOT Standard Plan geosynthetic wall), it is still designed as a flexible faced wall, since the more rigid concrete facia is added after the wrapped face wall is constructed.

### Table 15-E-3  Design properties for wall

<table>
<thead>
<tr>
<th>Property</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moist soil unit weight (pcf)</td>
<td>130</td>
</tr>
<tr>
<td>Triaxial drained peak friction angle $\phi_{tx}$ (°)</td>
<td>34</td>
</tr>
<tr>
<td>Min. available, but not wall system specific, 1,000 hr, 2% secant $J_i \times R_c$ (kips/ft)</td>
<td>8.6 × 1.0 = 8.6</td>
</tr>
<tr>
<td>$RF_{ID} \times RF_{CR} \times RF_D = RF$</td>
<td>1.12 × 1.5 × 1.3 = 2.18</td>
</tr>
<tr>
<td>Coverage ratio, $R_c$</td>
<td>1.0</td>
</tr>
<tr>
<td>Facing welded wire basket height (ft)</td>
<td>1.0 (i.e., 2 baskets between reinforcement layers)</td>
</tr>
<tr>
<td>Facing welded wire basket width, $W_u$ (face to tail) (ft)</td>
<td>1.0</td>
</tr>
<tr>
<td>Connection strength as fraction of $T_{ult}$, $CR_u$</td>
<td>1.0</td>
</tr>
</tbody>
</table>

In **Table 15-E-3**, and all subsequent uses, $J_i$ is defined as the geosynthetic product secant creep stiffness at 1,000 hrs and 2% strain, per unit of reinforcement width. When $J_i$ is multiplied by $R_c$, the resulting stiffness is per unit of wall width rather than per unit of reinforcement width.
Since, for this example, the wall is designed assuming that the wall face is flexible, $\Phi_{fs} = 1.0$. It is assumed that the design can be completely generic (e.g., the WSDOT Standard Plan Geosynthetic wall). For this flexible wall face example, connection strength is assumed to not be a consideration, so either reinforcement stiffness or reinforcement rupture will likely control the design.

The wall geometry is based on Figure 15-E-2. Example calculations are demonstrated for one of the middle layers (i.e., Layer 6 in Figure 15-E-2, and for the soil failure and pullout limit states, Layer 10 is also used to illustrate calculations).

\[
S = \text{equivalent uniform height of surcharge} = 0 \text{ ft}
\]

\[
K_{awh} = K_a = \frac{1 - \sin \varphi_r}{1 + \sin \varphi_r} = \frac{1 - \sin 34^\circ}{1 + \sin 34^\circ} = 0.283
\]

For walls with a facing batter $\omega > 0$, the formula below is used to compute $K_{abh}$ which appears in the facing batter factor equation ($\Phi_{fb}$). Since the wall in this example is vertical ($\omega = 0$), $K_{abh} = K_{awh}$ as shown here:

\[
K_{abh} = \frac{\cos^2(\varphi_r + \omega)}{\cos^2\omega \left(1 + \sin \frac{\varphi_r}{\cos \omega}\right)^2} = \frac{\cos^2(34^\circ + 0)}{\cos^2 0 \left(1 + \sin \frac{34^\circ}{\cos 34^\circ}\right)^2} = 0.283
\]

The reinforcement stiffness values used in the calculations to follow need to be adjusted to account for the reinforcement coverage ratio, $R_c$. However, for this simple example, $R_c$ is assumed to be 1.0, which is the typical case for flexible faced walls anyway.

Since the soil failure limit state usually controls the amount of reinforcement required, this limit state is evaluated first, and $T_{max}$ calculated, for the wall design.
15-E-5.2 Calculations for Soil Failure Limit State Design (Service I)

The goal of this limit state is to ensure that the factored reinforcement strain in each layer is less than the target maximum strain in the wall required to prevent a contiguous shear surface through the backfill soil from developing (i.e., soil failure limit state). The AASHTO LRFD Bridge Design Specifications (AASHTO 2020) require that the factored reinforcement peak strain for each layer be 2.5% or less for a flexible faced wall. Some trial-and-error is typically required to establish what reinforcement stiffness values are required. As a first trial, use the minimum 1,000-hour 2% strain secant stiffness available for geosynthetic reinforcement products, $J_i = 8.6 \text{ kips/ft}$ for all layers (see Table 15-E-3).

Note that the creep stiffness and strength for specific products are available in the WSDOT Qualified Products List (QPL) Appendix E. Alternatively, this data can be obtained from NTPEP (2019) at the AASHTO NTPEP website (https://data.ntpep.org/REGEO/Products), and once there, click on “Construction” and then “Geosynthetic Reinforcement”. In addition, Allen and Bathurst (2019) summarize all the NTPEP low strain 1,000 hour creep stiffness data available at that time, additional creep stiffness data found in the literature, and generic relationships between $T_{ult}$ and the 1,000 hour 2% secant creep stiffness for various geosynthetic reinforcement product types.

The contributing factors, coefficients and parameters that comprise the $T_{max}$ equation can be found in Allen and Bathurst (2015) and Table 15-E-1. To calculate $T_{max}$, the reinforcement stiffness must be per unit of wall width rather than per unit of reinforcement product width; hence, $RcJ_i$ must be used where the reinforcement stiffness value is required. Therefore, the parameters used to determine $T_{max}$ are calculated as follows:

$$S_{global} = \frac{J_{ave}}{(H/n)} = \frac{\sum_{i=1}^{n} RcJ_i}{H} = (10 \times 1.0 \times 8.6 \text{ kips/ft})/20 \text{ ft} = 4.30 \text{ ksf}$$
(applies to whole wall section)

$$\phi_g = \alpha \left( \frac{S_{global}}{P_a} \right)^\beta = 0.16 \times (4.30 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.193 \text{ (applies to whole wall section)}$$

$$\phi_{fb} = \left( \frac{K_{aabh}}{K_{avh}} \right)^d = (0.283/0.283)^{0.4} = 1.0 \text{ (applies to whole wall section)}$$

$$S_{local} = \left( \frac{RcJ_i}{S_v} \right) = (1.0 \times 8.6 \text{ kips/ft})/(2.0 \text{ ft}) = 4.30 \text{ ksf for Layer 6}$$

$$\frac{\sum (RcJ_i/S_v)}{n} = \frac{3.69 + 8 \times 4.30 + 5.15}{10} = 4.32 \text{ ksf}$$

where, $n = 10$ is the number of reinforcement layers. Therefore, $\phi_{local}$ is calculated as follows:

$$\phi_{local} = \left( \frac{S_{local}}{S_{localave}} \right)^{0.5} = \left( \frac{4.30 \text{ ksf}}{4.32 \text{ ksf}} \right)^{0.5} = 1.00 \text{ (layer 6)}$$
The facing stiffness factor $\Phi_{fs}$ is equal to 1.0, since the facing is considered flexible.

Since $c = 0$, the cohesion factor, $\Phi_c = 1.0$.

$D_{t_{max}}$ is determined for Layer 6 as follows:

$$z_b = C_h \times (H)^y \times \Phi_{fb} = (0.32 \times (20 \text{ ft})^{1/2}) \times 1.0 = 11.65 \text{ ft}$$

For $z \leq z_b$: $D_{t_{max}} = D_{t_{max}0} + (z/z_b) \times (1 - D_{t_{max}0}) = 0.12 + (9.33 \text{ ft}/11.65 \text{ ft}) \times (1 - 0.12) = 0.825$

For bottom layers where $z > z_b$: $D_{t_{max}} = 1.0$

$T_{max}$ for layer 6 is calculated as follows:

$$T_{max} = 2.0 \text{ ft} \left[ 20 \text{ ft} \times 0.130 \ kcf \times 0.825 + \left( \frac{20 \text{ ft}}{20 \text{ ft}} \right) 0 \text{ ft} \times 0.130 \ kcf + 0 \right] \times 0.193 \times 1.0 \times 1.0 \times 1.0 \times 0.283 \times 1.0$$

$T_{max} = 0.233 \text{ kips/ft of wall width}$

Using Equation 15-14 with load factor $\gamma_{sf} = 1.2$, and resistance factor $\phi_{sf} = 1.0$ (Table 15-5), the factored reinforcement strain corresponding to layer 10 is computed as:

$$\varepsilon_{rein} = \frac{\gamma_p-E_{Esf}T_{max}}{\phi_{sf} R_{ci} l_i} = \frac{1.2 \times 0.067 \text{ kips/ft}}{1.0 \times 1.0 \times 8.6 \text{ kips/ft}} \times 100\% = 0.94\% \leq 2.5\% \ \text{OK}$$

For layer 6:

$$\varepsilon_{rein} = \frac{\gamma_p-E_{Esf}T_{max}}{\phi_{sf} R_{ci} l_i} = \frac{1.2 \times 0.233 \text{ kips/ft}}{1.0 \times 1.0 \times 8.6 \text{ kips/ft}} \times 100\% = 3.25\% \leq 2.5\% \ \text{No.}$$

$T_{max}$, and the calculated parameters needed to calculate $T_{max}$, are summarized in Table 15-E-4 for the rest of the layers. As can be seen in the table and the calculations above, the calculated factored strains are greater than 2.5% in the lower half of the wall, which exceeds the soil failure limit state strain criterion provided in AASHTO (2020) of 2.5% for flexible faced walls.
Summary of Example 1 wall design calculations using Stiffness Method considering only the Service Limit State, first trial using only the minimum stiffness geogrid product available.

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>z (ft)</th>
<th>S (ft)</th>
<th>$^8 \text{R}_J$ (kips/ft)</th>
<th>$S_{\text{global}}$ (ksf)</th>
<th>$S_{\text{local}}$ (ksf)</th>
<th>$D_{\text{max}}$</th>
<th>$F_T$</th>
<th>$\Phi_g$</th>
<th>$\Phi_{\text{local}}$</th>
<th>$\Phi_{fb}$</th>
<th>$\Phi_{fs}$</th>
<th>$^7 T_{\text{max}}$ (kips/ft) (Equation 15-E-1)</th>
<th>Factored $\varepsilon_{\text{com}}$ (%)</th>
<th>$^9 T_{at}$ Corresponding to $J_i$ (kips/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 (top)</td>
<td>1.33</td>
<td>2.33</td>
<td>8.6</td>
<td>4.30</td>
<td>3.69</td>
<td>0.220</td>
<td>N/A</td>
<td>0.193</td>
<td>0.92</td>
<td>1.0</td>
<td>1.0</td>
<td>0.067</td>
<td>0.94</td>
<td>0.67</td>
</tr>
<tr>
<td>9</td>
<td>3.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>3.72</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.105</td>
<td>0.147</td>
<td>1.46</td>
<td>0.67</td>
</tr>
<tr>
<td>8</td>
<td>5.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>0.523</td>
<td>N/A</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>1.0</td>
<td>0.190</td>
<td>0.233</td>
<td>2.06</td>
<td>0.67</td>
</tr>
<tr>
<td>7</td>
<td>7.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>0.674</td>
<td>N/A</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>1.0</td>
<td>0.275</td>
<td>0.233</td>
<td>3.25</td>
<td>0.67</td>
</tr>
<tr>
<td>6</td>
<td>9.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>0.825</td>
<td>N/A</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>1.0</td>
<td>0.282</td>
<td>0.275</td>
<td>3.84</td>
<td>0.67</td>
</tr>
<tr>
<td>5</td>
<td>11.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>0.976</td>
<td>N/A</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>1.0</td>
<td>0.282</td>
<td>0.282</td>
<td>3.94</td>
<td>0.67</td>
</tr>
<tr>
<td>4</td>
<td>13.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>1.00</td>
<td>N/A</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>1.0</td>
<td>0.282</td>
<td>0.282</td>
<td>3.94</td>
<td>0.67</td>
</tr>
<tr>
<td>3</td>
<td>15.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>1.00</td>
<td>N/A</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>1.0</td>
<td>0.282</td>
<td>0.282</td>
<td>3.94</td>
<td>0.67</td>
</tr>
<tr>
<td>2</td>
<td>17.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>1.00</td>
<td>N/A</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>1.0</td>
<td>0.282</td>
<td>0.282</td>
<td>3.94</td>
<td>0.67</td>
</tr>
<tr>
<td>1</td>
<td>19.33</td>
<td>1.67</td>
<td>8.6</td>
<td>4.30</td>
<td>5.15</td>
<td>1.00</td>
<td>N/A</td>
<td>0.193</td>
<td>1.09</td>
<td>1.0</td>
<td>0.258</td>
<td>0.282</td>
<td>3.60</td>
<td>0.67</td>
</tr>
<tr>
<td>Base of Wall</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\sum T_{\text{max}} = 2.12$</td>
<td></td>
<td>$\sum T_{at} = 6.70$</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Minimum stiffness needed to meet only the soil failure limit, considering all available geosynthetic reinforcement products.

*All values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (2020).

*For comparison to geogrid product MARV tensile strength that is not reduced by $R_c$ (i.e., load per unit of reinforcement width basis).
To keep the peak reinforcement strains to less than 2.5% so that the soil failure limit state is met, the design stiffness values need to be increased. Through trial-and-error, the minimum stiffness values needed to keep the peak strains below 2.5% are as shown in Table 15-E-5.

Table 15-E-5  
Creep stiffness values (i.e., at 2% strain and 1,000 h) for geogrids used in wall

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Geogrid Designation</th>
<th>$J_i$ (per Unit Width of Reinforcement, in kips/ft)</th>
<th>$^*R_c \times J_i$ (per Unit Width of Wall, in kips/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>a</td>
<td>8.6</td>
<td>8.6</td>
</tr>
<tr>
<td>9</td>
<td>a</td>
<td>8.6</td>
<td>8.6</td>
</tr>
<tr>
<td>8</td>
<td>a</td>
<td>8.6</td>
<td>8.6</td>
</tr>
<tr>
<td>7</td>
<td>a</td>
<td>8.6</td>
<td>8.6</td>
</tr>
<tr>
<td>6</td>
<td>b</td>
<td>17.0</td>
<td>17.0</td>
</tr>
<tr>
<td>5</td>
<td>b</td>
<td>17.0</td>
<td>17.0</td>
</tr>
<tr>
<td>4</td>
<td>b</td>
<td>17.0</td>
<td>17.0</td>
</tr>
<tr>
<td>3</td>
<td>b</td>
<td>17.0</td>
<td>17.0</td>
</tr>
<tr>
<td>2</td>
<td>b</td>
<td>17.0</td>
<td>17.0</td>
</tr>
<tr>
<td>1</td>
<td>b</td>
<td>17.0</td>
<td>17.0</td>
</tr>
</tbody>
</table>

*This is the stiffness value which has been corrected for $R_c$ to calculate $T_{max}$. Since $R_c = 1.0$, this stiffness is the same as the stiffness per unit of reinforcement width.

The input parameters to calculate $T_{max}$ are recalculated as follows using the revised stiffness values:

$$S_{global} = \frac{J_{ave}}{(H/n)} = \frac{\sum_{i=1}^{n} R_c J_i}{H} = (4 \times 1.0 \times 8.6 \text{ kips/ft} + 6 \times 1.0 \times 17.0 \text{ kips/ft})/20 \text{ ft} = 6.82 \text{ ksf}$$

$$\phi_g = a \left(\frac{S_{global}}{p_a}\right)^B = 0.16 \times (6.82 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.217$$

$$\phi_{fb} = \left(\frac{K_{abh}}{K_{avh}}\right)^d = (0.283/0.283)^{0.4} = 1.0$$

$$S_{local} = \left(\frac{R_c J}{S_v}\right)_i = (1.0 \times 17.0 \text{ kips/ft})/(2.0 \text{ ft}) = 8.50 \text{ ksf for Layer 6}$$

$$S_{localave} = \frac{\sum \left(R_c J/S_v\right)_i}{n} = \frac{3.65 + 3 \times 4.25 + 5 \times 8.50 + 10.2}{10} = 6.91 \text{ ksf}$$

where, $n = 10$ is the number of reinforcement layers. Therefore, $\phi_{local}$ is calculated as follows:

$$\phi_{local} = \left(\frac{S_{local}}{S_{localave}}\right)^{0.5} = \left(\frac{8.5 \text{ ksf}}{6.91 \text{ ksf}}\right)^{0.5} = 1.11 \text{ (layer 6)}$$
For a flexible wall face, the facing stiffness factor is assumed to be 1.0.

Since \( c = 0 \), the cohesion factor, \( \Phi_c = 1.0 \).

\( D_{t_{\text{max}}} \) does not change relative to the previous calculation (i.e., \( D_{t_{\text{max}}} \) for Layer 6 is 0.825).

\( T_{\text{max}} \) for Layer 6 is re-calculated as follows:

\[
T_{\text{max}} = S_v \left[ H H_c D_{t_{\text{max}}} + \left( \frac{H_{\text{ref}}}{H} \right) S Y_f + LL \right] K_{\text{avh}} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c
\]

\[
T_{\text{max}} = 2.0 \text{ ft} \left[ 20 \text{ ft} \times 0.130 \text{ kcf} \times 0.825 + \left( \frac{20 \text{ ft}}{20 \text{ ft}} \right) 0 \text{ ft} \times 0.130 \text{ kcf} + 0 \right] 0.283
\times 1.0 \times 0.217 \times 1.0 \times 1.11 \times 1.0
\]

\( T_{\text{max}} = 0.292 \text{ kips/ft} \)

\( T_{\text{max}} \), and the calculated parameters needed to calculate \( T_{\text{max}} \), are summarized in Table 15-E-6 for the rest of the layers.

### 15-E-5.3 Calculations for Soil Failure Limit State Design (Service I)

Using Equation 15-14, with load factor \( \gamma_{sf} = 1.2 \) and resistance factor \( \phi_{sf} = 1.0 \) (Table 15-5), the factored reinforcement strain corresponding to Layer 10 is computed as:

\[
\varepsilon_{\text{rein}} = \frac{\gamma_{p-EVsf} T_{\text{max}}}{\phi_{sf} R_{cj}} = \frac{1.2 \times 0.060 \text{kips/ft}}{1.0 \times 1.0 \times 8.6 \text{kips/ft}} \times 100\% = 0.83\% \leq 2.5\% \quad OK
\]

For Layer 6, using the revised layer stiffness values:

\[
\varepsilon_{\text{rein}} = \frac{\gamma_{p-EVsf} T_{\text{max}}}{\phi_{sf} R_{cj}} = \frac{1.2 \times 0.292 \text{kips/ft}}{1.0 \times 1.0 \times 17.0 \text{kips/ft}} \times 100\% = 2.06\% \leq 2.5\% \quad OK
\]

See Table 15-E-6 for the calculation results for the rest of the layers. As can be seen in the table, the new (increased) stiffness values are adequate to meet the soil failure limit criterion for all layers.

To estimate the equivalent tensile strength that corresponds to the stiffness values needed to meet the soil failure limit state criterion of 2.5% strain maximum, use the relationships provided in Allen and Bathurst (2019), or alternatively use the product line specific relationships provided in NTPEP (2019, or most current values). Allen and Bathurst (2019) recommend the following generic relationship between creep stiffness and ultimate tensile strength for geogrids:

\[
T_{\text{ult}} = 0.17 \text{j/i}
\] (15-E-28)
For the two stiffness values required to meet the soil failure limit state, the approximate ultimate tensile strength per ft of wall width needed is:

\[ T_{ult} = 0.17 \times (8.6 \text{ kips/ft}) = 1.46 \text{ kips/ft} \quad \text{(applicable to Layer 10)} \]

\[ T_{ult} = 0.17 \times (17.0 \text{ kips/ft}) = 2.89 \text{ kips/ft} \quad \text{(applicable to Layer 6)} \]

To determine \( T_{al} \), divide \( T_{ult} \) by \( RF = 1.12 \times 1.5 \times 1.3 = 2.18 \)

Therefore, the approximate long-term tensile strengths implied by the stiffness required for the soil failure limit state for the reinforcement product (i.e., per ft of reinforcement product width) are:

\[ T_{al} = \frac{1.46 \text{ kips/ft}}{2.18} = 0.67 \text{ kips/ft} \quad \text{(applicable to Layer 10)} \]

\[ T_{al} = \frac{2.89 \text{ kips/ft}}{2.18} = 1.32 \text{ kips/ft} \quad \text{(applicable to Layer 6)} \]

15-E-5.4 Calculations for Reinforcement Rupture Limit State (Strength I)

Since no live load is present, the required long-term reinforcement strength (\( T_{al} \)) for Layer 6 is computed using Equation 15-E-5, solving for \( T_{al} \).

The minimum required value of reinforcement product \( T_{al} \) and \( T_{ult} \), on a strength per reinforcement width basis, for Layer 6 is therefore:

\[ T_{al} \geq \frac{Y_{p-EL} T_{max}}{\phi_{rr} R_c} = \frac{1.35 \times 0.292 \frac{\text{kips}}{\text{ft}}}{0.80 \times 1.0} = 0.49 \frac{\text{kips}}{\text{ft}} \]

\[ T_{ult} = T_{al} R F_{ID} R F_{CR} R F_{D} = 0.49 \times 1.12 \times 1.5 \times 1.3 = 1.07 \frac{\text{kips}}{\text{ft}} \]

For layer 6, \( T_{al} \) for reinforcement rupture is less than \( T_{al} \) needed to achieve the stiffness required for the soil failure limit state (i.e., \( 0.49 \text{ kips/ft} \ll 1.32 \text{ kips/ft} \)). Therefore, the soil failure limit state controls the design. See Table 15-E-6 for the calculation results for the rest of the layers.
15-E-5.5 Calculations for Pullout Limit State Design (Strength I)

The default pullout parameters $\alpha$ and $F^*$ specified in AASHTO (2020) are used for this example design. For geosynthetic reinforcement, $\alpha = 0.8$ and $F^* = 0.67 \times \tan \phi$. Since the design $\phi = 34^\circ$, $F^* = 0.67 \times \tan 34^\circ = 0.452$.

The vertical stress, $\sigma_v$, over the reinforcement anchorage length, $L_e$, can be approximated as:

$$\sigma_v = z\gamma_r + S_{\text{sur}}\gamma_f$$

(15-E-29)

Where,

- $z$ = depth of the reinforcement layer below the wall top,
- $S_{\text{sur}}$ = surcharge height directly above the active zone/resistant zone boundary at the layer,
- $\gamma_r$ = unit weight of reinforced soil backfill, and
- $\gamma_f$ = unit weight of surcharge soil.

Note that the AASHTO (2020) specifications allow the vertical stress to be calculated at the mid-point of $L_e$ relative to the soil surface immediately above the layer at that location. Equation 15-E-29 is a simpler and slightly conservative version of this calculation (for design). Since pullout is usually most critical near the wall top, this example pullout calculation is carried out for Layer 10. Therefore, for Layer 10, the vertical stress used for the pullout calculation is:

$$\sigma_v = (1.33 \text{ ft})(0.13 \text{ kcf}) + (0)(0.13 \text{ kcf}) = 0.173 \text{ ksf}$$

Combining equations 15-E-15 and 15-E-16 and solving for $L_e$, the required factored length of reinforcement in the resistant zone (i.e., behind the active wedge) is:

$$L_e = \frac{\gamma_{p-EV}T_{\text{max}}}{\phi_p C (\alpha F^*) \sigma_v R_c}$$

(15-E-30)

Where,

- $C$ = an overall reinforcement surface geometry factor (set at 2 for strip, grid and sheet type reinforcements).

As before, $R_c = 1.0$ and all other parameters and their values have been defined earlier. For layer 10:

$$L_e = \frac{1.35 \times 0.060 \frac{kips}{ft}}{0.70 \times 2(0.8 \times 0.452)(0.173 \text{ ksf})1.0} = 0.925 \text{ ft}$$
To determine the total reinforcement length needed, \( L \), the length of reinforcement within the active zone, \( L_a \), must be calculated. For geosynthetic walls, the active zone is assumed to be bounded by the Rankine active zone (wedge), illustrated in Figure 15-E-2. \( L_a \) is calculated as follows for a vertical wall (at layer 10):

\[
L_a = (H - z) \tan (45° - \phi_r/2) = (20 \text{ ft} - 1.33 \text{ ft}) \tan (45° - 34°/2) = 9.93 \text{ ft}
\]

The minimum length allowed for \( L_e \) is 3 ft (AASHTO 2020), which is greater than the calculated \( L_e \) required for pullout for layer 10. Therefore, using \( L_e = 3 \text{ ft} \), the total reinforcement length required for layer 10 is:

\[
L = L_a + L_e = 9.93 \text{ ft} + 3 \text{ ft} = 12.9 \text{ ft}
\]

Pullout calculation results for the other layers are summarized in Table 15-E-6.

It should be recalled that the minimum length of reinforcement for typical reinforced soil walls is \( 0.7H = 0.7 \times (20 \text{ ft}) = 14 \text{ ft} \). Therefore, pullout does not control the reinforcement length required. Note that other limit state design calculations can result in greater reinforcement length, such as external stability (e.g., sliding) or global stability.

15-E-5.6 Calculations for Determination of \( T_{\text{max}} + T_{\text{md}} \) (Extreme Event I - Seismic)

The calculation of \( T_{\text{max}} \) as described and carried out earlier in this example using the Stiffness Method is also applicable for seismic design. \( T_{\text{md}} \), the incremental dynamic inertia force per reinforcement layer, must be added to \( T_{\text{max}} \) using superposition to determine the total reinforcement load for each layer during seismic loading.

\( T_{\text{md}} \) is calculated using equations 15-E-17 through 15-E-19 as follows:

\[
A_s = \text{PGA} \times F_{\text{pga}} = 0.50g
\]

(Note: the site PGA and \( F_{\text{pga}} \) is determined from seismic maps provided in GDM Chapter 6; for this example the value of \( A_s \) has been arbitrarily picked for illustration purposes, as this example is not for a specific site).

Assume a maximum lateral deflection of 2 inches is allowed/anticipated. Using Equation 15-E-18, \( k_h \) is determined as follows:

\[
k_h = 0.74A_s \left( \frac{A_s}{d} \right)^{0.25} = 0.74 \times 0.50g \times \left( \frac{0.50}{2} \right)^{0.25} = 0.262g
\]
The inertial force, \( P_i \), is calculated using Equation 15-E-19 as follows:

\[
P_i = k_h \left( \gamma_r \times A_{active} + \gamma_{facing} \times T_f \times H \right) = 0.262 \times \left( 0.130 \text{kcf} \times 0.5 \times \left( 20 \text{ ft} \times \tan \left( 45^\circ - \frac{34^\circ}{2} \right) \right) \times 20 \text{ ft} + 0.0 \text{kcf} \times 0 \text{ ft} \times 0 \text{ ft} \right) = 3.62 \frac{k \text{ips}}{\text{ft}}
\]

And therefore, \( T_{md} \) is calculated using Equation 15-E-17 as shown below:

\[
T_{md} = \left( \frac{P_i}{n} \right) = 3.62 \frac{k \text{ips}}{\text{ft}} \times \frac{10}{1} = 0.362 \frac{k \text{ips}}{\text{ft}}
\]

Note that the weight of any facing was not included in this calculation. If the facing is welded wire, the additional weight would be insignificant. If a second stage concrete facia is added, the weight of that facing should be included in the determination of \( T_{md} \).

The load factor used for Extreme Event I (seismic) is equal to 1.0, and is applied to both the static portion and dynamic portion of the loading, as the probability of failure used for this limit state is much higher than what is used for the Strength Limit state. The use of a higher probability of failure is due to the low probability of occurrence of this load combination as well as greater tolerance for deformation and damage allowed, simply targeting no collapse for life safety. The total load per reinforcement layer during seismic shaking, \( T_{totalf} \), is then calculated using superposition (Equation 15-E-20) as follows, for Layer 6):

\[
T_{totalf} = \gamma_{seis} (T_{max} + T_{ma}) = 1.0 \left( 0.292 \frac{k \text{ips}}{\text{ft}} + 0.362 \frac{k \text{ips}}{\text{ft}} \right) = 0.654 \frac{k \text{ips}}{\text{ft}}
\]

For seismic pullout design, \( T_{totalf} \) is used. However, for reinforcement rupture and connection rupture under seismic loading, the static and dynamic components of the reinforcement load must be handled separately. The reason for this is that the strength required to resist \( T_{max} \) must include the effects of creep because it is a sustained load, but the strength required to resist \( T_{md} \) should not include the effects of creep due to the transient nature of \( T_{md} \).
15-E-5.7 Calculations for Reinforcement Rupture (Extreme Event I - Seismic)

Using equations 15-E-21, 15-E-22, and 15-E-23, $T_{ult}$ for static portion of load at Layer 6 is calculated as follows:

$$S_{rs} = \frac{\gamma_{seis} T_{max} R_F}{\phi R_c} = \frac{1.0 \times 0.292 \text{kips}/\text{ft}}{1.0 \times 1.0} = 0.636 \text{kips}/\text{ft}$$

$T_{ult}$ for dynamic portion of load at Layer 6 is calculated as follows:

$$S_{rt} = \frac{\gamma_{seis} T_{md} R_{FD}}{\phi R_c} = \frac{1.0 \times 0.362 \text{kips}/\text{ft}}{1.0 \times 1.0} = 0.527 \text{kips}/\text{ft}$$

Therefore, the minimum required strength per unit width of reinforcement is as follows:

$$T_{ult} = S_{rs} + S_{rt} = 0.636 \text{kips/ft} + 0.527 \text{kips/ft} = 1.16 \text{kips/ft}$$

$$T_{al} = 1.16 \text{kips/ft}/2.18 = 0.532 \text{kips/ft}$$

For the soil failure limit in the Service Limit State, $T_{al}$ that corresponds to the stiffness needed is 1.32 kips/ft $>>$ 0.532 kips/ft. Therefore, the soil failure limit is still controlling the reinforcement design.

15-E-5.8 Calculations for Pullout Limit State Design (Extreme Event I - Seismic)

Using equations 15-E-27 and 15-E-29, $L_e$ for Layer 10 (i.e., at the wall top) is determined as follows:

$$\sigma_v = z \gamma_f + S_\text{sur} \gamma_l$$

$$\sigma_v = (1.33 \text{ft})(0.13 \text{kcf}) + (0)(0.13 \text{kcf}) = 0.173 \text{ksf}$$

$$L_e = \frac{\gamma_{seis} (T_{max} + T_{md})}{\phi C (0.8 \alpha F^*) \sigma_p R_c} = \frac{1.0 (0.060 \text{kips}/\text{ft} + 0.362 \text{kips}/\text{ft})}{1.0 \times 2 \times (0.8 \times 0.8 \times 0.452) \times 0.173 \text{kcf} \times 1.0}$$

$$L_e = 4.21 \text{ft}$$

$$L_a = (H - z) \tan (45^\circ - \phi_r/2) = (20 \text{ ft} - 1.33 \text{ ft}) \tan (45^\circ - 34^\circ/2) = 9.93 \text{ ft}$$

$$L = L_a + L_e = 9.93 \text{ ft} + 4.21 \text{ ft} = 14.1 \text{ ft}$$

See Table 15-E-6 for the calculation results for the rest of the layers with regard to pullout length required.
15-E-5.9 Calculations to Check to Make Sure Stiffness and Strength Required are Properly Matched (Extreme Event I, Seismic)

If a substantial increase in tensile strength is required to achieve internal stability for Strength Limit or Extreme Event I (seismic) Limit loading, the stiffness needed to obtain the increased tensile strength required must be determined, $T_{\text{max}}$ recalculated, and all limit states recalculated. However, the tensile strength needed for strength and seismic limit design did not increase relative to what was required for the Service limit state. Therefore, no recalculations are required in this case.

15-E-5.10 Results of Example 1 Design Calculations

These calculation results for the Stiffness Method are summarized in Table 15-E-6, and are plotted and compared to design calculation results using the Simplified Method in figures 15-E-3, 15-E-4, and 15-E-5.

In summary, for the final internal stability design for Example 1, the following would be specified with regard to reinforcement properties and spacing for the optimized design, and the following observations can be made:

- The vertical spacing of reinforcements is 2 ft throughout the wall height, and the coverage ratio is a minimum of 1.0. Again, this large a spacing is used in this example to facilitate making direct comparisons with the examples that follow easier.

- The minimum reinforcement length for internal stability (i.e., pullout) is 14 ft for the Strength Limit, and all layers except the top layer for seismic design did not exceed this minimum length for both $T_{\text{max}}$ methods. The top layer length required is 14.1 ft for seismic design for the Stiffness Method (slightly less than this for the Simplified Method), which is slightly greater than the 0.7H minimum. The longer reinforcement length due to pullout is consistent with good detailing for MSE walls in AASHTO (2020) LRFD Article 11.10.7.4.

- Assuming a PET uniaxial geogrid, to meet reinforcement rupture requirements, the reinforcement must have a minimum short-term (ultimate) and long-term (i.e., $T_{\text{al}}$) tensile strength as tabulated in Table 15-E-6. The strength and stiffness needed to meet the Soil Failure Limit State controls the design for all layers. Note that these values are based on strength per unit of reinforcement width. Final selection of reinforcements result in a total $T_{\text{al}}$ for the wall section of 10.6 kips/ft for the Stiffness Method and 11.9 kips/ft for the Simplified Method if the minimum strength needed is set at 0.67 kips/ft based on product availability.

- Overall, the required strength and stiffness of the reinforcement needed is in the low to mid-range of the geogrid product lines available. The Stiffness Method requires less reinforcement than the Simplified Method in the bottom portion of the wall, and in this case seismic does not control the design. The ground acceleration used represents what would be included in Seismic Zone 4, which is the highest seismic zone.
Table 15-E-6  Summary of Example 1 wall design calculations using Stiffness Method and $R_c = 1.0$ (Service, Strength, and Extreme Event I Limit States): (a) Calculation of $T_{\text{max}}$, (b) Service and Strength Limit State calculations, and (c) Extreme Event I (seismic) Limit State calculations.

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<tr>
<th>Layer Number</th>
<th>$z$ (ft)</th>
<th>$S_v$ (ft)</th>
<th>$R_c$</th>
<th>$S_{\text{global}}$ (ksf)</th>
<th>$S_{\text{local}}$ (ksf)</th>
<th>$D_{\text{max}}$</th>
<th>$F_t$</th>
<th>$\Phi_f$</th>
<th>$\Phi_{\text{local}}$</th>
<th>$\Phi_{\text{th}}$</th>
<th>$\Phi_{\text{th}}$</th>
<th>$\sum T_{\text{max}}$ (kips/ft)</th>
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**Base of wall** 20  \( \sum T_{\text{max}} = 2.47 \)

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<th>Reinforcement Product</th>
<th>$T_{ul}$ (kips/ft)</th>
<th>$T_{ul} = T_{ul} \times RF$ (kips/ft)</th>
<th>Factored $\varepsilon_{ul}$ (%)</th>
<th>$\Phi_{\text{local}}$</th>
<th>$\Phi_{\text{th}}$</th>
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<th>Minimum allowed</th>
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<td>0.60</td>
<td>1.30</td>
<td>2.50</td>
<td>1.32</td>
<td>2.89</td>
<td>2.89</td>
<td>0.473</td>
</tr>
<tr>
<td>2</td>
<td>17.33</td>
<td>Geogrid b</td>
<td>0.60</td>
<td>1.30</td>
<td>2.50</td>
<td>1.32</td>
<td>2.89</td>
<td>2.89</td>
<td>0.418</td>
</tr>
<tr>
<td>1</td>
<td>19.33</td>
<td>Geogrid b</td>
<td>0.55</td>
<td>1.19</td>
<td>2.28</td>
<td>1.32</td>
<td>2.89</td>
<td>2.89</td>
<td>0.343</td>
</tr>
</tbody>
</table>

**Base of wall** 20  \( \sum T_{ul} = 4.17 \)  \( \sum T_{ul} = 9.12 \)  \( \sum T_{ul} = 10.6 \)  \( \sum T_{ul} = 23.2 \)
Table 15-E-6, continued

Summary of Example 1 wall design calculations using Stiffness Method and $R_c = 1.0$ (Service, Strength, and Extreme Event I Limit States): (c) Extreme Event I (seismic) Limit State calculations.

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>z (ft)</th>
<th>Soil Failure (Service Limit, for Comparison)</th>
<th>Reinforcement Rupture (Extreme Event I Limit)</th>
<th>Pullout (Extreme Event I Limit)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>*$T_{al}$ (kips/ft) *$T_{al} = T_{al} \times RF$ (kips/ft)</td>
<td>Minimum Required Strength per Unit of Reinforcement</td>
<td>Anchorage length $L_{sup}$ (ft)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_{al}$</td>
<td>$T_{al} = T_{al} \times RF$</td>
<td>Required</td>
</tr>
<tr>
<td>10 (top)</td>
<td>1.33</td>
<td>0.67</td>
<td>0.30</td>
<td>0.66</td>
</tr>
<tr>
<td>9</td>
<td>3.33</td>
<td>0.67</td>
<td>0.33</td>
<td>0.73</td>
</tr>
<tr>
<td>8</td>
<td>5.33</td>
<td>0.67</td>
<td>0.37</td>
<td>0.81</td>
</tr>
<tr>
<td>7</td>
<td>7.33</td>
<td>0.67</td>
<td>0.41</td>
<td>0.90</td>
</tr>
<tr>
<td>6</td>
<td>9.33</td>
<td>1.32</td>
<td>0.53</td>
<td>1.16</td>
</tr>
<tr>
<td>5</td>
<td>11.33</td>
<td>1.32</td>
<td>0.59</td>
<td>1.28</td>
</tr>
<tr>
<td>4</td>
<td>13.33</td>
<td>1.32</td>
<td>0.59</td>
<td>1.30</td>
</tr>
<tr>
<td>3</td>
<td>15.33</td>
<td>1.32</td>
<td>0.59</td>
<td>1.30</td>
</tr>
<tr>
<td>2</td>
<td>17.33</td>
<td>1.32</td>
<td>0.59</td>
<td>1.30</td>
</tr>
<tr>
<td>1</td>
<td>19.33</td>
<td>1.32</td>
<td>0.56</td>
<td>1.23</td>
</tr>
<tr>
<td>Base of wall</td>
<td>20</td>
<td>$\sum T_{max} = 10.6$</td>
<td>$\sum T_{al} = 4.89$</td>
<td>$\sum T_{alt} = 10.7$</td>
</tr>
</tbody>
</table>

*These values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (AASHTO 2020).

*For comparison to geogrid product tensile strength that is not reduced by $R_c$ (i.e., load per unit of reinforcement width basis).

*Tmd for all reinforcement layers is 0.362 kips/ft.
Figure 15-E-3  Comparison of Stiffness Method internal stability design to the Simplified Method design, for Service, Strength, and Extreme Event I (seismic) limit states (Example 1).

Figure 15-E-4  Comparison of Stiffness Method internal stability design to the Simplified Method design, for Extreme Event I (seismic) limit state, pullout (Example 1).
15-E-6  Stiffness Method Design Example 2: Block Faced Geosynthetic Wall with Mechanical Connections, \( R_c = 1.0 \)

15-E-6.1  General

This second example is also a fairly simple design case. Subsequent examples will add features that increase in complexity to illustrate how various scenarios are handled when using the Stiffness Method.

Figure 15-E-5 shows a cross-section of the wall for this design example, and material properties are provided in Table 15-E-7. The reinforcement coverage ratio, \( R_c \) is set at 1.0. Assume for this example that polyester (PET) geogrid will be used for the soil reinforcement. Assume dry cast concrete blocks will be used for the facing. A mechanical facing-geogrid connection will be assumed, so the connection strength is a constant fraction of the geogrid ultimate tensile strength and is not affected by the normal force between the blocks in the facing column. The wall backfill is assumed to be a well-graded gravelly sand, with no cohesion.

The scope of this design example is limited to the service (soil failure limit only), strength, and Extreme Event I (seismic) limit states. Furthermore, for simplicity, live loads are not included in the calculations to follow. It is also assumed that the foundation soil has sufficient shear strength to prevent global instability, bearing capacity failure and excessive settlement. For seismic design, assume that the ground acceleration, \( A_s \), is 0.50g.

Table 15-E-7  Design properties for wall

<table>
<thead>
<tr>
<th>Property</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moist soil unit weight (pcf)</td>
<td>130</td>
</tr>
<tr>
<td>Triaxial drained peak friction angle ( \phi_{tx} ) (°)</td>
<td>34</td>
</tr>
<tr>
<td>Min. available, but not wall system specific, 1,000 hr, 2% secant ( J \times R_c ) (kips/ft)</td>
<td>8.6 \times 1.0 = 8.6</td>
</tr>
<tr>
<td>( RF_{ID} ) ( RF_{CR} ) ( RF_D = RF )</td>
<td>1.12 \times 1.5 \times 1.3 = 2.18</td>
</tr>
<tr>
<td>Coverage ratio, ( R_c )</td>
<td>1.0</td>
</tr>
<tr>
<td>Facing block unit weight, ( \gamma_{block} ) (pcf)</td>
<td>120</td>
</tr>
<tr>
<td>Facing block height (ft)</td>
<td>0.67</td>
</tr>
<tr>
<td>Facing block width, ( W_u ) (face to tail) (ft)</td>
<td>1.0</td>
</tr>
<tr>
<td>Connection strength as fraction of ( T_{ult} ), ( CR_u )</td>
<td>0.75</td>
</tr>
</tbody>
</table>
The wall geometry is based on Figure 15-E-5. As is true in Example 1, Example 2 calculations are demonstrated for one of the middle layers (i.e., Layer 6 in Figure 15-E-5, and for the soil failure and pullout limit states, Layer 10 is also used to illustrate calculations).

S = equivalent uniform height of surcharge = 0 ft. Since the wall in this example is vertical ($\omega = 0$) and is using the same soil as used for Example 1, $K_{abh} = K_{avh} 0.283$, and $\Phi_B = 1.0$.

Since the soil failure limit state usually controls the amount of reinforcement required, this limit state is evaluated first, and $T_{max}$ is calculated, for the wall design.
15-E-6.2 Calculations for Soil Failure Limit State Design (Service I)

As is true for Example 1, the goal of this limit state is to ensure that the factored reinforcement strain in each layer is less than the target maximum strain in the wall required to prevent a contiguous shear surface through the backfill soil from developing (i.e., soil failure limit state). The factored reinforcement peak strain for each layer should be 2.0% or less per AASHTO (2020) for a stiff faced wall such as the modular block faced wall in this example. Some trial-and-error is typically required to establish what reinforcement stiffness values are required, starting with the stiffness values required to meet the soil failure limit, as the soil failure limit often controls the design of geosynthetic walls. As a first trial, use the minimum 1,000-hour 2% strain secant stiffness available for geosynthetic reinforcement products, $J_i = 8.6$ kips/ft per unit of reinforcement width for all layers (see Table 15-E-7).

The contributing factors, coefficients and parameters that comprise the $T_\text{max}$ equation can be found in Allen and Bathurst (2015) and Table 15-E-1. To calculate $T_\text{max}$, the reinforcement stiffness must be per unit of wall width rather than per unit of reinforcement product width; hence, $RcJ_i$ must be used where the reinforcement stiffness value is required. Therefore, the parameters used to determine $T_\text{max}$ are calculated as follows:

$$S_{\text{global}} = \frac{J_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} RcJ_i}{H} = \frac{(10 \times 1.0 \times 8.6 \text{ kips/ft})}{20 \text{ ft}} = 4.30 \text{ ksf (applies to whole wall section)}$$

$$\Phi_g = \alpha \left( \frac{S_{\text{global}}}{P_a} \right)^B = 0.16 \times (4.30 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.193 \text{ (applies to whole wall section)}$$

$$\Phi_{fb} = \frac{K_{a\beta \eta}}{K_{a \text{av} \eta}} = (0.283/0.283)^{0.4} = 1.0 \text{ (applies to whole wall section)}$$

$$S_{\text{local}} = \frac{(J_{\text{ave}})}{S_p} = (8.6 \text{ kips/ft})/(2.0 \text{ ft}) = 4.30 \text{ ksf for Layer 6}$$

$$S_{\text{localave}} = \frac{\sum \left( \frac{J_{\text{ave}}}{S_p} \right)_i}{n} = \frac{3.69 + 8 \times 4.30 + 5.15}{10} = 4.32 \text{ ksf}$$

where, $n = 10$ is the number of reinforcement layers. Therefore, $\Phi_{\text{local}}$ is calculated as follows:

$$\Phi_{\text{local}} = \left( \frac{S_{\text{local}}}{S_{\text{localave}}} \right)^{0.5} = \left( \frac{4.30 \text{ ksf}}{4.32 \text{ ksf}} \right)^{0.5} = 1.00 \text{ (layer 6)}$$
To determine the facing stiffness factor, one must first determine the facing stiffness parameter, \( F_f \). To calculate the facing stiffness parameter \( F_f \), the equivalent height of an unjointed facing column that is approximately 100% efficient in transmitting moment through the height of the facing column, \( h_{eff} \), must be determined. Since the facing stiffness factor \( \Phi_{fs} \) is intended to be a single value for the wall, a single representative value of \( h_{eff} \) must be selected. Typically, \( h_{eff} \) is set equal to the reinforcement vertical spacing in modular block-type structures since the reinforcement is located at the horizontal joints between facing units. For blocks that do not have a reinforcement layer at the horizontal joints, these facings will have better interlock and moment transfer from block to block. If the reinforcement spacing is non-uniform, the smallest predominate spacing (e.g., for 3 or more layers) should be used for this calculation. Smaller \( h_{eff} \) values will lead to more conservative (safer) design because the facing stiffness factor will be larger.

In this example, the vertical spacing is reasonably uniform throughout the wall height at 2.0 ft; thus, \( h_{eff} = 2.0 \) ft is selected in the calculations to follow. The facing stiffness parameter \( F_f \) is calculated using \( h_{eff} = 2.0 \) ft, \( H = 20 \) ft, \( W_u = b = 1 \) ft, and \( E = 157,000 \) ksf. This value for \( E \) is for dry cast concrete, which has a typical value of \( E = 209,000 \) ksf, but has been reduced here to reflect the non-uniform cross-section of the facing unit (typical for modular dry cast concrete blocks). Therefore:

\[
F_f = \frac{1.5 H^3 p_a}{E b h_{eff}} = \frac{1.5(20 \text{ ft})^3(2.11 \text{ ksf})}{(157,000 \text{ ksf})(1 \text{ ft})^3} \frac{2 \text{ ft}}{20 \text{ ft}} = 1.61 \text{ (applies to whole wall section)}
\]

and the facing stiffness factor is:

\[
\Phi_{fs} = \eta \left( \frac{S_{global}}{p_a} \right)^k F_f = 0.57 \times \left( \frac{4.30 \text{ ksf}}{2.11 \text{ ksf}} \right) \times 1.61 = 0.681 \text{ (applies to whole wall section)}
\]

Since \( c = 0 \), the cohesion factor, \( \Phi_c = 1.0 \).

\( D_{tmax} \) does not change relative to the previous calculation (i.e., \( D_{tmax} \) for Layer 6 is 0.825).

\( T_{max} \) for Layer 6 is calculated as follows:

\[
T_{max} = S_v \left[ H Y_r D_{tmax} + \left( \frac{H_{ref}}{H} \right) S Y_f + LL \right] K_{avh} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c
\]

\[
T_{max} = 2.0 \text{ ft} \left[ 20 \text{ ft} \times 0.130 \text{ kcf} \times 0.825 + \left( \frac{20 \text{ ft}}{20 \text{ ft}} \right) 0 \text{ ft} \times 0.130 \text{ kcf} + 0 \right] 0.283 \times 1.0 \times 0.193 \times 0.681 \times 1.00 \times 1.0 = 0.159 \text{ kips ft of wall width}
\]
Using Equation 15-14 with load factor $\gamma_f = 1.2$, and resistance factor $\phi_{sf} = 1.0$ (Table 15-5), the factored reinforcement strain corresponding to layer 6 is computed as:

$$\varepsilon_{rein} = \frac{Y_p - E_{sf} f_{Tmax}}{\phi_{sf} R_{fJi}} = \frac{1.2 \times 0.159 \frac{kips}{ft}}{1.0 \times 1.0 \times 8.6 \frac{kips}{ft}} \times 100\% = 2.21\% \leq 2\% \quad No$$

As shown for Example 1 (Equation 15-E-28), the equivalent ultimate tensile strength that corresponds to the stiffness values needed to meet the soil failure limit state is equal to $0.17 J_i$. For the stiffness value used in this first trial, the approximate ultimate tensile strength per ft of wall width needed is:

$$T_{ult} = 0.17 \times (8.6 \text{ kips/ft}) = 1.46 \text{ kips/ft} \quad (\text{applicable to all layers for first trial})$$

To determine $T_{al}$, divide $T_{ult}$ by $RF = 1.12 \times 1.5 \times 1.3 = 2.18$. Therefore, $T_{al}$ per ft of wall width and per ft of reinforcement width, since $R_c = 1.0$, is:

$$T_{al} = (1.46 \text{ kips/ft})/2.18 = 0.67 \text{ kips/ft} \quad (\text{applicable to all layers})$$

$T_{max}$, the calculated parameters needed to calculate $T_{max}$, and the predicted reinforcement strains for all the layers are summarized in Table 15-E-8 for the rest of the layers.

As can be seen in the table, the calculated factored strains are greater than 2% in the lower half of the wall, which exceeds the soil failure limit state strain criterion of 2% for stiff faced walls. Therefore, the reinforcement stiffness needs to be increased in the lower half of the wall. Through some trial-and-error, the soil failure limit state is met for this example using the stiffness values provided in Table 15-E-9.
Table 15-E-8  Summary of Example 2 wall design calculations using Stiffness Method considering only the Service Limit State, first trial using only the minimum stiffness geogrid product available.

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>z (ft)</th>
<th>Ss (ft)</th>
<th>$^a$RcJi (kips/ft)</th>
<th>Sglobal (ksf)</th>
<th>Slocal (ksf)</th>
<th>Dmax</th>
<th>Ff</th>
<th>$^a$Φg</th>
<th>$^a$Φlocal</th>
<th>$^a$Φfb</th>
<th>$^a$Φfs</th>
<th>$^a$Tmax (kips/ft) (Equation 15-E-1)</th>
<th>Factor d $g_{fas}$ (%)</th>
<th>Soil Failure Limit State</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 (top)</td>
<td>1.33</td>
<td>2.33</td>
<td>8.6</td>
<td>4.30</td>
<td>3.69</td>
<td>0.220</td>
<td>1.61</td>
<td>0.193</td>
<td>0.92</td>
<td>1.0</td>
<td>0.681</td>
<td>0.046</td>
<td>0.64</td>
<td>0.67</td>
</tr>
<tr>
<td>9</td>
<td>3.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>4.30</td>
<td>0.372</td>
<td>1.61</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>0.681</td>
<td>0.071</td>
<td>0.71</td>
<td>0.67</td>
</tr>
<tr>
<td>8</td>
<td>5.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>4.30</td>
<td>0.523</td>
<td>1.61</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>0.681</td>
<td>0.101</td>
<td>1.00</td>
<td>0.67</td>
</tr>
<tr>
<td>7</td>
<td>7.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>4.30</td>
<td>0.674</td>
<td>1.61</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>0.681</td>
<td>0.130</td>
<td>1.00</td>
<td>0.67</td>
</tr>
<tr>
<td>6</td>
<td>9.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>4.30</td>
<td>0.825</td>
<td>1.61</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>0.681</td>
<td>0.159</td>
<td>1.00</td>
<td>0.67</td>
</tr>
<tr>
<td>5</td>
<td>11.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>4.30</td>
<td>0.976</td>
<td>1.61</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>0.681</td>
<td>0.188</td>
<td>2.02</td>
<td>0.67</td>
</tr>
<tr>
<td>4</td>
<td>13.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>4.30</td>
<td>1.00</td>
<td>1.61</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>0.681</td>
<td>0.192</td>
<td>2.68</td>
<td>0.67</td>
</tr>
<tr>
<td>3</td>
<td>15.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>4.30</td>
<td>1.00</td>
<td>1.61</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>0.681</td>
<td>0.192</td>
<td>2.68</td>
<td>0.67</td>
</tr>
<tr>
<td>2</td>
<td>17.33</td>
<td>2.00</td>
<td>8.6</td>
<td>4.30</td>
<td>4.30</td>
<td>1.00</td>
<td>1.61</td>
<td>0.193</td>
<td>1.00</td>
<td>1.0</td>
<td>0.681</td>
<td>0.192</td>
<td>2.68</td>
<td>0.67</td>
</tr>
<tr>
<td>1</td>
<td>19.33</td>
<td>1.67</td>
<td>8.6</td>
<td>4.30</td>
<td>5.15</td>
<td>1.00</td>
<td>1.61</td>
<td>0.193</td>
<td>1.09</td>
<td>1.0</td>
<td>0.681</td>
<td>0.176</td>
<td>2.45</td>
<td>0.67</td>
</tr>
<tr>
<td>Base of Wall</td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\sum T_{\text{max}} = 1.45$</td>
<td></td>
<td>$\sum T_{\text{at}} = 6.70$</td>
</tr>
</tbody>
</table>

*Minimum stiffness needed to meet only the soil failure limit, considering all geosynthetic reinforcement products, and not just the specific products available for the wall system (i.e., weaker than Geogrid A in this example).

$^a$All values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (AASHTO 2020).

$^a$For comparison to geogrid product MARV tensile strength that is not reduced by Rc (i.e., load per unit of reinforcement width basis).
Table 15-E-9  Creep stiffness values (i.e., at 2% strain and 1,000 h) for geogrids used in wall

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Geogrid Designation</th>
<th>Ji (per Unit Width of Reinforcement, in kips/ft)</th>
<th>*Rc × Ji (per Unit Width of Wall, in kips/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>a</td>
<td>8.6</td>
<td>8.6</td>
</tr>
<tr>
<td>9</td>
<td>a</td>
<td>8.6</td>
<td>8.6</td>
</tr>
<tr>
<td>8</td>
<td>a</td>
<td>8.6</td>
<td>8.6</td>
</tr>
<tr>
<td>7</td>
<td>a</td>
<td>8.6</td>
<td>8.6</td>
</tr>
<tr>
<td>6</td>
<td>b</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>5</td>
<td>b</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>4</td>
<td>b</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>3</td>
<td>b</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>2</td>
<td>b</td>
<td>14.5</td>
<td>14.5</td>
</tr>
<tr>
<td>1</td>
<td>b</td>
<td>14.5</td>
<td>14.5</td>
</tr>
</tbody>
</table>

*This is the stiffness value which has been corrected for Rc to calculate Tmax. Since Rc = 1.0, this stiffness is the same as the stiffness per unit of reinforcement width.

Using the Table 15-E-9 stiffness values, considering a total of 10 layers, Sglobal is recalculated as:

\[
S_{\text{global}} = \frac{J_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} R_c J_i}{H} = \frac{(4 \times 1.0 \times 8.6 \text{ kips/ft} + 6 \times 1.0 \times 14.5 \text{ kips/ft})}{20 \text{ ft}} = 6.07 \text{ ksf}
\]

\[
\Phi_g = \alpha \left( \frac{S_{\text{global}}}{p_a} \right)^\beta = 0.16 \times (6.07 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.211
\]

Slocal is recalculated as:

\[
S_{\text{local}} = \left( \frac{R_c}{S_y} \right)_i = \frac{(1.0 \times 14.5 \text{ kips/ft})}{(2.0 \text{ ft})} = 7.25 \text{ ksf for Layer 6}
\]

\[
S_{\text{localave}} = \frac{\sum \left( \frac{R_c}{S_y} \right)_i}{n} = \frac{3.69 + 3 \times 4.30 + 5 \times 7.25 + 8.68}{10} = 6.15 \text{ ksf}
\]

where, n = 10 is the number of reinforcement layers. Therefore, \( \Phi_{\text{local}} \) is recalculated as follows:

\[
\Phi_{\text{local}} = \left( \frac{S_{\text{local}}}{S_{\text{localave}}} \right)^{0.5} = \left( \frac{7.25 \text{ ksf}}{6.15 \text{ ksf}} \right)^{0.5} = 1.09 \quad (\text{Layer 6})
\]
To determine the facing stiffness factor, the facing stiffness parameter, $F_f$ does not change, and the facing stiffness factor is recalculated as:

$$
\Phi_{fs} = n \left( \frac{S_{global}}{p_a} \right)^\kappa \times \left( \frac{6.07 \text{ ksf}}{2.11 \text{ ksf}} \times 1.61 \right)^{0.15} = 0.718
$$

Since $c = 0$, the cohesion factor, $\Phi_c = 1.0$.

$D_{tmax}$ remains unchanged.

$T_{max}$ for Layer 6 is now recalculated as follows:

$$
T_{max} = S_v \left[ H \gamma r \cdot D_{tmax} + \left( \frac{H_{ref}}{H} \right) S \gamma r \cdot L \right] K_{avh} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c
$$

$$
T_{max} = 2.0 \text{ ft} \left[ 20 \text{ ft} \times 0.130 \text{ kcf} \times 0.825 + \left( \frac{20 \text{ ft}}{20 \text{ ft}} \right) 0 \text{ ft} \times 0.130 \text{ kcf} + 0 \right] \times 1.0 \times 0.211 \times 0.718 \times 1.09 \times 1.0
$$

$$
T_{max} = 0.199 \text{ kips/ft (load per unit of wall width)}
$$

$T_{max}$, and the recalculated parameters needed to calculate $T_{max}$, are summarized in Table 15-E-10 for the rest of the layers for the Service and Strength limit states.

Using Equation 15-14 with load factor $\gamma = 1.2$, and resistance factor $\phi = 1.0$ (Table 15-5), the factored reinforcement strain corresponding to layer 10 is recomputed as:

$$
\epsilon_{rein} = \frac{\gamma_p \cdot E}{{\phi_s f R}_{cji}} T_{max} = \frac{1.2 \times 0.044 \text{ kips/ft}}{1.0 \times 1.0 \times 8.6 \text{ kips/ft}} \times 100\% = 0.62\% \leq 2\% \quad OK
$$

For layer 6:

$$
\epsilon_{rein} = \frac{\gamma_p \cdot E}{{\phi_s f R}_{cji}} T_{max} = \frac{1.2 \times 0.199 \text{ kips/ft}}{1.0 \times 1.0 \times 14.5 \text{ kips/ft}} \times 100\% = 1.65\% \leq 2\% \quad OK
$$

See Table 15-E-10 for the calculation results for the rest of the layers. Therefore, the soil failure limit state is met for all the reinforcement layers.

To estimate the equivalent tensile strength that corresponds to the stiffness values needed to meet the soil failure limit state criterion of 2% strain maximum, use the relationships provided in Allen and Bathurst (2019), i.e., $T_{ult} = 0.17 J_i$, or alternatively use the product line specific relationships provided in NTPEP (2019).

For the two stiffness values required to meet the soil failure limit state, the approximate ultimate tensile strength per unit of wall width to obtain the stiffness needed is:

$$
T_{ult} = 0.17 \times (8.6 \text{ kips/ft}) = 1.46 \text{ kips/ft} \quad \text{(applicable to Layer 10)}
$$

$$
T_{ult} = 0.17 \times (14.5 \text{ kips/ft}) = 2.47 \text{ kips/ft} \quad \text{(applicable to Layer 6)}
$$

To determine $T_{ali}$, divide $T_{ult}$ by $RF = 1.12 \times 1.5 \times 1.3 = 2.18$
Therefore, the approximate long-term tensile strengths implied by the stiffness required for the soil failure limit state for the reinforcement product (i.e., per unit of wall width and reinforcement product width, since $R_c = 1.0$) are:

$$T_{al} = \frac{1.46 \text{ kips/ft}}{2.18} = 0.67 \text{ kips/ft} \quad \text{(applicable to Layer 10)}$$

$$T_{al} = \frac{2.47 \text{ kips/ft}}{2.18} = 1.13 \text{ kips/ft} \quad \text{(applicable to Layer 6)}$$

### 15-E-6.3 Calculations for Reinforcement Rupture Limit State (Strength I)

Since no live load is present, the required long-term reinforcement strength ($T_{al}$) for Layer 6 is computed using Equation 15-E-5.

The minimum required value of $T_{al}$ and $T_{ult}$ for Layer 6, on a strength per unit of reinforcement width basis (i.e., this value is what would be compared to the MARV of the tensile strength of specific reinforcement products), is therefore:

$$T_{mr} = \frac{Y_{p-EV} T_{max}}{\phi_{rr} R_c} = \frac{1.35 \times 0.199 \text{kips/ft}}{0.80 \times 1.0} = 0.338 \text{kips/ft}$$

$$T_{ult} = T_{al} R F_{D} R F_C R F_D = 0.338 \times 1.12 \times 1.5 \times 1.3 = 0.737 \text{kips/ft}$$

For layer 6, $T_{al}$ for reinforcement rupture is significantly less than $T_{al}$ needed to achieve the stiffness required for the soil failure limit state (i.e., $0.338 \text{kips/ft} < 1.13 \text{kips/ft}$).

### 15-E-6.4 Calculations for Connection Strength Design (Strength I)

To determine the minimum tensile strength needed at the connection to the facing, connection strength data for the facing block – geosynthetic combinations anticipated are needed. It has been assumed for this example that a mechanical type connection between the facing blocks and geogrid will be used (i.e., not dominated by friction).

For the purposes of this example and to be consistent with the approach taken in the current AASHTO (2020) specifications, the load and resistance factors for the connection rupture limit state are assigned the same values as those used for geosynthetic rupture limit state at locations away from the connection (i.e., $\gamma_{p-EV} = \gamma_{p-EV} = 1.35$ and $\phi_{cr} = \phi_{rr} = 0.80$), and $T_0 = T_{max}$.

To determine the long-term connection strength available, since a mechanical connection is used in this example, the connection strength will not be a function of the normal load on the facing blocks. Expressed as a portion of $T_{l0}$, the roll specific tensile strength for the connection testing, the short-term connection strength $C R_u = \frac{T_{ult} \text{conn}}{T_{l0}} = 0.75$. For this example, it will be assumed that this value of $C R_u$ is applicable for all geogrids (this is likely not the case, but to keep the example as simple as possible, this assumption is made).
Equation 15-E-9 is used to calculate the minimum long-term connection strength needed, $T_{ac, \text{required}}$, which essentially is the factored connection load. For the purposes of this example and to be consistent with the approach taken in the current AASHTO (2020) specifications, the load and resistance factors for the connection rupture limit state are assigned the same values as those used for geosynthetic rupture limit state at locations away from the connection (i.e., $\gamma_{p-EVc} = \gamma_{p-EV} = 1.35$ and $\phi_{cr} = \phi_{rr} = 0.80$), and $T_o = T_{\text{max}}$.

Therefore, using the load side of the connection limit state design Equation 15-E-9, at Layer 6, the factored connection load is calculated as follows:

$$T_{ac, \text{required}} = (\gamma_{p-EVc})T_0 = (1.35) \times 0.199 \frac{\text{kips}}{ft} = 0.269 \frac{\text{kips}}{ft} \text{ of wall width}$$

To determine the long-term connection strength available, since a mechanical connection is used in this example, the connection strength will not be a function of the normal load on the facing blocks, $N$. Using the resistance side of Equation 15-E-9 (i.e., the limit state equation for connection design), and the equation for $T_{ac}$ (Equation 15-E-11), the available long-term connection strength available is calculated as follows, assuming that the minimum $T_{ult}$ needed is equal to the $T_{ult}$ needed to obtain the stiffness required to meet the soil failure limit state:

$$T_{ac, \text{available}} = \phi_{cr} T_{ac} R_c = \frac{\phi_{cr} \times T_{ult} \times (C_{Ra} \times R_D \times F_C \times R_F)}{C_{Ra} \times R_D} \tag{15-E-31}$$

$$T_{ac, \text{available}} = \frac{0.80 \times 2.47 \frac{\text{kips}}{ft} \times (0.75 \times 1.0)}{1.3} = 0.760 \text{ kips/ft}$$

$0.269 \text{ kips/ft} < 0.760 \text{ kips/ft}?$  OK

Combining Equation 15-E-9 with Equation 15-E-31 and solving for $T_{ult}$, at layer 6, can determine the minimum $T_{ult}$ required to just satisfy connection strength requirements as follows:

$$T_{ult, \text{min. required}} = (\gamma_{p-EVc})T_0 \times R_D \times F_C \times \left(\frac{1}{C_{Ra} \times R_D} \right) \tag{15-E-32}$$

$$T_{ult, \text{min. required}} = \frac{1.35}{0.8 \times 1.0} \times 0.199 \times 1.3 \times 1.5 \left(\frac{1}{0.75} \right) = 0.873 \frac{\text{kips}}{ft}$$

On a strength per unit of reinforcement width basis (and also strength per unit of wall width, since $R_c = 1.0$), this minimum required geosynthetic $T_{ult}$ of 0.873 kips/ft is below the $T_{ult}$ value of 2.47 kips/ft (i.e., $T_{a} = 2.47/2.18 = 1.13 \text{ kips/ft}$) estimated to provide the needed stiffness for the soil failure limit state. Therefore, the soil failure limit state is still controlling the tensile strength required at this point (i.e., only considering the Service and Strength Limit States).
15-E-6.5 Calculations for Pullout Limit State Design (Strength I)

The default pullout parameters $\alpha$ and $F^*$ specified in AASHTO (2020) are used for this example design and are the same as in Example 1 ($\alpha = 0.8$ and $F^* = 0.452$). The vertical stress, $\sigma_v$, over the reinforcement anchorage length, conservatively calculated using Equation 15-E-29, is the same as in Example 1 (0.173 ksf). Since pullout is usually most critical near the wall top, this example pullout calculation is carried out for Layer 10.

As before, $R_c = 1.0$. Using Equation 15-E-30, the required factored length of reinforcement in the resistant zone (i.e., behind the active wedge) for Layer 10 is calculated as follows:

$$L_e = \frac{\gamma_p \gamma_{Ev} T_{max}}{\phi_{po} C(\alpha F^*) \sigma_v R_c}$$

$$L_e = \frac{1.35 \times 0.044 \text{kips/ft}}{0.70 \times 2(0.8 \times 0.452)(0.173 \text{ksf})1.0} = 0.678 \text{ft}$$

To determine the total reinforcement length needed, $L$, the length of reinforcement within the active zone, $L_a$, must be calculated. For geosynthetic walls, the active zone is assumed to be bounded by the Rankine active zone (wedge). $L_a$ is calculated as follows for a vertical wall (at layer 10):

$$L_a = (H - z) \tan \left(45^\circ - \frac{\phi_r}{2}\right) = (20 \text{ ft} - 1.33 \text{ ft}) \tan \left(45^\circ - 34^\circ/2\right) = 9.93 \text{ ft}$$

The minimum length allowed for $L_e$ is 3 ft (AASHTO 2020), which is greater than the calculated $L_e$ required for pullout for layer 10. Therefore, using $L_e = 3 \text{ ft}$, the total reinforcement length required for layer 10 is:

$$L = L_a + L_e = 9.93 \text{ ft} + 3 \text{ ft} = 12.9 \text{ ft}$$

Pullout calculation results for the other layers are summarized in Table 15-E-10. It should be recalled that the minimum length of reinforcement for typical reinforced soil walls is $0.7H = 0.7 \times (20 \text{ ft}) = 14 \text{ ft}$. Therefore, pullout does not control the reinforcement length required. Note that other limit state design calculations can result in greater reinforcement length, such as external stability (e.g., sliding) or global stability.
15-E-6.6 Calculations for Determination of $T_{\text{max}} + T_{\text{md}}$ (Extreme Event I - Seismic)

The calculation of $T_{\text{max}}$ as described and carried out earlier in this example using the Stiffness Method is also applicable for seismic design for the static portion of the reinforcement load. $T_{\text{md}}$, the incremental dynamic inertia force per reinforcement layer, must be added to $T_{\text{max}}$ to determine the total reinforcement load for each layer during seismic loading.

$T_{\text{md}}$ is calculated using equations 15-E-17 through 15-E-19 as follows:

$$A_s = \text{PGA} \times F_{pga} = 0.50g \quad \text{(same as in previous example)}$$

Assume a maximum lateral deflection of 2 inches is allowed/anticipated. Therefore,

$$k_h = 0.74A_s \left( \frac{A_s}{d} \right)^{0.25} = 0.74 \times 0.50g \times \left( \frac{0.50}{2} \right)^{0.25} = 0.262g$$

Including the weight of the facing blocks, using Equation 15-E-19,

$$P_i = k_h \left( \gamma_r \times A_{\text{active}} + \gamma_{\text{facing}} \times T_f \times H \right)$$

$$= 0.262 \times \left( 0.130 \text{kcf} \times 0.5 \times \left( 20 \text{ ft} \times Tan \left( 45 - \frac{34^\circ}{2} \right) \right) \times 20 \text{ ft}
+ 0.12 \text{kcf} \times 1 \text{ ft} \times 20 \text{ ft} \right) = 4.24 \frac{\text{kips}}{\text{ft}}$$

Using Equation 15-E-17,

$$T_{\text{md}} = \left( \frac{P_i}{n} \right) = \frac{4.24 \frac{\text{kips}}{\text{ft}}}{10} = 0.424 \frac{\text{kips}}{\text{ft}}$$

The total load per reinforcement layer, on a load per unit of wall width basis, during seismic shaking, $T_{\text{total f}}$, is then calculated using superposition as follows, for Layer 6:

$$T_{\text{total f}} = \gamma_{\text{seis}} (T_{\text{max}} + T_{\text{md}}) = \gamma_{\text{seis}} \left( 0.199 \frac{\text{kips}}{\text{ft}} + 0.424 \frac{\text{kips}}{\text{ft}} \right) = 0.623 \frac{\text{kips}}{\text{ft}}$$

For seismic pullout design, $T_{\text{total f}}$ is used. However, for reinforcement rupture and connection rupture under seismic loading, the static and dynamic components of the reinforcement load must be handled separately. The reason for this is that the strength required to resist $T_{\text{max}}$ must include the effects of creep because it is a sustained load, but the strength required to resist $T_{\text{md}}$ should not include the effects of creep due to the transient nature of $T_{\text{md}}$. 
15-E-6.7 Calculations for Reinforcement Rupture (Extreme Event I - Seismic)

Using equations 15-E-21, 15-E-22, and 15-E-23, calculate $T_{ult}$ for static portion of load at Layer 6:

$$S_{rs} = \frac{Y_{seis}T_{max}RF}{\phi R_c} = \frac{1.0 \times 0.199 \frac{kips}{ft} \times 2.18}{1.0 \times 1.0} = 0.434 \frac{kips}{ft}$$

$T_{ult}$ for dynamic portion of load at Layer 6:

$$S_{rt} = \frac{Y_{seis}T_{mag}RF_{ID}RF_{D}}{\phi R_c} = \frac{1.0 \times 0.424 \frac{kips}{ft} \times 1.12 \times 1.3}{1.0 \times 1.0} = 0.617 \frac{kips}{ft}$$

$$T_{ult} = S_{rs} + S_{rt} = 0.434 \text{ kips/ft} + 0.617 \text{ kips/ft} = 1.05 \text{ kips/ft of reinforcement unit width}$$

$$T_{al} = 1.05 \text{ kips/ft/2.18} = 0.483 \text{ kips/ft of reinforcement unit width}$$

15-E-6.8 Calculations for Connection Rupture (Extreme Event I - Seismic)

Using equations 15-E-24, 15-E-25, and 15-E-26, $T_{ult}$ for static portion of load at Layer 6 is determined as follows:

$$C_{Rcr} = \frac{C_{Rc}}{RF_{CR}} = \frac{0.75}{1.5} = 0.500$$

Since this is a mechanical connection, $F_r$ is set equal to 1.0.

$$S_{rsc} = \frac{Y_{seis}T_{mag}RF_{D}}{F_r\phi CR_{cr}R_c} = \frac{1.0 \times 0.199 \frac{kips}{ft} \times 1.3}{1.0 \times 1.0 \times 0.500 \times 1.0} = 0.517 \frac{kips}{ft}$$

$$S_{rtc} = \frac{Y_{seis}T_{mag}RF_{D}}{F_r\phi CR_{cr}R_c} = \frac{1.0 \times 0.424 \frac{kips}{ft} \times 1.3}{1.0 \times 1.0 \times (0.75) \times 1.0} = 0.735 \frac{kips}{ft}$$

$$T_{ult} = S_{rsc} + S_{rtc} = 0.517 \text{ kips/ft} + 0.735 \text{ kips/ft} = 1.25 \text{ kips/ft of reinforcement product width}$$

On a strength per reinforcement width basis, this minimum required geosynthetic $T_{ult}$ of 1.25 kips/ft is still below the $T_{ult}$ value of 2.47 kips/ft (i.e., a $T_{al} = 2.47/2.18 = 1.13$ kips/ft) estimated to provide the needed stiffness for the soil failure limit state. So the soil failure limit state is still controlling the tensile strength required considering the Extreme Event I Limit State (i.e., seismic).
15-E-6.9 Calculations for Pullout Limit State Design (Extreme Event I - Seismic)

Using Equation 15-E-27, \( L_e \) for Layer 10 (i.e., at the wall top) is determined as follows:

\[
L_e = \frac{\gamma f f_{li} (T_{max} + T_{md})}{\phi C (0.8 \alpha F^*) \sigma_v R_C} = \frac{1.0(0.044 + 0.424)}{1.0 \times 2 \times (0.8 \times 0.8 \times 0.452) \times 0.173 \times 1.0}
\]

\[ L_e = 4.68 \text{ ft} \]

\[ L_a = (H - z) \tan (45^\circ - \phi_r/2) = (20 \text{ ft} - 1.33 \text{ ft}) \tan (45^\circ - 34^\circ/2) = 9.93 \text{ ft} \]

\[ L = L_a + L_e = 9.93 \text{ ft} + 4.68 \text{ ft} = 14.6 \text{ ft} \]

This required length is just barely greater than 70% of the wall height, so it does control pullout length at the wall top.

15-E-6.10 Calculations to Check to Make Sure Stiffness and Strength Required are Properly Matched (Extreme Event I, Seismic)

Since the soil failure limit state is still controlling the design, the stiffness determined to meet the soil failure limit state is still the correct stiffness to use. Therefore, an additional iteration with stiffness values that are consistent with the tensile strengths needed is not required. Had one of the other limit states controlled the \( T_{ult} \) needed, then it would have been necessary to recheck the design using an increased stiffness value that is consistent with the higher tensile strength. Fortunately, this does not happen very often (may only occur for block faced walls with very inefficient connections between the geosynthetic and the facing blocks).

15-E-6.11 Summary for Example 2 Design

See Table 15-E-10 for the calculation results for all of the layers for the Service I, Strength I, and Extreme Event I limit states. These calculation results are plotted and compared to design calculation results using the Simplified Method in figures 15-E-6 through 15-E-8.

In summary, for the final internal stability design for Example 2 using the Stiffness Method, the following would be specified with regard to reinforcement properties and spacing for the optimized design, and the following observations can be made:

- The vertical spacing of reinforcements is 2 ft throughout the wall height, and the coverage ratio is a minimum of 1.0.

- The minimum reinforcement length for internal stability (i.e., pullout) is 14 ft for the Strength Limit, and all layers except the top layer for seismic did not exceed this minimum length. The top layer length required is 14.6 ft for seismic for the Stiffness Method, which is greater than the 0.7H minimum (see Figure 15-E-8). The longer reinforcement length due to pullout is consistent with good detailing for MSE walls in AASHTO LRFD Article 11.10.7.4.
• The lowest strength PET geogrid available was greater in strength and stiffness than required by the Stiffness Method design for the top 4 layers (see figures 15-E-6 and 15-E-7). This was not the case for the Simplified Method, as higher reinforcement strengths than the minimum available were required for most of the layers.

• The Simplified Method required a total long-term tensile strength $T_{al}$ of 13.2 kips/ft for the entire wall section, whereas the Stiffness Method required 5.4 kips/ft for the entire wall section for seismic reinforcement connection rupture. However, the $T_{al}$ needed to obtain the stiffness needed to meet the soil failure limit state controlled the design, for which the total $T_{al}$ for the wall section was 9.4 kips/ft, which is just over 70% of the total tensile strength needed by the Simplified Method. Note that in the upper third of the wall, all limit states for both methods will be limited to the minimum strength shown in the plots as a dashed vertical line. If that is considered, the Simplified Method $T_{al}$ required would increase to 13.9 kips/ft, making the Stiffness Method required soil reinforcement strength equal to 68% of what is required by the Simplified Method.

• Overall, the required strength and stiffness of the reinforcement needed is in the low to mid-range of the geogrid product lines available. Note that the ground acceleration used for the seismic design represents what would be included in Seismic Zone 4, which is the highest seismic zone.
Table 15-E-10  Summary of Example 2 wall design calculations using Stiffness Method and $R_c = 1.0$ (Service, Strength, and Extreme Event I Limit States): (a) Calculation of $T_{max}$, (b) Service and Strength Limit State calculations, and (c) Extreme Event I (seismic) Limit State calculations.

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<th>$z$ (ft)</th>
<th>$S$, (ft)</th>
<th>$R_c J_i$ (kips/ft)</th>
<th>$S_{global}$ (ksf)</th>
<th>$S_{local}$ (ksf)</th>
<th>$D_{max}$</th>
<th>$F_t$</th>
<th>$F_e$</th>
<th>$F_{local}$</th>
<th>$F_{n}$</th>
<th>$^\ast T_{max}$ and $T_0$ (kips/ft)</th>
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b)  

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<th>Connection Rupture (Strength Limit)</th>
<th>Soil Failure (Service Limit)</th>
<th>Pullout (Strength Limit)</th>
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</thead>
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<td>$T_a$ (kips/ft)</td>
<td>$T_{all}$ (kips/ft)</td>
<td>$T_{all}$ (kips/ft)</td>
<td>$\varepsilon_{min}$ (%)</td>
</tr>
<tr>
<td>10 (top)</td>
<td>1.33</td>
<td>Geogrid a</td>
<td>0.075</td>
<td>0.16</td>
<td>0.19</td>
<td>0.62</td>
</tr>
<tr>
<td>9</td>
<td>3.33</td>
<td>Geogrid a</td>
<td>0.12</td>
<td>0.25</td>
<td>0.30</td>
<td>0.96</td>
</tr>
<tr>
<td>8</td>
<td>5.33</td>
<td>Geogrid a</td>
<td>0.16</td>
<td>0.36</td>
<td>0.43</td>
<td>1.35</td>
</tr>
<tr>
<td>7</td>
<td>7.33</td>
<td>Geogrid a</td>
<td>0.21</td>
<td>0.46</td>
<td>0.55</td>
<td>1.75</td>
</tr>
<tr>
<td>6</td>
<td>9.33</td>
<td>Geogrid b</td>
<td>0.34</td>
<td>0.73</td>
<td>0.87</td>
<td>1.65</td>
</tr>
<tr>
<td>5</td>
<td>11.33</td>
<td>Geogrid b</td>
<td>0.40</td>
<td>0.87</td>
<td>1.03</td>
<td>1.95</td>
</tr>
<tr>
<td>4</td>
<td>13.33</td>
<td>Geogrid b</td>
<td>0.41</td>
<td>0.89</td>
<td>1.06</td>
<td>2.00</td>
</tr>
<tr>
<td>3</td>
<td>15.33</td>
<td>Geogrid b</td>
<td>0.41</td>
<td>0.89</td>
<td>1.06</td>
<td>2.00</td>
</tr>
<tr>
<td>2</td>
<td>17.33</td>
<td>Geogrid b</td>
<td>0.41</td>
<td>0.89</td>
<td>1.06</td>
<td>2.00</td>
</tr>
<tr>
<td>1</td>
<td>19.33</td>
<td>Geogrid b</td>
<td>0.37</td>
<td>0.81</td>
<td>0.97</td>
<td>1.82</td>
</tr>
<tr>
<td>Base of wall</td>
<td>20</td>
<td></td>
<td>$\sum T_{all}=2.89$</td>
<td>$\sum T_{all}=6.31$</td>
<td>$\sum T_{all}=7.52$</td>
<td>$\sum T_{all}=9.4$</td>
</tr>
</tbody>
</table>
Table 15-E-10, continued

Summary of Example 2 wall design calculations using Stiffness Method and $R_c = 1.0$ (Service, Strength, and Extreme Event I Limit States): (c) Extreme Event I (seismic) Limit State calculations.

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>z (ft)</th>
<th>Reinforcement Product</th>
<th>Soil Failure (Service Limit, for Comparison)</th>
<th>Reinforcement Rupture (Extreme Event I Limit)</th>
<th>Connection Rupture (Extreme Event I Limit)</th>
<th>Pullout (Extreme Event I Limit)</th>
<th>Anchorage length $L_{min}$ (ft)</th>
<th>Total Reinforcement Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 (top)</td>
<td>1.33</td>
<td>Geogrid a</td>
<td>0.67</td>
<td>0.33</td>
<td>0.71</td>
<td>0.75</td>
<td>0.39</td>
<td>0.85</td>
</tr>
<tr>
<td>9</td>
<td>3.33</td>
<td>Geogrid a</td>
<td>0.67</td>
<td>0.35</td>
<td>0.77</td>
<td>0.75</td>
<td>0.42</td>
<td>0.92</td>
</tr>
<tr>
<td>8</td>
<td>5.33</td>
<td>Geogrid a</td>
<td>0.67</td>
<td>0.38</td>
<td>0.83</td>
<td>0.75</td>
<td>0.45</td>
<td>0.99</td>
</tr>
<tr>
<td>7</td>
<td>7.33</td>
<td>Geogrid a</td>
<td>0.67</td>
<td>0.41</td>
<td>0.89</td>
<td>0.75</td>
<td>0.49</td>
<td>1.06</td>
</tr>
<tr>
<td>6</td>
<td>9.33</td>
<td>Geogrid a</td>
<td>1.13</td>
<td>0.48</td>
<td>1.05</td>
<td>0.75</td>
<td>0.57</td>
<td>1.25</td>
</tr>
<tr>
<td>5</td>
<td>11.33</td>
<td>Geogrid b</td>
<td>1.13</td>
<td>0.52</td>
<td>1.13</td>
<td>0.75</td>
<td>0.62</td>
<td>1.35</td>
</tr>
<tr>
<td>4</td>
<td>13.33</td>
<td>Geogrid b</td>
<td>1.13</td>
<td>0.52</td>
<td>1.14</td>
<td>0.75</td>
<td>0.62</td>
<td>1.36</td>
</tr>
<tr>
<td>3</td>
<td>15.33</td>
<td>Geogrid b</td>
<td>1.13</td>
<td>0.52</td>
<td>1.14</td>
<td>0.75</td>
<td>0.62</td>
<td>1.36</td>
</tr>
<tr>
<td>2</td>
<td>17.33</td>
<td>Geogrid b</td>
<td>1.13</td>
<td>0.52</td>
<td>1.14</td>
<td>0.75</td>
<td>0.62</td>
<td>1.36</td>
</tr>
<tr>
<td>1</td>
<td>19.33</td>
<td>Geogrid b</td>
<td>1.13</td>
<td>0.50</td>
<td>1.10</td>
<td>0.75</td>
<td>0.60</td>
<td>1.31</td>
</tr>
</tbody>
</table>

$\sum T_{ul} = 9.4 \quad \sum T_{ul} = 4.54 \quad \sum T_{ult} = 9.92 \quad \sum T_{ul} = 5.41 \quad \sum T_{ult} = 11.8$

*These values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (AASHTO 2020).

*For comparison to geogrid product MARV tensile strength that is not reduced by $R_c$ (i.e., load per unit of reinforcement width basis).

*T_{md} for all reinforcement layers is 0.424 kips/ft.
Figure 15-E-6 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Service and Strength limit states, block faced wall with mechanical connection, $R_c = 1.0$ (Example 2).

Figure 15-E-7 Comparison of Stiffness Method internal stability design to the Simplified Method design, for Service and Extreme Event I (seismic) limit states, block faced wall with mechanical connection, $R_c = 1.0$ (Example 2).
Figure 15-E-8  Comparison of Stiffness Method internal stability design to the Simplified Method design, for Extreme Event I (seismic) limit state, pullout, block faced wall with mechanical connection, $R_c = 1.0$ (Example 2).
15-E-7  Stiffness Method Design Example 3: Block Faced Geosynthetic Wall with Mechanical Connections, $R_c = 0.9$

15-E-7.1  General

*Figure 15-E-9* shows a cross-section of the wall for this design example. The procedures and results for the case when the coverage ratio $R_c$ for the reinforcement is less than 1.0 (in this example $R_c$ is set equal to 0.90) are illustrated. Material properties are provided in *Table 15-E-7*, except that the minimum geogrid stiffness available on a stiffness per unit of wall width basis is now reduced using $R_c = 0.90$ to $J Rc = 8.6 \times 0.9 = 7.7$ kips/ft. All other aspects of this example are the same as Example 2.

As is true of Example 2, the scope of Example 3 is limited to the service (soil failure limit only), strength, and Extreme Event I (seismic) limit states. Furthermore, for simplicity, live loads are not included in the calculations to follow. It is also assumed that the foundation soil has sufficient shear strength to prevent global instability, bearing capacity failure and excessive settlement. For seismic design, assume that the ground acceleration, $A_s$, is 0.50g.

*Figure 15-E-9*  Wall geometry and preliminary PET reinforcement layout for Design Example 3
The wall geometry is based on Figure 15-E-9. As is true for Example 2, Example 3 calculations are demonstrated for one of the middle layers (i.e., Layer 6 in Figure 15-E-10, and for the soil failure and pullout limit states, Layer 10 is also used to illustrate calculations).

\[ S = \text{equivalent uniform height of surcharge} = 0 \text{ ft} \]

\[ K_{avh} \text{ and } K_{abh} \text{ remain unchanged relative to Example 1 and Example 2 at } 0.283, \text{ and } \Phi_{fh} = 1.0. \]

The reinforcement stiffness values used in the calculations to follow need to be adjusted to account for the reinforcement coverage ratio, \( R_c \) of 0.90.

Since the soil failure limit state usually controls the amount of reinforcement required, this limit state is evaluated first, and \( T_{\text{max}} \) is calculated, for the wall design.

### 15-E-7.2 Calculations for Soil Failure Limit State Design (Service I)

As is true for Example 2, for Example 3 the factored reinforcement peak strain for each layer should be 2.0% or less for a modular block faced wall (i.e., stiff face) in this example. Some trial-and-error is typically required to establish what reinforcement stiffness values are required, starting with the stiffness values required to meet the soil failure limit, as the soil failure limit often controls the design of geosynthetic walls. Using trial-and-error, use the minimum 1,000-hour 2% strain secant stiffness available for geosynthetic reinforcement products, \( J_i = 7.7 \) kips/ft for the top 4 layers, and increase the stiffness of the bottom 6 layers to 14.5 kips/ft, as shown in Table 15-E-11.

<table>
<thead>
<tr>
<th>Layer No.</th>
<th>Geogrid Designation</th>
<th>( J_i ) (per Unit Width of Reinforcement, in kips/ft)</th>
<th>( R_c \times J_i ) (per Unit Width of Wall, in kips/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>a</td>
<td>8.6</td>
<td>7.7</td>
</tr>
<tr>
<td>9</td>
<td>a</td>
<td>8.6</td>
<td>7.7</td>
</tr>
<tr>
<td>8</td>
<td>a</td>
<td>8.6</td>
<td>7.7</td>
</tr>
<tr>
<td>7</td>
<td>a</td>
<td>8.6</td>
<td>7.7</td>
</tr>
<tr>
<td>6</td>
<td>b</td>
<td>16.1</td>
<td>14.5</td>
</tr>
<tr>
<td>5</td>
<td>b</td>
<td>16.1</td>
<td>14.5</td>
</tr>
<tr>
<td>4</td>
<td>b</td>
<td>16.1</td>
<td>14.5</td>
</tr>
<tr>
<td>3</td>
<td>b</td>
<td>16.1</td>
<td>14.5</td>
</tr>
<tr>
<td>2</td>
<td>b</td>
<td>16.1</td>
<td>14.5</td>
</tr>
<tr>
<td>1</td>
<td>b</td>
<td>16.1</td>
<td>14.5</td>
</tr>
</tbody>
</table>

*This is the stiffness value, corrected for \( R_c \), which is used to calculate \( T_{\text{max}} \).*
The contributing factors, coefficients and parameters that comprise the $T_{\text{max}}$ equation can be found in Allen and Bathurst (2015) and Table 15-E-1. To calculate $T_{\text{max}}$, the reinforcement stiffness must be per unit of wall width rather than per unit of reinforcement product width; hence, $RcJ_i$ must be used to calculate $T_{\text{max}}$ where the reinforcement stiffness value is required. Using the Table 15-E-11 stiffness values, considering a total of 10 layers, the parameters used to determine $T_{\text{max}}$ are calculated as follows:

$$S_{\text{global}} = \frac{J_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} RcJ_i}{H} = (4 \times 0.9 \times 8.6 \text{ kips/ft} + 6 \times 0.9 \times 16.1 \text{ kips/ft})/20 \text{ ft} = 5.89 \text{ ksf}$$

$$\Phi_g = \alpha \left( \frac{S_{\text{global}}}{p_a} \right) = 0.16 \times (5.89 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.209$$

$S_{\text{local}}$ is calculated as:

$$S_{\text{local}} = \left( \frac{RcJ_i}{S_v} \right) = (0.9 \times 16.1 \text{ kips/ft})/(2.0 \text{ ft}) = 7.25 \text{ ksf} \text{ for Layer 6}$$

$$S_{\text{locave}} = \frac{\sum \left( \frac{RcJ_i}{S_v} \right)}{n} = \frac{3.30 + 3 \times 3.85 + 5 \times 7.25 + 8.68}{10} = 5.98 \text{ ksf}$$

where, $n = 10$ is the number of reinforcement layers. Therefore, $\Phi_{\text{local}}$ is recalculated as follows:

$$\Phi_{\text{local}} = \left( \frac{S_{\text{local}}}{S_{\text{locave}}} \right)^{0.5} = \left( \frac{7.25 \text{ ksf}}{5.98 \text{ ksf}} \right)^{0.5} = 1.10 \text{ (Layer 6)}$$

To determine the facing stiffness factor, the facing stiffness parameter, $F_f$ does not change, and the facing stiffness factor is calculated as:

$$\Phi_{fs} = \eta \left( \frac{S_{\text{global}}}{p_a} \right)^{k} = 0.57 \times \left( \frac{5.89 \text{ ksf}}{2.11 \text{ ksf}} \times 1.61 \right)^{0.15} = 0.714$$

Since $c = 0$, the cohesion factor, $\Phi_c = 1.0$.

$D_{\text{max}}$ does not change relative to the previous calculation (i.e., $D_{\text{max}}$ for Layer 6 is 0.825).

$T_{\text{max}}$ for Layer 6 is now recalculated using the updated values as follows:

$$T_{\text{max}} = S_v \left[ H\gamma_rD_{\text{max}} + \left( \frac{H_{ref}}{H} \right) S\gamma_f + LL \right] K_{avh} \Phi_{fb} \Phi_{g} \Phi_{fs} \Phi_{local} \Phi_c$$

$$T_{\text{max}} = 2.0 \text{ ft} \left[ 20 \text{ ft} \times 0.130 \text{ kcf} \times 0.825 + \left( \frac{20 \text{ ft}}{20 \text{ ft}} \right) 0 \text{ ft} \times 0.130 \text{ kcf} + 0 \right] 0.283 \times 1.0 \times 0.209 \times 0.714 \times 1.10 \times 1.0$$

$$T_{\text{max}} = 0.199 \text{ kips/ft of wall width}$$
$T_{\text{max}}$, and the calculated parameters needed to calculate $T_{\text{max}}$, are summarized in Table 15-E-12 for the rest of the layers for the Service and Strength limit states.

Using Equation 15-14 with load factor $\gamma_{sf} = 1.2$, and resistance factor $\phi_{sf} = 1.0$ (Table 15-5), the factored reinforcement strain corresponding to Layer 10 is computed as:

$$\varepsilon_{\text{rein}} = \frac{\gamma_{p-EVsf} T_{\text{max}}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.042 \text{kips/ft}}{1.0 \times 0.9 \times 8.6 \text{kips/ft}} \times 100\% = 0.65\% \leq 2\% \quad \text{OK}$$

For Layer 6:

$$\varepsilon_{\text{rein}} = \frac{\gamma_{p-EVsf} T_{\text{max}}}{\phi_{sf} R_c J_i} = \frac{1.2 \times 0.199 \text{kips/ft}}{1.0 \times 0.9 \times 16.1 \text{kips/ft}} \times 100\% = 1.65\% \leq 2\% \quad \text{OK}$$

See Table 15-E-12 for the calculation results for the rest of the layers. As shown in the table, the soil failure limit state is met by all the reinforcement layers.

As shown for examples 1 and 2, the equivalent ultimate tensile strength that corresponds to the stiffness values needed to meet the soil failure limit state is equal to approximately 0.17$L_i$. For the two stiffness values required to meet the soil failure limit state, the approximate ultimate tensile strength per ft of wall width needed is:

- $T_{\text{ult}} = 0.17 L_i = 0.17 \times (8.6 \text{kips/ft}) = 1.46 \text{kips/ft}$ (applicable to Layer 10)
- $T_{\text{ult}} = 0.17 \times (16.1 \text{kips/ft}) = 2.74 \text{kips/ft}$ (applicable to Layer 6)

To determine $T_{al}$, divide $T_{\text{ult}}$ by $RF = 1.12 \times 1.5 \times 1.3 = 2.18$. Therefore, the approximate long-term tensile strengths implied by the stiffness required for the soil failure limit state for the reinforcement product (i.e., per ft of wall width) are:

- $T_{al} = (1.46 \text{kips/ft})/2.18 = 0.67 \text{kips/ft}$ (applicable to Layer 10)
- $T_{al} = (2.74 \text{kips/ft})/2.18 = 1.26 \text{kips/ft}$ (applicable to Layer 6)

### 15-E-7.3 Calculations for Reinforcement Rupture Limit State (Strength I)

Since no live load is present, the required long-term reinforcement strength ($T_{al}$) for Layer 6 is computed using Equation 15-E-5.

The minimum required value of $T_{al}$ and $T_{\text{ult}}$ for Layer 6, on a strength per unit of reinforcement width basis (i.e., this value is what would be compared to the MARV of the tensile strength of specific reinforcement products), is therefore:

$$T_{al} \geq \frac{\gamma_{p-EV} T_{\text{max}}}{\phi_{rr} R_c} = \frac{1.35 \times 0.199 \text{kips/ft}}{0.80 \times 0.90} = 0.373 \text{kips/ft}$$

$$T_{\text{ult}} = T_{al} R F_{ID} R F_{CR} R F_D = 0.373 \times 1.12 \times 1.5 \times 1.3 = 0.815 \text{kips/ft}$$

For Layer 6, $T_{al}$ for reinforcement rupture is significantly less than $T_{al}$ needed to achieve the stiffness required for the soil failure limit state (i.e., 0.373 kips/ft $<<$ 1.26 kips/ft).
15-E-7.4 Calculations for Connection Strength Design (Strength I)

With regard to connection strength, the same value as was used in Example 2 is used for Example 3 (i.e., a mechanical type connection between the facing blocks and geogrid with a CRu of 0.75). For the connection design, it is assumed that $\gamma_{p-EV_c} = \gamma_{p-EV} = 1.35$, $\phi_{cr} = \phi_{rr} = 0.80$, and $T_0 = T_{max}$. It will also be assumed that this value of CRu is applicable for all geogrids.

Therefore, using the load side of the connection limit state design Equation 15-E-9, at Layer 6, the factored connection load is calculated as follows:

$$T_{ac}(\text{required}) = (\gamma_{p-EV_c})T_0 = (1.35) \times 0.199 \text{kips/ft} = 0.269 \text{kips/ft of wall width}$$

To determine the long-term connection strength available, since a mechanical connection is used in this example, the connection strength will not be a function of the normal load on the facing blocks, N. Using the resistance side of Equation 15-E-9 (i.e., the limit state equation for connection design), and the equation for $T_{ac}$ (Equation 15-E-11), the available long-term connection strength available is calculated as follows, assuming that the minimum $T_{ult}$ needed is equal to the $T_{ult}$ needed (strength per unit of wall width) to obtain the stiffness required to meet the soil failure limit state:

$$T_{ac}(\text{available}) = \phi_{cr} T_{ac} R_c = \frac{\phi_{cr} \times T_{ult} \times (CR_u/RF_{CR}) R_c}{RF_D}$$

$$T_{ac}(\text{available}) = \frac{0.80 \times 2.47 \text{kips/ft} \times (0.75/1.5) 0.9}{1.3} = 0.684 \text{kips/ft}$$

0.269 kips/ft $<$ 0.684 kips/ft? OK

Combining Equation 15-E-9 with Equation 15-E-33 and solving for $T_{ult}$, at layer 6, can determine the minimum $T_{ult}$ required to just satisfy connection strength requirements as follows:

$$T_{ult}(\text{min. required}) = \left(\frac{\gamma_{p-EV_c}}{\phi_{cr} R_c}\right) T_0 RF_D RF_{CR} \left(\frac{1}{CR_u}\right)$$

$$T_{ult}(\text{min. required}) = \left(\frac{1.35}{0.8 \times 0.90}\right) \times 0.199 \times 1.3 \times 1.5 \left(\frac{1}{0.75}\right) = 0.970 \text{kips/ft}$$

The only difference between examples 2 and 3 regarding the connection strength and the $T_{ult}$ needed to meet connection strength requirements is $R_c$ (i.e., 0.873/0.970 = 0.90). These calculations demonstrate that $R_c$ has been handled correctly.

On a strength per unit of reinforcement width basis, this minimum required geosynthetic $T_{ult}$ of 0.970 kips/ft is below the $T_{ult}$ value of 2.74 kips/ft (i.e., a $T_{al} = 2.74/2.18 = 1.26$ kips/ft) estimated to provide the needed stiffness for the soil failure limit state. So the soil failure limit state is still controlling the tensile strength required at this point (i.e., only considering the Service and Strength Limit States).
15-E-7.5 **Calculations for Pullout Limit State Design (Strength I)**

The default pullout parameters $\alpha$ and $F^*$ specified in AASHTO (2020) are used for this example design and are the same as in Example 1 ($\alpha = 0.8$ and $F^* = 0.452$). $R_c$ in this example is smaller than in the previous two examples ($R_c = 0.90$). The vertical stress, $\sigma_v$, over the reinforcement anchorage length, conservatively calculated using Equation 15-E-29, is the same as in Example 1 (0.173 ksf). Since pullout is usually most critical near the wall top, this example pullout calculation is carried out for Layer 10.

Using Equation 15-E-30, the required factored length of reinforcement in the resistant zone (i.e., behind the active wedge) for Layer 10 is:

$$L_e = \frac{\gamma_{p-EV} T_{max}}{\phi \rho C (\alpha F^*) \sigma_v R_c} \times \frac{1.35 \times 0.042 \text{kips}}{0.70 \times 2(0.8 \times 0.452)(0.173 \text{ksf})0.90} = 0.719 \text{ft}$$

To determine the total reinforcement length needed, $L$, the length of reinforcement within the active zone, $L_a$, must be calculated. For geosynthetic walls, the active zone is assumed to be bounded by the Rankine active zone (wedge). $L_a$ is calculated as follows for a vertical wall (at Layer 10):

$$L_a = (H - z) \tan (45^\circ - \phi / 2) = (20 \text{ ft} - 1.33 \text{ ft}) \tan (45^\circ - 34^\circ / 2) = 9.93 \text{ ft}$$

The minimum length allowed for $L_e$ is 3 ft (AASHTO 2020), which is greater than the calculated $L_e$ required for pullout for layer 10. Therefore, using $L_e = 3 \text{ ft}$, the total reinforcement length required for layer 10 is:

$$L = L_a + L_e = 9.93 \text{ ft} + 3 \text{ ft} = 12.9 \text{ ft}$$

Pullout calculation results for the other layers are summarized in Table 15-E-12.

It should be recalled that the minimum length of reinforcement for typical reinforced soil walls is $0.7H = 0.7 \times (20 \text{ ft}) = 14 \text{ ft}$. Therefore, pullout does not control the reinforcement length required. Note that other limit state design calculations can result in greater reinforcement length, such as external stability (e.g., sliding) or global stability.

15-E-7.6 **Calculations for Determination of $T_{max} + T_{md}$ (Extreme Event I - Seismic)**

$T_{md}$ is calculated as shown for Example 2, Layer 6, and is equal to 0.424 kips/ft. $T_{totalf}$ is also the same as shown for Example 2 and is equal to 0.623 kips/ft of wall width.
15-E-7.7 Calculations for Reinforcement Rupture (Extreme Event I - Seismic)

Using equations 15-E-21, 15-E-22, and 15-E-23, calculate \( T_{ult} \) for static portion of load at Layer 6:

\[
S_{rs} = \frac{Y_{seis} T_{max} RF}{\phi R_c} = \frac{1.0 \times 0.199 \frac{\text{kips}}{\text{ft}} \times 2.18}{1.0 \times 0.90} = 0.482 \frac{\text{kips}}{\text{ft}}
\]

\( T_{ult} \) for dynamic portion of load at Layer 6:

\[
S_{rt} = \frac{Y_{seis} T_{md} R F_{1D} R F_{D}}{\phi R_c} = \frac{1.0 \times 0.424 \frac{\text{kips}}{\text{ft}} \times 1.12 \times 1.3}{1.0 \times 0.90} = 0.686 \frac{\text{kips}}{\text{ft}}
\]

\( T_{ult} = S_{rs} + S_{rt} = 0.482 \text{kips/ft} + 0.686 \text{kips/ft} = 1.17 \text{kips/ft of reinforcement width} \)

\( T_{al} = 1.17 \text{kips/ft/2.18} = 0.537 \text{kips/ft of reinforcement width} \)

The only difference between these calculated values and those calculated for Example 2 is the coverage ratio of 0.90 (i.e., for \( T_{al} \), 0.434/0.537 = 0.90).

15-E-7.8 Calculations for Connection Rupture (Extreme Event I - Seismic)

Using equations 15-E-24, 15-E-25, and 15-E-26, \( T_{ult} \) for static portion of load at Layer 6:

\[
C_{R_c} = \frac{C_{R_u} R F_{CR}}{RF_{CR}} = \frac{0.75 \frac{\text{kips}}{\text{ft}}}{1.5 \frac{\text{kips}}{\text{ft}}} = 0.500
\]

Since this is a mechanical connection, \( F_r \) is set equal to 1.0.

\[
S_{rsc} = \frac{Y_{seis} T_0 R F_{D}}{F_r \phi C_{R_c} R_c} = \frac{1.0 \times 0.199 \frac{\text{kips}}{\text{ft}} \times 1.3}{1.0 \times 1.0 \times 0.500 \times 0.90} = 0.575 \frac{\text{kips}}{\text{ft}}
\]

\[
S_{rtc} = \frac{Y_{seis} T_{md} R F_{D}}{F_r \phi C_{R_u} R_c} = \frac{1.0 \times 0.424 \frac{\text{kips}}{\text{ft}} \times 1.3}{1.0 \times 1.0 \times (0.75) \times 0.90} = 0.817 \frac{\text{kips}}{\text{ft}}
\]

\( T_{ult} = S_{rsc} + S_{rtc} = 0.575 \text{kips/ft} + 0.817 \text{kips/ft} = 1.39 \text{kips/ft of reinforcement product width} \)

On a strength per unit of reinforcement width basis, this minimum required geosynthetic \( T_{ult} \) of 1.39 kips/ft is still below the \( T_{ult} \) value of 2.74 kips/ft (i.e., a \( T_{al} = 2.74/2.18 = 1.26 \) kips/ft) estimated to provide the needed stiffness for the soil failure limit state. So the soil failure limit state is still controlling the tensile strength required considering the Extreme Event I Limit State (i.e., seismic).

Again, the only difference between these calculated results and those determined for Example 2 is the coverage ratio of 0.90.
15-E-7.9  Calculations for Pullout Limit State Design  
(Extreme Event I - Seismic)

Using Equation 15-E-27, $L_e$ for Layer 10 (i.e., at the wall top) is determined as follows:

$$\sigma_v = z\gamma_r + S_{sur}\gamma_f$$

$$\sigma_v = (1.33 \text{ ft})(0.13 \text{ kcf}) + (0)(0.13 \text{ kcf}) = 0.173 \text{ ksf}$$

$$L_e = \frac{\gamma_{seis}(T_{max} + T_{mad})}{\phi C(0.8\alpha F^*)\sigma_v R_c} = \frac{1.0(0.042 + 0.424)}{1.0 \times 2 \times (0.8 \times 0.8 \times 0.452) \times 0.173 \text{ kcf} \times 0.90}$$

$$L_e = 5.18 \text{ ft}$$

$$L_a = (H - z) \tan (45^\circ - \phi_r/2) = (20 \text{ ft} - 1.33 \text{ ft}) \tan (45^\circ - 34^\circ/2) = 9.93 \text{ ft}$$

$$L = L_a + L_e = 9.93 \text{ ft} + 5.18 \text{ ft} = 15.1 \text{ ft}$$

This required length is greater than 70% of the wall height, so it does control pullout length at the wall top.

15-E-7.10  Calculations to Check to Make Sure Stiffness and Strength Required are Properly Matched  
(Extreme Event I, Seismic)

Since the soil failure limit state is still controlling the design, the stiffness determined to meet the soil failure limit state is still the correct stiffness to use. Therefore, an additional iteration with higher stiffness values that are consistent with the tensile strengths needed is not required. Had one of the other limit states controlled the $T_{ult}$ needed, then it would have been necessary to recheck the design using a stiffness value that is consistent with the higher tensile strength. Fortunately, this does not happen very often (only would occur for block faced walls with very inefficient connections between the geosynthetic and the facing blocks).

15-E-7.11  Summary for Example 3 Design

See Table 15-E-12 for the calculation results for all of the layers for the Service I, Strength I, and Extreme Event I limit states. These calculation results are plotted and compared to design calculation results using the Simplified Method in figures 15-E-10 through 15-E-12.

In summary, for the final internal stability design for Example 3 using the Stiffness Method, the following would be specified with regard to reinforcement properties and spacing for the optimized design, and the following observations can be made:

- The vertical spacing of reinforcements is 2 ft throughout the wall height, and the coverage ratio is a minimum of 0.90.
- The minimum reinforcement length for internal stability (i.e., pullout) is 14 ft for the Strength Limit, and all layers except the top layer for seismic did not exceed this minimum length. The top layer length required is 15.1 ft for seismic for the Stiffness Method, which is greater than the 0.7H minimum. The longer reinforcement length due to pullout is consistent with good detailing for MSE walls in AASHTO (2020) LRFD Article 11.10.7.4.
• The lowest strength PET geogrid available was greater in strength and stiffness than required by the Stiffness Method design for the top 4 layers (see figures 15-E-11 and 15-E-12). This was not the case for the Simplified Method, as higher reinforcement strengths than the minimum available were required for most of the layers.

• The Simplified Method required a total long-term tensile strength $T_{al}$ of 13.5 kips/ft for the entire wall section (seismic connection rupture controlled the design), whereas the Stiffness Method required only 5.99 kips/ft for the entire wall section for seismic reinforcement connection rupture. However, the $T_{al}$ needed to obtain the stiffness needed to meet the soil failure limit state controlled the design, for which the total $T_{al}$ for the wall section was 10.2 kips/ft, which is just over 75% of the total tensile strength needed by the Simplified Method. Note that in the upper third of the wall, all limit states for both methods will be limited to the minimum strength shown in the plots as a dashed vertical line. If that is considered, the Simplified Method $T_{al}$ required would increase to 13.7 kips/ft, making the Stiffness Method required soil reinforcement strength equal to 74% of what is required by the Simplified Method.

• Overall, the required strength and stiffness of the reinforcement needed is in the low to mid-range of the geogrid product lines available. Note that the ground acceleration used for the seismic design represents what would be included in Seismic Zone 4, which is the highest seismic zone.
### Table 15-E-12

Summary of Example 3 wall design calculations using Stiffness Method and $R_c = 0.90$ (Service, Strength, and Extreme Event I Limit States): (a) Calculation of $T_{max}$, (b) Service and Strength Limit State calculations, and (c) Extreme Event I (seismic) Limit State calculations.

#### a) $T_{max}$ Equation (Eq. 15-E-1) Parameters

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>$z$ (ft)</th>
<th>$S$ (ft)</th>
<th>$R_{J_i}$ (kips/ft)</th>
<th>$S_{global}$ (ksf)</th>
<th>$S_{local}$ (ksf)</th>
<th>$D_{max}$</th>
<th>$F_t$</th>
<th>$\Phi_e$</th>
<th>$\Phi_{local}$</th>
<th>$\Phi_b$</th>
<th>$\Phi_n$</th>
<th>$T_{max}$ and $T_0$ (kips/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 (top)</td>
<td>1.33</td>
<td>2.33</td>
<td>7.7</td>
<td>5.89</td>
<td>3.30</td>
<td>0.220</td>
<td>1.61</td>
<td>0.209</td>
<td>0.74</td>
<td>1.0</td>
<td>0.714</td>
<td>0.042</td>
</tr>
<tr>
<td>9</td>
<td>3.33</td>
<td>2.00</td>
<td>7.7</td>
<td>5.89</td>
<td>3.85</td>
<td>0.372</td>
<td>1.61</td>
<td>0.209</td>
<td>0.80</td>
<td>1.0</td>
<td>0.714</td>
<td>0.065</td>
</tr>
<tr>
<td>8</td>
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<td>2.00</td>
<td>7.7</td>
<td>5.89</td>
<td>3.85</td>
<td>0.523</td>
<td>1.61</td>
<td>0.209</td>
<td>0.80</td>
<td>1.0</td>
<td>0.714</td>
<td>0.092</td>
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<tr>
<td>7</td>
<td>7.33</td>
<td>2.00</td>
<td>7.7</td>
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<td>0.674</td>
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<td>0.825</td>
<td>1.61</td>
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<td>1.10</td>
<td>1.0</td>
<td>0.714</td>
<td>0.199</td>
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<tr>
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<td>5.89</td>
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<td>0.976</td>
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<td>0.714</td>
<td>0.236</td>
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<td>5.89</td>
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<td>1.00</td>
<td>1.61</td>
<td>0.209</td>
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<td>1.0</td>
<td>0.714</td>
<td>0.242</td>
</tr>
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<td>0.209</td>
<td>1.10</td>
<td>1.0</td>
<td>0.714</td>
<td>0.242</td>
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<td>2.00</td>
<td>14.5</td>
<td>5.89</td>
<td>7.25</td>
<td>1.00</td>
<td>1.61</td>
<td>0.209</td>
<td>1.10</td>
<td>1.0</td>
<td>0.714</td>
<td>0.242</td>
</tr>
<tr>
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<td>5.89</td>
<td>8.68</td>
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<td>1.61</td>
<td>0.209</td>
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<td>1.0</td>
<td>0.714</td>
<td>0.221</td>
</tr>
<tr>
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<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\sum T_{max} = 1.70$</td>
</tr>
</tbody>
</table>

#### b) Reinforcement Rupture (Strength Limit)

- **Minimum Required Strength per Unit Width of Reinforcement**
  - $T_{el}$ (kips/ft)
  - $T_{ul}$ (kips/ft)

- **Connection Rupture (Strength Limit)**
  - $J_1$ (kips/ft)
  - $J_2$ (kips/ft)

- **Soil Failure (Service Limit)**
  - Factor $\varepsilon_{res}$ (%)
  - Factor $\varepsilon_{res}$ Corresponding to $J_1$ (kips/ft)

- **Pullout (Strength Limit)**
  - Anchorage length $L_a$ (ft)
  - Required
  - Min. allowed

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>$z$ (ft)</th>
<th>Reinforcement Product</th>
<th>$T_{el}$ (kips/ft)</th>
<th>$T_{ul}$ (kips/ft)</th>
<th>$T_{el}$ (kips/ft)</th>
<th>$T_{ul}$ (kips/ft)</th>
<th>$\varepsilon_{res}$ (%)</th>
<th>$\varepsilon_{res}$ Corresponding to $J_1$ (kips/ft)</th>
<th>Anchorage length $L_a$ (ft)</th>
<th>Required</th>
<th>Min. allowed</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 (top)</td>
<td>1.33</td>
<td>Geogrid a</td>
<td>0.079</td>
<td>0.17</td>
<td>0.75</td>
<td>0.09</td>
<td>0.20</td>
<td>0.65</td>
<td>0.67</td>
<td>0.72</td>
<td>&lt; 3.0 (OK)</td>
</tr>
<tr>
<td>9</td>
<td>3.33</td>
<td>Geogrid a</td>
<td>0.12</td>
<td>0.27</td>
<td>0.75</td>
<td>0.15</td>
<td>0.32</td>
<td>1.02</td>
<td>0.67</td>
<td>0.45</td>
<td>&lt; 3.0 (OK)</td>
</tr>
<tr>
<td>8</td>
<td>5.33</td>
<td>Geogrid a</td>
<td>0.17</td>
<td>0.38</td>
<td>0.75</td>
<td>0.21</td>
<td>0.45</td>
<td>1.43</td>
<td>0.67</td>
<td>0.39</td>
<td>&lt; 3.0 (OK)</td>
</tr>
<tr>
<td>7</td>
<td>7.33</td>
<td>Geogrid a</td>
<td>0.22</td>
<td>0.49</td>
<td>0.75</td>
<td>0.26</td>
<td>0.58</td>
<td>1.85</td>
<td>0.67</td>
<td>0.37</td>
<td>&lt; 3.0 (OK)</td>
</tr>
<tr>
<td>6</td>
<td>9.33</td>
<td>Geogrid b</td>
<td>0.37</td>
<td>0.82</td>
<td>0.75</td>
<td>0.44</td>
<td>0.97</td>
<td>1.65</td>
<td>1.26</td>
<td>0.49</td>
<td>&lt; 3.0 (OK)</td>
</tr>
<tr>
<td>5</td>
<td>11.33</td>
<td>Geogrid b</td>
<td>0.44</td>
<td>0.97</td>
<td>0.75</td>
<td>0.53</td>
<td>1.15</td>
<td>1.95</td>
<td>1.26</td>
<td>0.47</td>
<td>&lt; 3.0 (OK)</td>
</tr>
<tr>
<td>4</td>
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<td>Geogrid b</td>
<td>0.45</td>
<td>0.99</td>
<td>0.75</td>
<td>0.54</td>
<td>1.18</td>
<td>2.00</td>
<td>1.26</td>
<td>0.41</td>
<td>&lt; 3.0 (OK)</td>
</tr>
<tr>
<td>3</td>
<td>15.33</td>
<td>Geogrid b</td>
<td>0.45</td>
<td>0.99</td>
<td>0.75</td>
<td>0.54</td>
<td>1.18</td>
<td>2.00</td>
<td>1.26</td>
<td>0.36</td>
<td>&lt; 3.0 (OK)</td>
</tr>
<tr>
<td>2</td>
<td>17.33</td>
<td>Geogrid b</td>
<td>0.45</td>
<td>0.99</td>
<td>0.75</td>
<td>0.54</td>
<td>1.18</td>
<td>2.00</td>
<td>1.26</td>
<td>0.32</td>
<td>&lt; 3.0 (OK)</td>
</tr>
<tr>
<td>1</td>
<td>19.33</td>
<td>Geogrid b</td>
<td>0.41</td>
<td>0.90</td>
<td>0.75</td>
<td>0.49</td>
<td>1.08</td>
<td>1.83</td>
<td>1.26</td>
<td>0.26</td>
<td>&lt; 3.0 (OK)</td>
</tr>
<tr>
<td>Base of wall</td>
<td>20</td>
<td></td>
<td>$\sum T_{el} = 3.18$</td>
<td>$\sum T_{ul} = 6.96$</td>
<td>$\sum T_{el} = 3.79$</td>
<td>$\sum T_{ul} = 8.28$</td>
<td>$\sum T_{el} = 10.2$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 15-E-12, continued

Summary of Example 3 wall design calculations using Stiffness Method and $R_c = 0.90$ (Service, Strength, and Extreme Event I Limit States): (c) Extreme Event I (seismic) Limit State calculations.

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>z (ft)</th>
<th>Reinforcement Product</th>
<th>Soil Failure (Service Limit, for Comparison)</th>
<th>Reinforcement Rupture (Extreme Event I Limit)</th>
<th>Connection Rupture (Extreme Event I Limit)</th>
<th>Pullout (Extreme Event I Limit)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$^*_{\text{Tensile Strength \ } \mathbf{T}_m \ \text{corresponding \ to \ Reinforcement \ Stiffness \ Required}} \ \text{(kips/ft)}$</td>
<td>$^*_{\text{Minimum Required Strength per Unit Width of Reinforcement}} \ \text{(kips/ft)}$</td>
<td>$^*_{\text{Minimum Required Strength per Unit Width of Reinforcement}} \ \text{(kips/ft)}$</td>
<td>Anchorage length $L_{\text{min}}$ (ft)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\mathbf{T}_m$</td>
<td>$\mathbf{T}_m$</td>
<td>$\mathbf{T}_m$</td>
<td>$\mathbf{T}_m$</td>
</tr>
<tr>
<td>10 (top)</td>
<td>1.33</td>
<td>Geogrid a</td>
<td>0.67</td>
<td>0.36</td>
<td>0.79</td>
<td>0.75</td>
</tr>
<tr>
<td>9</td>
<td>3.33</td>
<td>Geogrid a</td>
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<td>0.39</td>
<td>0.85</td>
<td>0.75</td>
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<td>0.75</td>
</tr>
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<td>Geogrid a</td>
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<td>0.54</td>
<td>1.17</td>
<td>0.75</td>
</tr>
<tr>
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<td>Geogrid b</td>
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<td>0.58</td>
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<td>4</td>
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<td>0.58</td>
<td>1.27</td>
<td>0.75</td>
</tr>
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</tr>
<tr>
<td>Base of wall</td>
<td>20</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- These values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (AASHTO 2020).
- For comparison to geogrid product MARV tensile strength that is not reduced by $R_c$ (i.e., load per unit of reinforcement width basis).
- \(T_{\text{md}}\) for all reinforcement layers is 0.424 kips/ft.
Figure 15-E-10  Comparison of Stiffness Method internal stability design to the Simplified Method design, for Service and Strength limit states, block faced wall with mechanical connection, $R_c = 0.90$ (Example 3).

Figure 15-E-11  Comparison of Stiffness Method internal stability design to the Simplified Method design, for Service and Extreme Event I (seismic) limit states, block faced wall with mechanical connection, $R_c = 0.90$ (Example 3).
Figure 15-E-12  Comparison of Stiffness Method internal stability design to the Simplified Method design, for Extreme Event I (seismic) limit state, pullout, block faced wall with mechanical connection, $R_c = 0.90$ (Example 3).
15-E-8 Stiffness Method Design Example 4: Block Faced Geosynthetic Wall System with Frictional Facing Connection

15-E-8.1 General

Figure 15-E-9 shows a cross-section of the wall for this design example. Material properties are provided in Table 15-E-7, except that the minimum geogrid stiffness available on a stiffness per unit of wall width basis is reduced using $R_c = 0.90$ (i.e., same as Example 3). All other aspects of this example are the same as Example 2. Assume for this example that polyester (PET) geogrid will be used for the soil reinforcement, and dry cast facing blocks with a frictional connection between the geogrid and facing blocks are used. Because of the need to assess connection strength, and because connection strength is geosynthetic and facing block specific (i.e., wall system specific), example system specific ultimate connection strength data ($T_{ultconn}$), and properties for the geosynthetic used with the wall system, are provided in Figure 15-E-13 and Table 15-E-13. The wall backfill is assumed to be a well-graded gravelly sand, with no cohesion.

Figure 15-E-13 Block-geogrid connection test results for Design Example 4, in units of peak tensile capacity (i.e., $T_{ultconn}$) per unit of reinforcement width.
Table 15-E-13  Connection strength equations for example wall system.

<table>
<thead>
<tr>
<th>Geogrid Product</th>
<th>Approx. Wall Height, H (ft)</th>
<th>Normal Load, N (lbs/ft)</th>
<th>$T_{\text{ult conn}}$ (lbs/ft)</th>
<th>$T_{\text{tot}}$ (lbs/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geogrid A</td>
<td>H &lt; 9</td>
<td>N &lt; 1344</td>
<td>976 + N tan 42°</td>
<td>3484</td>
</tr>
<tr>
<td></td>
<td>H &lt; 20</td>
<td>1344 &lt; N &lt; 2268</td>
<td>1989 + N tan 8.1°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 20</td>
<td>N &gt; 2268</td>
<td>2416</td>
<td></td>
</tr>
<tr>
<td>Geogrid B</td>
<td>H &lt; 16</td>
<td>N &lt; 1724</td>
<td>1305 + N tan 36°</td>
<td>4927</td>
</tr>
<tr>
<td></td>
<td>H &lt; 30</td>
<td>1724 &lt; N &lt; 3424</td>
<td>2045 + N tan 16°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 30</td>
<td>N &gt; 3424</td>
<td>3030</td>
<td></td>
</tr>
<tr>
<td>Geogrid C</td>
<td>H &lt; 16</td>
<td>N &lt; 1681</td>
<td>1221 + N tan 37°</td>
<td>6109</td>
</tr>
<tr>
<td></td>
<td>H &lt; 30</td>
<td>1681 &lt; N &lt; 3479</td>
<td>1642 + N tan 26°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 30</td>
<td>N &gt; 3479</td>
<td>3339</td>
<td></td>
</tr>
<tr>
<td>Geogrid D</td>
<td>H &lt; 16</td>
<td>N &lt; 1695</td>
<td>1146 + N tan 42°</td>
<td>7897</td>
</tr>
<tr>
<td></td>
<td>H &lt; 30</td>
<td>1695 &lt; N &lt; 3380</td>
<td>1657 + N tan 31°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 30</td>
<td>N &gt; 3380</td>
<td>3688</td>
<td></td>
</tr>
<tr>
<td>Geogrid E</td>
<td>H &lt; 16</td>
<td>N &lt; 1695</td>
<td>1094 + N tan 45°</td>
<td>10795</td>
</tr>
<tr>
<td></td>
<td>H &gt; 16</td>
<td>1695 &lt; N &lt; 3373</td>
<td>1640 + N tan 33°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 30</td>
<td>3373</td>
<td>3830</td>
<td></td>
</tr>
</tbody>
</table>

Data for geogrids B through E are faded in the figure and table since, as will be shown later, for the Stiffness Method, only the weakest geogrid will be needed. However, for the comparison to the Simplified Method provided at the end of this example, the other geogrids will be needed. Showing the other geogrids is also useful to demonstrate how the Geogrid A connection strength plot compares with the stronger geogrids. The pattern shown in Figure 15-E-13 is typical of modular block wall system connection strength data in which the connections are mostly frictional. Only when the normal stress between blocks gets high enough do significant connection strength differences between geogrids with a range of tensile strengths occur, transitioning from mostly friction controlled to reinforcement rupture controlled. Because of this, at facing block normal loads that are relatively low, increasing the geogrid tensile strength may not help much, and the only choice may be to reduce the reinforcement spacing. As is shown later, this will not be an issue for the Stiffness Method, but this will be an issue for the Simplified Method. Another approach to address this problem is to conduct 1,000 hour connection creep tests, as for frictional systems, it is likely that a lower reduction factor for creep, $RF_{\text{CR}}$, could be used instead of the $RF_{\text{CR}}$ determined for the geogrid in isolation (in this case, $RF_{\text{CR}}$ for the geogrid is 1.5, but for the connection, a $RF_{\text{CR}}$ of only 1.2 or lower could be used, as shown in Figure 15-E-14). However, for this example, to keep the example as simple as possible, the $RF_{\text{CR}}$ for the geogrid of 1.5 is used (i.e., the data in Figure 15-E-5 is not used in this example, but is for information only).
As is true of examples 2 and 3, the scope of this design example is limited to the service (soil failure limit only), strength, and Extreme Event I (seismic) limit states. Furthermore, for simplicity, live loads are not included in the calculations to follow. It is also assumed that the foundation soil has sufficient shear strength to prevent global instability, bearing capacity failure and excessive settlement. For seismic design, assume that the ground acceleration, \( A_s \), is 0.50g.

The wall geometry is based on Figure 15-E-9 (i.e., same as for Example 3). As is true for examples 2 and 3, Example 4 calculations are demonstrated for one of the middle layers (i.e., Layer 6 in Figure 15-E-10, and for the soil failure and pullout limit states, Layer 10 is also used to illustrate calculations).

**Figure 15-E-14**  Example block-geogrid creep connection test results for a wall system, in units of peak tensile capacity per unit of reinforcement width.

\[
S = \text{equivalent uniform height of surcharge} = 0 \text{ ft}
\]

\( K_{avh} \) and \( K_{abh} \) remain unchanged relative to Example 1 and Example 2 at 0.283, and \( \Phi_b = 1.0 \).

The reinforcement stiffness values used in the calculations to follow, the geogrid stiffness needs to be adjusted to account for the reinforcement coverage ratio, \( R_c \) of 0.90.

Since the soil failure limit state usually controls the amount of reinforcement required, this limit state is evaluated first, and \( T_{max} \) is calculated, for the wall design.
15-E-8.2 Calculations for Soil Failure Limit State Design (Service I)

As is true for examples 2 and 3, for Example 4 the factored reinforcement peak strain for each layer should be 2.0% or less for a modular block faced wall (i.e., stiff face) in this example. Some trial-and-error is typically required to establish what reinforcement stiffness values are required, starting with the stiffness values required to meet the soil failure limit, as the soil failure limit often controls the design of geosynthetic walls. However, since this is a specific hypothetical wall system, the weakest geogrid available for the wall system should be used as the starting point to check the soil failure limit state. Therefore, begin by calculating $T_{\text{max}}$ using the minimum stiffness reinforcement product available for the wall system, which is Geogrid A in Figure 15-E-13. The creep stiffness, $J_i$, of Geogrid A is 19.2 kips/ft (per unit of reinforcement product width), and its tensile strength $T_{\text{MARV}}$ is 3.50 kips/ft and $T_{\text{ai}}$ is 1.61 kips/ft (also per unit of reinforcement product width).

The contributing factors, coefficients and parameters that comprise the $T_{\text{max}}$ equation can be found in Allen and Bathurst (2015) and Table 15-E-1. To calculate $T_{\text{max}}$, the reinforcement stiffness must be per unit of wall width rather than per unit of reinforcement product width; hence, $R_c J_i$ must be used where the reinforcement stiffness value is required. Therefore, considering a total of 10 layers, the parameters used to determine $T_{\text{max}}$ are calculated as follows:

\[
S_{\text{global}} = \frac{f_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} R_c J_i}{H} = (10 \times 0.90 \times 19.2 \text{ kips/ft})/20 \text{ ft} = 8.65 \text{ ksf}
\]

\[
\Phi_g = \alpha \left( \frac{S_{\text{global}}}{p_a} \right)^\beta = 0.16 \times (8.65 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.231
\]

\[
\Phi_f = \left( \frac{K_{\text{ab}}}{K_{\text{av}}^\text{f}} \right)^d = (0.283/0.283)^0.4 = 1.0
\]

\[
S_{\text{local}} = \left( \frac{R_c J_i}{S_v} \right)_i = (0.90 \times 19.2 \text{ kips/ft})/(2.0 \text{ ft}) = 8.65 \text{ ksf for Layer 6}
\]

\[
S_{\text{local ave}} = \frac{\sum \left( \frac{R_c J_i}{S_v} \right)_i}{n} = \frac{7.42 + 8 \times 8.65 + 10.4}{10} = 8.70 \text{ ksf}
\]

where, $n = 10$ is the number of reinforcement layers. Therefore, $\Phi_{\text{local}}$ is calculated as follows:

\[
\Phi_{\text{local}} = \left( \frac{S_{\text{local}}}{S_{\text{local ave}}} \right)^{0.5} = \left( \frac{8.70 \text{ ksf}}{8.70 \text{ ksf}} \right)^{0.5} = 1.00 \quad \text{(Layer 6)}
\]

To determine the facing stiffness factor, the facing stiffness parameter, $F_f$ does not change, and the facing stiffness factor is calculated as:

\[
\Phi_{FS} = \eta \left( \frac{S_{\text{global}}}{p_a} \right) F_f^\kappa = 0.57 \times \left( \frac{8.65 \text{ ksf}}{2.11 \text{ ksf}} \right) \times 1.61^{0.15} = 0.757
\]
Since $c = 0$, the cohesion factor, $\Phi_c = 1.0$.

$D_{t_{\text{max}}}$ does not change relative to the previous calculation (i.e., $D_{t_{\text{max}}}$ for Layer 6 is 0.825).

$T_{\text{max}}$ for Layer 6 is calculated as follows:

$$T_{\text{max}} = S_v \left[ H_{yr} D_{t_{\text{max}}} + \left( \frac{H_{ref}}{H} \right) S_{yr} + LL \right] \kappa_{avh} \Phi_{fb} \Phi_g \Phi_{fs} \Phi_{local} \Phi_c$$

$$T_{\text{max}} = 2.0 \text{ ft} \left[ 20 \text{ ft} \times 0.130 \text{ kcf} \times 0.825 + \left( \frac{20 \text{ ft}}{20 \text{ ft}} \right) 0 \text{ ft} \times 0.130 \text{ kcf} + 0 \right] 0.283 \times 1.0 \times 0.231 \times 0.757 \times 1.10 \times 1.0$$

$$T_{\text{max}} = 0.211 \text{ kips/ft of wall width}$$

$T_{\text{max}}$, and the calculated parameters needed to calculate $T_{\text{max}}$, are summarized in Table 15-E-14 for the rest of the layers.

Using Equation 15-14 with load factor $\gamma_{sf} = 1.2$, and resistance factor $\phi_{sf} = 1.0$ (Table 15-5), the factored reinforcement strain corresponding to Layer 10 is computed as:

$$\varepsilon_{\text{rein}} = \frac{\gamma_{p-EVSf} T_{\text{max}}}{\phi_{sf} R_{ci}} = \frac{1.2 \times 0.061 \text{ kips/ft}}{1.0 \times 0.90 \times 19.2 \text{ kips/ft}} \times 100\% = 0.42\% \leq 2\% \text{ OK}$$

For Layer 6:

$$\varepsilon_{\text{rein}} = \frac{\gamma_{p-EVSf} T_{\text{max}}}{\phi_{sf} R_{ci}} = \frac{1.2 \times 0.211 \text{ kips/ft}}{1.0 \times 0.90 \times 19.2 \text{ kips/ft}} \times 100\% = 1.47\% \leq 2\% \text{ OK}$$

See Table 15-E-14 for the calculation results for the rest of the layers for the soil failure limit state for Example 4. Note that the calculated factored strains are significantly less than the target maximum strain of 2.0%. This means that if a weaker geogrid reinforcement was available for this wall system, a weaker product could have been used. Alternatively, the coverage ratio $R_c$ could have been reduced further (i.e., to less than 0.90).
15-E-8.3 Calculations for Reinforcement Rupture Limit State (Strength I)

Since no live load is present, the required long-term reinforcement strength ($T_{al}$) for Layer 6 is computed using Equation 15-E-5.

The minimum required value of $T_{al}$ and $T_{ult}$ for Layer 6 is therefore:

$$T_{al} \geq \frac{\gamma_p EV}{\phi_{rr} R_c} = \frac{1.35 \times 0.211 \text{ kips}}{0.80 \times 0.90} = 0.398 \text{ kips/ft}$$

$$T_{ult} = T_{al}RF_IDRF_CRRF_D = 0.398 \times 1.12 \times 1.5 \times 1.3 = 0.868 \text{ kips/ft}$$

For Layer 6, the calculated $T_{al}$ for reinforcement rupture, using only a stiffness that is consistent with available wall system specific reinforcement products, is significantly less than $T_{al}$ needed to achieve the stiffness required for the soil failure limit state (i.e., $0.398 \text{ kips/ft} << 1.26 \text{ kips/ft}$) and significantly less that the $T_{al}$ for the weakest geogrid product available for the wall system (i.e., $0.398 \text{ kips/ft} << 1.61 \text{ kips/ft}$). Therefore, to meet the Strength Limit State, reinforcement rupture, Geogrid A can be used. See Table 15-E-14 for the calculation results for the rest of the wall layers.

15-E-8.4 Calculations for Connection Strength Design (Strength I)

To determine the minimum tensile strength needed at the connection to the facing, connection strength data for the facing block – geosynthetic combinations anticipated are needed. It has been assumed for this example that a frictional type connection between the facing blocks and geogrid will be used. Short-term connection test results for the geogrids under consideration in this example are shown in Figure 15-E-13 and Table 15-E-13. $T_{lot}$ values for the connection tests are also summarized in the figure.

Equation 15-E-9 is used to calculate the minimum long-term connection strength needed, $T_{ac}\text{(required)}$. For the purposes of this example and to be consistent with the approach taken in the current AASHTO (2020) specifications, the load and resistance factors for the connection rupture limit state are assigned the same values as those used for geosynthetic rupture limit state at locations away from the connection (i.e., $\gamma_p EV_c = \gamma_p EV = 1.35$ and $\phi_{cr} = \phi_{rr} = 0.80$), and $T_0 = T_{max}$.

Therefore, using Equation 15-E-9, at Layer 6, the factored connection load is:

$$T_{ac\text{(required)}} = (\gamma_p EV_c)T_0 = (1.35) \times 0.211 \frac{\text{kips}}{\text{ft}} = 0.285 \frac{\text{kips}}{\text{ft}} \text{ of wall width}$$

To determine the long-term connection strength available, since a dominantly frictional connection is used in this example, the connection strength will be a function of the normal load on the facing blocks, N, which is affected by the depth of the connection below the wall top if the facing is vertical (i.e., no facing batter), but is limited by the hinge height (see AASHTO 2020 LRFD Bridge Design Manual, Art. 11.10.6.4.4b) if the facing is battered. For this example, the facing is assumed to have no batter (i.e., is vertical).
To determine $T_{ult,conn}$, the normal force, $N$, on the facing blocks, in units of load per unit of wall width, must be determined at each layer depth. The following equation can be used for this purpose for a vertical wall:

$$N = \gamma_{block} \times z \times W_u$$  \hspace{1cm} (15-E-34)

where,

$\gamma_{block} = \text{average unit weight of block plus any soil placed in block hollow areas, if any are present (kcf)}$

$z = \text{depth below wall top at face to reinforcement layer (for battered walls, use the hinge height as a limit) (ft)}$

$W_u = \text{facing block width (face to back of block) (ft)}$

Using the relationships presented in Figure 15-E-13 and Table 15-E-13, $T_{ult,conn}$ is calculated as follows for Layer 6, using the connection test results for Geogrid A (i.e., this geogrid is the minimum strength geogrid available for the specific wall system that meets or exceeds soil failure limit state requirements):

$$N = 0.12 \text{kcf} \times 9.33 \text{ft} \times 1.0 \text{ft} = 1.12 \text{kips/ft}$$

Therefore,

$$T_{ult,conn} = 0.976 + 1.12 \times \tan 42^\circ = 1.98 \text{kips/ft}$$

Expressed as a portion of $T_{lot}$, the short-term connection strength $CR_u$ is $(1.98 \text{kips/ft})/(3.484 \text{kips/ft}) = 0.568$.

Combining equations 15-E-11 and 15-E-12, the available, factored long-term connection strength is calculated as follows:

$$T_{ac(available)} = \frac{\phi CR \times T_{ult,conn} \times \frac{T_{ult,conn}}{RF_{CR} \times T_{lot}} \times R_c}{RF_D}$$  \hspace{1cm} (15-E-35)

For Layer 6,

$$T_{ac(available)} = \frac{0.8 \times 3.50 \text{kips/ft} \times \frac{1.98 \text{kips/ft}}{1.5 \times 3.484 \text{kips/ft}} (0.9)}{1.3} = 0.734 \text{kips/ft}$$

Note that $T_{ult}$ in this equation is a Minimum Average Roll Value (MARV), whereas $T_{lot}$ is the tensile strength of the geogrid used for the connection testing.

At Layer 6,

$$T_{ac \text{ (available)}} > T_{ac \text{ (required)}} \text{ (i.e., } 0.734 \text{kips/ft} >> 0.285 \text{kips/ft)}.$$  

Therefore, connection strength does not control the design.

Focusing instead on the minimum geogrid tensile strength needed to safely meet the demand at the connection, determine $T_{ult}$ as follows:

$$T_{ult \text{ (min. required)}} = \left(\frac{\gamma_{p-EV} C}{\phi CR} \right) T_0 RF_D RF_{CR} \left(\frac{T_{lot}}{T_{ult,conn}}\right)$$  \hspace{1cm} (15-E-36)
Note that \( T_{\text{lot}}/T_{\text{ultconn}} \), which essentially is \( 1/CR_u \), will be specific to the geosynthetic reinforcement and block/connector system used, in addition to being a function of the normal force between blocks at the connection, \( N \). This equation can be used to estimate the ultimate geogrid tensile strength, \( T_{\text{ult}} \), required at the connections to compare to the \( T_{\text{ult}} \) required for the other limit states, to help determine which limit state is controlling the design.

With the above in mind, for Layer 6, \( T_{\text{ult}} \) (min. required, on a strength per unit of reinforcement width basis) for connection strength is as follows, using \( RF_{CR} \) determined from the in-isolation geogrid creep rupture data:

\[
T_{\text{ult}}(\text{min. required}) = \left( \frac{1.35}{0.8 \times 0.9} \right) \times 0.211 \times 1.3 \times 1.5 \left( \frac{3.48}{1.98} \right) = 1.36 \text{ kips/ft},
\]

\[
T_{\text{al}} = \frac{(1.36 \text{ kips/ft})}{2.18} = 0.623 \text{ kips/ft}.
\]

On a strength per unit of reinforcement width basis, this minimum required geosynthetic \( T_{\text{al}} \) of 0.623 kips/ft is below the \( T_{\text{al}} \) value of the weakest geogrid product available for this wall system (i.e., Geogrid A, in which \( T_{\text{MARV}} = 3.50 \text{ kips/ft} \) and a \( T_{\text{al}} = 3.50/2.18 = 1.61 \text{ kips/ft} \)). So at this point, the minimum strength product available for this wall system is in fact controlling design.

### 15-E-8.5 Calculations for Pullout Limit State Design (Strength I)

The default pullout parameters \( \alpha \) and \( F^* \) specified in AASHTO (2020) are used for this example design and are the same as in Example 1 (\( \alpha = 0.8 \) and \( F^* = 0.452 \)). The vertical stress, \( \sigma_v \), over the reinforcement anchorage length, conservatively calculated using Equation 15-E-29, is the same as in Example 1 (0.173 ksf). Since pullout is usually most critical near the wall top, this example pullout calculation is carried out for Layer 10.

Using Equation 15-E-30, the required factored length of reinforcement in the resistant zone (i.e., behind the active wedge) for Layer 10 is:

\[
L_e = \frac{\gamma_{p-EV} T_{\text{max}}}{\phi_{po} C(\alpha F^*) \sigma_p R_c}
\]

\( T_{\text{max}} \) used here corresponds to the minimum stiffness product available for the wall system (i.e., \( J_i = 19.2 \text{ kips/ft} \)). As before, \( R_c = 0.90 \) and all other parameters and their values have been defined earlier. For layer 10, per ft of reinforcement width:

\[
L_e = \frac{1.35 \times 0.061 \text{ kips/ft}}{0.70 \times 2(0.8 \times 0.452)(0.173 \text{ ksf})(0.90) = 1.04 \text{ ft}}.
\]

To determine the total reinforcement length needed, \( L \), the length of reinforcement within the active zone, \( L_a \), must be calculated. For geosynthetic walls, the active zone is assumed to be bounded by the Rankine active zone (wedge). \( L_a \) is calculated as follows for a vertical wall (at Layer 10):

\[
L_a = (H - z) \tan (45^\circ - \phi_t/2) = (20 \text{ ft} - 1.33 \text{ ft}) \tan (45^\circ - 34^\circ/2) = 9.93 \text{ ft}.
\]
The minimum length allowed for $L_e$ is 3 ft (AASHTO 2020), which is greater than the calculated $L_e$ required for pullout for layer 10. Therefore, using $L_e = 3$ ft, the total reinforcement length required for layer 10 is:

$$L = L_a + L_e = 9.93 \text{ ft} + 3 \text{ ft} = 12.9 \text{ ft}$$

Strength Limit State calculation results for all the layers (i.e., reinforcement rupture, connection rupture, and pullout) are summarized in Table 15-E-15 (a and b).

It should be recalled that the minimum length of reinforcement for typical reinforced soil walls is $0.7H = 0.7 \times (20 \text{ ft}) = 14 \text{ ft}$. Therefore, pullout does not control the reinforcement length required. Note that other limit state design calculations can result in greater reinforcement length, such as external stability (e.g., sliding) or global stability.

### 15-E-8.6 Calculations for Determination of $T_{\text{max}} + T_{\text{md}}$

**(Extreme Event I - Seismic)**

The calculation of $T_{\text{max}}$ as described and carried out earlier in this example using the Stiffness Method is also applicable for seismic design for the static portion of the reinforcement load. $T_{\text{md}}$, the incremental dynamic inertia force per reinforcement layer, must be added to $T_{\text{max}}$ to determine the total reinforcement load for each layer during seismic loading.

$T_{\text{md}}$ is calculated as shown for Example 2, considering the weight of the facing blocks.

$$T_{\text{md}} = \left( \frac{P_i}{n} \right) = \frac{4.24 \frac{kips}{ft}}{10} = 0.424 \frac{kips}{ft}$$

The total load per reinforcement layer during seismic shaking, $T_{\text{totalf}}$, is then calculated using superposition as follows, for Layer 6):

$$T_{\text{totalf}} = \gamma_{\text{seis}}(T_{\text{max}} + T_{\text{md}}) = 1.0 \left( 0.211 \frac{kips}{ft} + 0.424 \frac{kips}{ft} \right) = 0.635 \frac{kips}{ft}$$

For seismic pullout design, $T_{\text{totalf}}$ is used. However, for reinforcement rupture and connection rupture under seismic loading, the static and dynamic components of the reinforcement load must be handled separately. The reason for this is that the strength required to resist $T_{\text{max}}$ must include the effects of creep because it is a sustained load, but the strength required to resist $T_{\text{md}}$ should not include the effects of creep due to the transient nature of $T_{\text{md}}$. 
15-E-8.7 Calculations for Reinforcement Rupture (Extreme Event I - Seismic)

Using equations 15-E-21, 15-E-22, and 15-E-23, \( T_{ult} \) for the static portion of load at Layer 6 is:

\[
S_{rs} = \frac{Y_{seis} T_{max} RF}{\phi R_c} = \frac{1.0 \times 0.211 \frac{kips}{ft} \times 2.18}{1.0 \times 0.90} = 0.511 \frac{kips}{ft}
\]

\( T_{ult} \) for dynamic portion of load at Layer 6 is:

\[
S_{rt} = \frac{Y_{seis} T_{md} R_{F1D} R_D}{\phi R_c} = \frac{1.0 \times 0.424 \frac{kips}{ft} \times 1.12 \times 1.3}{1.0 \times 0.90} = 0.686 \frac{kips}{ft}
\]

\( T_{ult} = S_{rs} + S_{rt} = 0.511 \text{ kips/ft} + 0.687 \text{ kips/ft} = 1.20 \text{ kips/ft} \)

\( T_{al} = 1.20 \text{ kips/ft} / 2.18 = 0.550 \text{ kips/ft of reinforcement product width.} \)

15-E-8.8 Calculations for Connection Rupture (Extreme Event I - Seismic)

To determine the \( T_{ult} \) needed to prevent connection rupture during seismic loading, need \( CR_u \) and \( CR_{cr} \), which are calculated as follows for Layer 6:

\[
CR_u = \frac{T_{ultconn}}{T_{tot}} = \frac{1.98 \frac{kips}{ft}}{3.484 \frac{Kips}{ft}} = 0.568
\]

\[
CR_{cr} = \frac{T_{ultconn}}{R_{F_C} T_{tot}} = \frac{1.98 \frac{kips}{ft}}{1.5 \times 3.484 \frac{Kips}{ft}} = 0.379
\]

Because this is a frictional connection, the connection resistance is reduced using a factor, \( F_r \), of 0.8 to account for potential loss of frictional resistance due to the earthquake ground motion.

\[
S_{rsc} = \frac{Y_{seis} T_0 R_D}{F_r \phi CR_{cr} R_c} = \frac{1.0 \times 0.211 \frac{kips}{ft} \times 1.3}{0.8 \times 1.0 \times 0.379 \times 0.90} = 1.01 \frac{kips}{ft}
\]

\[
S_{rtc} = \frac{Y_{seis} T_{md} R_D}{F_r \phi CR_{cr} R_c} = \frac{1.0 \times 0.424 \frac{kips}{ft} \times 1.3}{0.8 \times 1.0 \times (0.568) \times 0.90} = 1.35 \frac{kips}{ft}
\]

\( T_{ult} = S_{rsc} + S_{rtc} = 1.01 \text{ kips/ft} + 1.35 \text{ kips/ft} = 2.36 \text{ kips/ft} \)

\( T_{al} = 2.36 \text{ kips/ft} / 2.18 = 1.08 \text{ kips/ft of reinforcement product width.} \)

On a load per reinforcement product width basis, this minimum required geosynthetic \( T_{al} \) of 1.08 kips/ft is still below the \( T_{al} \) value of the minimum geogrid product tensile strength \( T_{al} \) available for this wall system (i.e., Geogrid A) of 1.61 kips/ft. Therefore, the minimum strength product available for this wall system is still controlling the design and can be used for all the reinforcement layers.
15-E-8.9 Calculations for Pullout Limit State Design (Extreme Event I - Seismic)

Using equations 15-E-29 and 15-E-30, \( L_e \) for Layer 10 (i.e., at the wall top) is determined as follows:

\[
\sigma_v = z \gamma_f + S_{sur}\gamma_f
\]

\[
\sigma_v = (1.33 \text{ ft})(0.13 \text{ kcf}) + (0)(0.13 \text{ kcf}) = 0.173 \text{ ksf}
\]

\[
L_e = \frac{y_{seis}(T_{\text{max}} + T_{\text{mat}})}{\phi C (0.8 \alpha F^*) \sigma_v R_c}
\]

\[
L_e = \frac{1.0(0.061 + 0.424)}{1.0 \times 2 \times (0.8 \times 0.8 \times 0.452) \times 0.173 \text{ kcf} \times 0.90}
\]

\[
L_e = 5.39 \text{ ft}
\]

\[
L_a = (H - z) \tan (45^\circ - \phi_f/2) = (20 \text{ ft} - 1.33 \text{ ft}) \tan (45^\circ - 34^\circ/2) = 9.93 \text{ ft}
\]

\[
L = L_a + L_e = 9.93 \text{ ft} + 5.39 \text{ ft} = 15.3 \text{ ft}
\]

Therefore, at the wall top, pullout is controlling the reinforcement length needed.

15-E-8.10 Calculations to Check to Make Sure Stiffness and Strength Required are Properly Matched (Extreme Event I, Seismic)

A substantial increase in tensile strength was required to achieve internal stability for seismic loading, given the high ground acceleration required for this hypothetical site in the Puget Sound region of western Washington. However, the weakest geogrid product available for the example wall system (i.e., Geogrid A) has a \( T_{\text{al}} \) of 1.61 kips/ft of reinforcement product width, which is significantly greater than the \( T_{\text{al}} \) needed of 1.08 kips/ft. Therefore, an additional iteration to match the available reinforcement strength and stiffness to the demand is not required.

Typically, this check will show that another iteration to complete the wall design is not required, except possibly for very inefficient facing/reinforcement connections combined with high seismic loading.

15-E-8.11 Summary for Example 4 Design

See Table 15-E-14 for the calculation results for all of the layers for the Service I, Strength I, and Extreme Event I limit states. These calculation results are plotted in figures 15-E-15 through 15-E-17. In Figure 15-E-15, the minimum tensile strength needed to just meet the soil failure limit state is also shown for illustration purposes, which demonstrates that the strength required to just meet the soil failure limit is significantly less than the strength of the minimum tensile strength geogrid (i.e., Geogrid A) available for the wall system.
In summary, for the final internal stability design for Example 4 using the Stiffness Method, the following would be specified with regard to reinforcement properties and spacing for the optimized design, and the following observations can be made:

- The vertical spacing of reinforcements is 2 ft throughout the wall height, and the coverage ratio is a minimum of 0.90.

- The minimum reinforcement length for internal stability (i.e., pullout) is 14 ft for the Strength Limit, and all layers except the top layer for seismic did not exceed this minimum length. The top layer length required is 15.3 ft for seismic for the Stiffness Method, which is greater than the 0.7H minimum. The longer reinforcement length due to pullout is consistent with good detailing for MSE walls in AASHTO LRFD Article 11.10.7.4.

- The lowest strength PET geogrid included with the wall system was greater in strength and stiffness than required by the Stiffness Method design (see figures 15-E-15 and 15-E-16). Comparative calculations were done with the Simplified Method, but those calculations showed that greater tensile strength than the minimum strength product for the wall system and reduced vertical spacing were required to have enough reinforcement for equilibrium (not shown in figures 15-E-15 and 15-E-16, as a layer by layer comparison between methods was not possible due to the increase in the number of layers needed for the Simplified Method).

- Using Geogrid A as the minimum strength geogrid available, the Simplified Method required a total long-term tensile strength $T_{ul}$ of 30.1 kips/ft for the entire wall section (distributed among 15 reinforcement layers), whereas the Stiffness Method would allow Geogrid A to be used for all layers, but distributed among only 10 reinforcement layers, for a total of 16.1 kips/ft for the entire wall section. Therefore, the Stiffness Method would require only 53% of the total reinforcement strength required by the Simplified Method. The Stiffness method could have allowed even less total reinforcement strength to be used if the coverage ratio $R_c$ was reduced to less than 0.90, or if a weaker geogrid was available for this system.

- Overall, the required strength and stiffness of the reinforcement needed is in the low to mid-range of the geogrid product lines available. Note that the ground acceleration used for the seismic design represents what would be included in Seismic Zone 4, which is the highest seismic zone.
Table 15-E-14  Summary of Example 4 wall design calculations using Stiffness Method, $R_c = 0.90$, and frictional wall face connection (Service, Strength, and Extreme Event I Limit States): (a) Calculation of $T_{max}$ using minimum strength product available for wall system, (b) Service and Strength Limit State calculations, and (c) Extreme Event I (seismic) Limit State calculations.

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>z (ft)</th>
<th>S (ft)</th>
<th>$^a R_{LJ}$ (kips/ft)</th>
<th>$S_{global}$ (kssf)</th>
<th>$S_{local}$ (kssf)</th>
<th>$D_{max}$</th>
<th>$F_t$</th>
<th>$\Phi_s$</th>
<th>$\Phi_{local}$</th>
<th>$\Phi_b$</th>
<th>$\Phi_a$</th>
<th>$T_{max}$ and $T_s$ (kips/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 (top)</td>
<td>1.33</td>
<td>2.33</td>
<td>17.3</td>
<td>8.65</td>
<td>7.42</td>
<td>0.220</td>
<td>1.61</td>
<td>0.231</td>
<td>0.92</td>
<td>1.0</td>
<td>0.757</td>
<td>0.061</td>
</tr>
<tr>
<td>9</td>
<td>3.33</td>
<td>2.00</td>
<td>17.3</td>
<td>8.65</td>
<td>8.65</td>
<td>0.372</td>
<td>1.61</td>
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<td>1.00</td>
<td>1.0</td>
<td>0.757</td>
<td>0.095</td>
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<tr>
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<td>17.3</td>
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<td>8.65</td>
<td>0.523</td>
<td>1.61</td>
<td>0.231</td>
<td>1.00</td>
<td>1.0</td>
<td>0.757</td>
<td>0.134</td>
</tr>
<tr>
<td>7</td>
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<td>8.65</td>
<td>8.65</td>
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<td>8.65</td>
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<td>1.61</td>
<td>0.231</td>
<td>1.00</td>
<td>1.0</td>
<td>0.757</td>
<td>0.256</td>
</tr>
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<td>15.33</td>
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<td>17.3</td>
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<td>8.65</td>
<td>1.00</td>
<td>1.61</td>
<td>0.231</td>
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<td>1.0</td>
<td>0.757</td>
<td>0.256</td>
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<td>2.00</td>
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<td>8.65</td>
<td>1.00</td>
<td>1.61</td>
<td>0.231</td>
<td>1.00</td>
<td>1.0</td>
<td>0.757</td>
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<td>0.234</td>
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</table>

Base of wall 20

\[ \sum T_{max} = 1.93 \]

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>z (ft)</th>
<th>$^a Reinforcement Product Available for Wall System</th>
<th>$^a Minimum Available Product Tensile Strength, $T_a$ (kips/ft)</th>
<th>$^a Minimum Required Strength per Unit Width of Reinforcement, $T_{min}$ = $T_a \times RF$ (kips/ft)</th>
<th>$^b Connection Capacity T_{pulldown}$ (kips/ft)</th>
<th>$^b Minimum required strength per Unit Width of Reinforcement, $T_{ul} = T_{pulldown} \times RF$ (kips/ft)</th>
<th>$^b Required Connection Rupture$ (Service Limit)</th>
<th>Anchorage length $L_a$ (ft)</th>
<th>Pullout (Strength Limit)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column→</td>
<td>1</td>
<td>Geogrid A</td>
<td>1.61</td>
<td>0.11</td>
<td>0.25</td>
<td>1.09/0.31</td>
<td>0.32</td>
<td>0.69</td>
<td>1.04/1.04</td>
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<td>Geogrid A</td>
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<td>0.91</td>
<td>0.86/0.65</td>
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<td>Geogrid A</td>
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<td>0.25</td>
<td>0.55</td>
<td>1.55/0.45</td>
<td>0.50</td>
<td>1.10</td>
<td>0.93/0.57</td>
</tr>
<tr>
<td>8</td>
<td>5.33</td>
<td>Geogrid A</td>
<td>1.61</td>
<td>0.32</td>
<td>0.71</td>
<td>1.79/0.51</td>
<td>0.57</td>
<td>1.24</td>
<td>1.20/0.54</td>
</tr>
<tr>
<td>7</td>
<td>7.33</td>
<td>Geogrid A</td>
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<td>0.40</td>
<td>0.87</td>
<td>1.95/0.56</td>
<td>0.62</td>
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<td>1.47/0.52</td>
</tr>
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<td>1.02</td>
<td>2.05/0.59</td>
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<td>1.46</td>
<td>1.73/0.50</td>
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<td>2.15/0.62</td>
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<td>1.78/0.44</td>
</tr>
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<td>0.48</td>
<td>1.05</td>
<td>2.24/0.64</td>
<td>0.66</td>
<td>1.45</td>
<td>1.78/0.38</td>
</tr>
<tr>
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<td>0.48</td>
<td>1.05</td>
<td>2.34/0.67</td>
<td>0.65</td>
<td>1.43</td>
<td>1.78/0.34</td>
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<td>0.44</td>
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<td>2.42/0.69</td>
<td>0.59</td>
<td>1.29</td>
<td>1.62/0.28</td>
</tr>
<tr>
<td>1</td>
<td>19.33</td>
<td>Geogrid A</td>
<td>1.61</td>
<td>0.44</td>
<td>0.96</td>
<td>2.42/0.69</td>
<td>0.59</td>
<td>1.29</td>
<td>1.62/0.28</td>
</tr>
</tbody>
</table>

Base of wall 20

\[ \sum T_{ul} = 16.1 \]
Table 15-E-14, continued.

Summary of Example 4 wall design calculations using Stiffness Method, $R_c = 0.90$, and frictional wall face connection (Service, Strength, and Extreme Event I Limit States): (c) Extreme Event I (seismic) Limit State calculations.

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>$z$ (ft)</th>
<th>Reinforcement Rupture (Extreme Event I Limit)</th>
<th>Connection Rupture (Extreme Event I Limit)</th>
<th>Pullout (Extreme Event I Limit)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$^\text{a}$Reinforcement Product Available for Wall System</td>
<td>$^\text{b}$Minimum Available Product Tensile Strength, $T_{ul}$ (kip/ft)</td>
<td>$^\text{c}$Minimum Required Strength per Unit Width of Reinforcement, $T_{ul} = T_{ul} \times RF (kip/ft)$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_{ul}$ (kip/ft)</td>
<td>$T_{ul} = T_{ul} \times RF (kip/ft)$</td>
<td>$T_{ul}$ (kip/ft)</td>
</tr>
<tr>
<td>Column→</td>
<td></td>
<td>$3$</td>
<td>$4$</td>
<td>$7$</td>
</tr>
<tr>
<td>10 (top)</td>
<td>1.33</td>
<td>1.61</td>
<td>0.38</td>
<td>0.83</td>
</tr>
<tr>
<td>9</td>
<td>3.33</td>
<td>1.61</td>
<td>0.42</td>
<td>0.92</td>
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<td>0.46</td>
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<td>0.51</td>
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</tr>
<tr>
<td>6</td>
<td>9.33</td>
<td>1.61</td>
<td>0.55</td>
<td>1.20</td>
</tr>
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<td>11.33</td>
<td>1.61</td>
<td>0.59</td>
<td>1.29</td>
</tr>
<tr>
<td>4</td>
<td>13.33</td>
<td>1.61</td>
<td>0.60</td>
<td>1.31</td>
</tr>
<tr>
<td>3</td>
<td>15.33</td>
<td>1.61</td>
<td>0.60</td>
<td>1.31</td>
</tr>
<tr>
<td>2</td>
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<td>1.61</td>
<td>0.60</td>
<td>1.31</td>
</tr>
<tr>
<td>1</td>
<td>19.33</td>
<td>1.61</td>
<td>0.57</td>
<td>1.25</td>
</tr>
<tr>
<td>Base of wall</td>
<td>20</td>
<td>$\sum T_{ul} = 16.1$</td>
<td>$\sum T_{ul} = 5.28$</td>
<td>$\sum T_{ul} = 11.5$</td>
</tr>
</tbody>
</table>

$^\text{a}$Minimum geosynthetic stiffness available for wall system (i.e., Geogrid A).

$^\text{b}$These values are on a load per unit of wall width basis in accordance with Article 11.10.6.4.1 of the AASHTO LRFD Bridge Design Manual (AASHTO 2020).

$^\text{c}$For comparison to geogrid product tensile strength that is not reduced by $R_c$ (i.e., strength per unit of reinforcement width basis).

$^\text{d}$Tmd for all reinforcement layers is 0.424 kips/ft.
Figure 15-E-15  Stiffness Method internal stability design for Service and Strength limit states (Example 4).

Figure 15-E-16  Stiffness Method internal stability design to the Simplified Method design, for Extreme Event I (seismic) limit state (Example 4).
Figure 15-E-17  Comparison of Stiffness Method internal stability design to the Simplified Method design, for Extreme Event I (seismic) limit state, pullout (Example 4).
15-E-9  Stiffness Method Design Example 5: Wrapped (Flexible) Faced Geosynthetic Wall Using Backfill with Small Cohesion

15-E-9.1  General

This example is an extension of Example 1 to consider the effect of wall backfill soil cohesion on wall behavior during and after wall construction. Therefore, this is not a completely developed example. The only purpose of this example is to demonstrate the types of design problems, and possibly long-term wall performance problems, that may occur if cohesion is present, especially if the design is conducted taking into account the “beneficial” effect that cohesion can have in reducing the reinforcement load, $T_{max}$. The key issue here is the reliability of the cohesion long-term (e.g., will changes in moisture content, softening of the clayey backfill, or soil creep over time occur, reducing the cohesion and allowing the reinforcement layer $T_{max}$ values to increase over time?). In addition to this, as the fines content and plasticity of the backfill increase, the more likely is the buildup of water in the backfill to occur, causing large increases in reinforcement load and wall face deformations (Allen and Bathurst 2009).

Material properties are the same as in Example 1, which are provided in Table 15-E-3, with the exception that the backfill is assumed to have some clayey fines, resulting in a relatively small soil cohesion of 0.20 ksf in addition to the friction angle of $34^\circ$.

Table 15-E-2 provides requirements for how to address soil cohesion in the wall backfill. In general, backfill materials that have some cohesion should be avoided, especially in western Washington where rainfall is relatively plentiful. However, in the unusual case in which MSE wall backfill with a limited amount of cohesion cannot be avoided, the effect of that soil cohesion on wall strains and deformations can be assessed using the Stiffness Method.

In this example, the effect of this cohesion on $T_{max}$ at end of construction (i.e., EOC) for the wall, and potential loss of that cohesion over time, is investigated. The results generated in this example will be compared to the Example 1 design, which in effect is the design that would be done if some cohesion is present, but the cohesion is ignored (i.e., using $\phi = 34^\circ$ and $c = 0$). The reinforcement stiffness for this example is assumed to be the same as is used in Example 1 (i.e., see Table 15-E-3).
For this example design, determination of the minimum reinforcement stiffness required to keep the peak strain level in the reinforcement layers at 2.5% or less required trial-and-error to optimize the reinforcement design. As in the previous example, Layer 6 will be the primary focus to illustrate the method, except for pullout, in which the uppermost layer (Layer 10) is the focus to illustrate the method. Note that the coverage ratio, \( R_c \), is equal to 1.0 for this example.

\[
S_{\text{global}} = \frac{J_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} R_c J_i}{H} = (4 \times 1.0 \times 8.6 \text{ kips/ft} + 6 \times 1.0 \times 17.0 \text{ kips/ft})/20 \text{ ft} = 6.82 \text{ ksf}
\]

\[
\Phi_g = \alpha \left( \frac{S_{\text{global}}}{p_u} \right)^{\beta} = 0.16 \times (6.82 \text{ ksf}/2.11 \text{ ksf})^{0.26} = 0.217
\]

\[
\Phi_{fb} = \left( \frac{K_{\text{abh}}}{K_{\text{ash}}} \right)^d = (0.283/0.283)^{0.4} = 1.0
\]

\[
S_{\text{local}} = \left( \frac{R_c J_i}{S_p} \right)_i = (1.0 \times 17.0 \text{ kips/ft})/(2.0 \text{ ft}) = 8.50 \text{ ksf for Layer 6}
\]

\[
S_{\text{localave}} = \frac{\sum \left( \frac{R_c J_i}{S_p} \right)_i}{n} = \frac{3.65 + 3 \times 4.25 + 5 \times 8.50 + 10.2}{10} = 6.91 \text{ ksf}
\]

where, \( n = 10 \) is the number of reinforcement layers. Therefore, \( \Phi_{\text{local}} \) is calculated as follows:

\[
\Phi_{\text{local}} = \left( \frac{S_{\text{local}}}{S_{\text{localave}}} \right)^{0.5} = \left( \frac{8.5 \text{ ksf}}{6.91 \text{ ksf}} \right)^{0.5} = 1.11 \ (\text{layer 6})
\]

For a flexible wall face, the facing stiffness factor is assumed to be 1.0.

Given \( c = 0.20 \text{ ksf} \), the cohesion factor, \( \Phi_c \) is calculated as follows:

\[
\Phi_c = e^{-16(c/(\gamma_r H))} = e^{-16(0.20 \text{ ksf} / (0.13 \text{ kcf} \times 20 \text{ ft}))} = 0.292
\]

\( D_{\text{tmax}} \) is determined for Layer 6 as follows:

\[
z_b = C_h \times (H)^{y} \times \Phi_b = (0.32 \times (20 \text{ ft})^{1.2}) \times 1.0 = 11.65 \text{ ft}
\]

For \( z \leq z_b \): \( D_{\text{tmax}} = D_{\text{tmax0}} + (z/z_b) \times (1 - D_{\text{tmax0}}) = 0.12 + (9.33 \text{ ft}/11.65 \text{ ft}) \times (1 - 0.12) = 0.825\)
For bottom layers where $z > z_b$: $D_{tmax} = 1.0$

$T_{max}$ for Layer 6 is calculated as follows:

$$T_{max} = S_v \left[ H \gamma_f D_{tmax} + \left( \frac{H_{ref}}{H} \right) S \gamma_f + LL \right] K_{avh} \Phi_f \Phi_g \Phi_{fs} \Phi_{locat} \Phi_c$$

$$T_{max} = 2.0 \text{ ft} \left[ 20 \text{ ft} \times 0.130 \text{ kcf} \times 0.825 + \left( \frac{20 \text{ ft}}{20 \text{ ft}} \right) 0 \text{ ft} \times 0.130 \text{ kcf} + 0 \right] \times 1.0 \times 0.217 \times 1.0 \times 1.11 \times 0.292$$

$$T_{max} = 0.0853 \text{ kips/ft}$$

$T_{max}$, and the calculated parameters needed to calculate $T_{max}$, are summarized in Table 15-E-15 for the rest of the layers.

Note that $T_{max}$ for this layer not considering cohesion is 0.292 kips/ft.

Using Equation 15-14 with load factor $\gamma_{sf} = 1.2$, and resistance factor $\phi_{sf} = 1.0$ (Table 15-5), the factored reinforcement strain corresponding to Layer 10 is computed as:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_{ci} \gamma} = \frac{1.2 \times 0.017 \text{ kips/ft}}{1.0 \times 1.0 \times 8.6 \text{ kips/ft}} \times 100\% = 0.25\% \leq 2.5\% \quad OK$$

Assuming no cohesion (i.e., as shown in Example 1), for Layer 10,

$$\varepsilon_{rein} = 0.83\% \not\text{ considering cohesion} \geq 0.25\% \text{ considering cohesion}$$

For Layer 6:

$$\varepsilon_{rein} = \frac{\gamma_{p-EVsf} T_{max}}{\phi_{sf} R_{ci} \gamma} = \frac{1.2 \times 0.0853 \text{ kips/ft}}{1.0 \times 1.0 \times 17.0 \text{ kips/ft}} \times 100\% = 0.60\% \leq 2.5\% \quad OK$$

Assuming no cohesion (i.e., as shown in Example 2), for Layer 6,

$$\varepsilon_{rein} = 2.06\% \not\text{ considering cohesion} \geq 0.60\% \text{ considering cohesion}$$

See Table 15-E-15 for the calculation results for the rest of the layers.

Based on these calculations, the reinforcement strains increase by a factor of approximately 3.5 post-construction (i.e., 0.83%/0.25%, and 2.06%/0.60%), as when the cohesion is ignored during design, the cohesion will still be present during construction and reduce the reinforcement loads and strains accordingly, as illustrated here. However, the final reinforcement strains and loads long-term will still be as designed with the cohesion ignored and will meet standards. The key is the effect that cohesion loss over time, if it occurs, will have on post-construction wall face deformation, and whether or not the wall, and whatever it supports, can successfully handle that post-construction deformation. Based on experience, a reinforcement strain increase of approximately 1 to 1.5% could result in a face deformation increase of approximately 1 inch for a 20 ft high wall.
Figure 15-E-18 provides plots that compare the strains that result for various assumptions regarding the short-term and long-term presence of cohesion. If the wall is designed considering this limited amount of cohesion (i.e., 0.20 ksf) and the absolute minimum creep stiffness needed to meet the design criteria, reinforcement strains quickly become excessive if that cohesion is lost over time post-construction (i.e., as high as 8.6% as shown in Table 15-E-15 and over 6% strain post-construction), and does not consider the effect of water build-up in the wall backfill due to reduced drainage characteristics (see Allen and Bathurst 2009 for an assessment of wall backfill water build-up on the probability of failure).

It is for this reason that completing the wall design taking into account the soil cohesion, which will result in a reduced reinforcement strength and stiffness, if backfill that has a small to moderate amount of cohesion is the only backfill available, shall not be done (i.e., for final wall design, always assume that $\Phi_c = 1.0$ whether or not some limited cohesion in the wall backfill is present). However, if a limited amount of cohesion is present in the backfill, the Stiffness Method may be used to assess how much post-construction strain in the reinforcement may occur if that cohesion disappears over time.
### Table 15-E-15  Summary of Example 5 wall design calculations using Simplified Stiffness Method (Service and Strength Limit States).

<table>
<thead>
<tr>
<th></th>
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<th></th>
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<td>1.0</td>
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<td>N/A</td>
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**Base of wall: 20 ft**

\[ T_{\text{max}} = 0.72 \]

### Table 15-E-15  Summary of Example 5 wall design calculations using Simplified Stiffness Method (Service and Strength Limit States).

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>z (ft)</th>
<th>Not Accounting for Cohesion (as Designed in Example 1)</th>
<th>Accounting for Cohesion, c = 0.2 ksf</th>
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<td>[( \bar{T}_{\text{max}} = 2.47 )]</td>
<td>[( \bar{T}_{\text{max}} = 0.72 )]</td>
</tr>
</tbody>
</table>

\[ \varepsilon_{\text{ave}} = 2.06 \]

\[ \varepsilon_{\text{ave}} = 0.60 \]

\[ \bar{T}_{\text{max}} = 0.47 \]

\[ \bar{T}_{\text{max}} = 1.61 \]

\[ \varepsilon_{\text{ave}} = 1.89 \]

\[ \varepsilon_{\text{ave}} = 6.46 \]

---

*Using minimum creep stiffness of 3.0 kips/ft for all layers, which is well below the minimum stiffness available of 8.2 kips/ft. However, this is for illustration purposes only, and not recommended for design.

*Cohesion is lost over time due to softening of the backfill due to moisture increase, or, in the case of apparent cohesion, due to backfill moisture content changes, over time.
Figure 15-E-18  Comparison of Stiffness Method predicted factored reinforcement strains for wrapped face (flexible) wall with some soil cohesion (Example 5): (a) designed assuming cohesion is not present (i.e., $\phi = 34^\circ$ and $c = 0$; same as Example 1), (b) designed assuming cohesion is present (i.e., $\phi = 34^\circ$ and $c = 0.2$ ksf).
Summary of Lessons Learned from Design Examples

The provided design examples illustrate the use of the Stiffness Method for several geosynthetic wall design scenarios. These scenarios include flexible and stiff facings, and for the stiff faced walls, mechanical (i.e., structural) and friction dominated facing/reinforcement connections, coverage ratios ranging from 0.9 to 1.0, and cohesionless and cohesive soil backfills. Designs are carried out using both the Stiffness Method and the Simplified Method. Designs were carried out for internal stability (soil failure, reinforcement and connection rupture, and pullout) considering Service, Strength, and Extreme Event I (seismic) limit states.

Lessons learned from these examples are as follows:

- In all cases, for the Stiffness Method designs, the Soil Failure Limit controlled the amount and strength of reinforcements needed. However, for Example 4 (i.e., the block faced wall with frictional facing reinforcement connections), since it represented a hypothetical proprietary wall system, the minimum strength geogrid available for the system was stronger than the strength required to meet the soil failure limit state.

- For Example 1 (i.e., the flexible faced wall), the difference between the Stiffness and Simplified method designs was the least of all the examples (i.e., total $T_{al}$ for wall section of 10.6 kips/ft for the Stiffness Method and 11.9 kips/ft for the Simplified Method). The Stiffness Method, however, required less reinforcement in the lower half of the wall and more reinforcement in the upper half of the wall relative to the Simplified Method distribution of reinforcement strength. Example 4 (block faced wall with primarily frictional facing/reinforcement connections) had the largest difference in the Stiffness and Simplified method designs regarding the total reinforcement strength $T_{al}$ needed (i.e., total $T_{al}$ needed for wall section of 16.1 kips/ft for the Stiffness Method and 30.1 kips/ft for the Simplified Method).

- The main reason for the larger difference in total $T_{al}$ needed between the methods for Example 4 was due to the connection strength design for the Simplified Method, especially for seismic loading. This was mainly due to the fact that the Stiffness Method predicts a significantly lower reinforcement load (i.e., $T_{max}$ and $T_0$) than does the Simplified Method due to the greater prediction accuracy of the Stiffness Method.

- Comparison of examples 1 and 2 can be used to assess the effect of facing stiffness on the magnitude of the total $T_{al}$ needed for the wall design using the Stiffness Method. For the flexible faced wall (Example 1), the total $T_{al}$ needed was 10.6 kips/ft, whereas for the comparable stiff (i.e., block) faced wall (Example 2), the total $T_{al}$ needed was 9.4 kips/ft. The main reason for the difference was the reduction in $T_{max}$ resulting from the facing stiffness and its effect on the strength required to meet the soil failure limit state requirements. Had the connection strength controlled the Stiffness Method design, the difference between the flexible and stiff faced wall examples would have varied depending on the efficiency of the connection.
Example 5 was used to demonstrate the effect soil cohesion can have on the predicted reinforcement loads and strains when using the Stiffness Method. Even a small amount of cohesion (i.e., 0.2 ksf) can have a big effect on the predicted reinforcement load, and the amount of soil reinforcement needed. In general, backfill soil with clayey fines should be avoided. Based on this example, provided the backfill cohesion is small, ignoring the contribution of the soil cohesion to the soil shear strength used for design will result in a wall design with minimal risk of poor performance provided the fines content is not too high and good drainage is provided. However, it must be recognized that the reinforcement loads at end of wall construction will be reduced due to the cohesion whether or not the cohesion is ignored for the wall design. In this case, the effect of potential loss of soil cohesion due to longer term soil moisture content changes on reinforcement load and wall face deformation changes after wall construction could be investigated using the Stiffness Method. However, the final wall design should not be conducted taking advantage of the reduced reinforcement loads due to soil cohesion, as post-construction changes in the reinforcement loads and wall face deformation are likely to be unacceptable.

15-E-11 References


Allen, T. M, and Bathurst, R. J. (2009), "Reliability of Geosynthetic Wall Designs and Factors Influencing Wall Performance," 4th International GSI-Taiwan Geosynthetics Conference, Pingtung, Taiwan, pp. 95-123.


Appendix 15-F  Description of Typical Temporary Shoring Systems and Selection Considerations

15-F-1  Fill Applications

While most temporary retaining systems are used in cut applications, some temporary retaining systems are also used in fill applications. Typical examples include the use of MSE walls to support preload fills that might otherwise encroach into a wetland or other sensitive area, the use of modular block walls or wrapped face geosynthetic walls to support temporary access road embankments or ramps, and the use of temporary wrapped face geosynthetic walls to support fills during intermediate construction stages.

MSE walls, including wrapped face geosynthetic walls, are well suited for the support of preload fills because they can be constructed quickly, are relatively inexpensive, are suitable for retaining tall fill embankments, and can tolerate significant settlements. Modular block walls without soil reinforcement (e.g., ecology block walls) are also easy to construct and relatively inexpensive; however they should only be used to support relatively short fill embankments and are less tolerant to settlement than MSE walls. Therefore, block walls are better suited to areas with firm subgrade soils where the retained fill thickness behind the walls is less than 15 feet.

15-F-2  MSE Walls

MSE walls are described briefly in Section 15.5.3, and extensively in Publication No. FHWA-NHI-00-043 (Elias, et al., 2001). In general, MSE walls consist of strips or sheets of steel or polymeric reinforcement placed as layers in backfill material and attached to a facing. Facings may consist of concrete blocks or panels, gabions, or a continuation of the reinforcement layer.

15-F-3  Prefabricated Modular Block Walls

Prefabricated modular block walls without soil reinforcement are discussed in Section 15.5.4 and should be designed as gravity retaining structures. Concrete blocks used for gravity walls typically consist of 2½- by 2½- by 5-foot solid rectangular concrete blocks designed to interlock with each other. They are typically cast from excess concrete at concrete batch plants and are relatively inexpensive. Because of their rectangular shape they can be stacked a variety of ways. Because of the tightly fitted configuration of a concrete block wall, oversized blocks will tend to fit together poorly. Occasionally, blocks from a concrete batch plant are found to vary in dimension by several inches.

15-F-4  Common Cut Applications

A wide range of temporary shoring systems are available for cut applications. Each temporary shoring system has advantages and disadvantages, conditions where the system is suitable or not suitable, and specific design considerations. The following sections provide a brief overview of many common temporary shoring systems for cut applications. The "Handbook of Temporary Structures in Construction" (Ratay, 1996) is another useful resource for information on the design and construction of temporary shoring systems.
15-F-5 **Trench Boxes**

Trench boxes are routinely used to protect workers during installation of utilities and other construction operations requiring access to excavations deeper than 4 feet. Trench boxes consist of two shields connected by internal braces and have a fixed width and height. The typical construction sequence consists of excavation of a trench and then setting the trench box into the excavation prior to allowing workers to gain access to the protected area within the trench box. For utility construction, the trench box is commonly pulled along the excavation by the excavator as the utility construction advances. Some trench boxes are designed such that the trench boxes can be stacked for deeper excavations.

The primary advantage of trench boxes is that they provide protection to workers for a low cost and no site specific design is generally required. Another advantage is that trench boxes are readily available and are easy to use. One disadvantage of trench boxes is that no support is provided to the soils—where existing improvements are located adjacent to the excavation, damage may result if the soils cave-in towards the trench box. Therefore, trench boxes are not suitable for soils that are too weak or soft to temporarily support themselves. Another disadvantage of trench boxes is the internal braces extend across the excavation and can impede access to the excavation. Finally, trench boxes provide no cutoff for groundwater; thus, a temporary dewatering system may be necessary for excavations that extend below the water table for trench boxes to be effective.

Trench boxes are most suitable for trenches or other excavations where the depth is greater than the width of the excavation and soil is present on both sides of the trench boxes. Trench boxes are not appropriate for excavations that are deeper than the trench box.

15-F-6 **Sheet Piling**

Sheet piling is a common temporary shoring system in cut applications and is particularly beneficial as the sheet piles can act as a diaphragm wall to reduce groundwater seepage into the excavation. Sheet piling typically consists of interlocking steel sheets that are much longer than they are wide. Sheets can also be constructed out of vinyl, aluminum, concrete, or wood; however, steel sheet piling is used most often due to its ability to withstand driving stresses and its ability to be removed and reused for other walls. Sheet piling is typically installed by driving with a vibratory pile driving hammer. For sheet piling in cut applications, the piling is installed first, then the soil in front of the wall is excavated or dredged to the design elevation. There are two general types of sheet pile walls: cantilever, and anchored/braced.

Sheet piling is most often used in waterfront construction; although, sheet piling can be used for many upland applications. One of the primary advantages of sheet piling is that it can provide a cutoff for groundwater flow and the piles can be installed without lowering the groundwater table. Another advantage of sheet piling is that it can be used for irregularly shaped excavations. The ability for the sheet piling to be removed makes sheet piling an attractive shoring alternative for temporary applications. The ability for sheet piling to be anchored by means of ground anchors or deadman anchors (or braced internally) allows sheet piling to be used where deeper excavations are planned or where large surcharge loading is present. One disadvantage of sheet piling is that it is installed by vibrating or driving; thus, in areas where vibration sensitive improvements or soils are
present, sheet piling may not be appropriate. Another disadvantage is that where very dense soils are present or where cobbles, boulders or other obstructions are present, installation of the sheets is difficult.

### 15-F-7 Soldier Piles

Soldier pile walls are frequently used as temporary shoring in cut applications. The ability for soldier piles to withstand large lateral earth pressures and the proven use adjacent to sensitive infrastructure make soldier piles an attractive shoring alternative. Soldier pile walls typically consist of steel beams installed in drilled shafts; although, drilled shafts filled with steel cages and concrete or precast reinforced concrete beams can be used. Following installation of the steel beam, the shaft is filled with structural concrete, lean concrete, or a combination of the two. The soldier piles are typically spaced 6 to 8 feet on center. As the soil is excavated from in front of the soldier piles, lagging is installed to retain the soils located between adjacent soldier piles. The lagging typically consists of timber; however, reinforced concrete beams, reinforced shotcrete, or steel plates can also be used as lagging. Ground anchors, internal bracing, rakers, or deadman anchors can be incorporated in soldier pile walls where the wall height is higher than about 12 feet, or where backslopes or surcharge loading are present.

Soldier piles are an effective temporary shoring alternative for a variety of soil conditions and for a wide range of wall heights. Soldier piles are particularly effective adjacent to existing improvements that are sensitive to settlement, vibration, or lateral movement. Construction of soldier pile walls is more difficult in soils prone to caving, running sands, or where cobbles, boulders or other obstructions are present; however, construction techniques are available to deal with nearly all soil conditions. The cost of soldier pile walls is higher than some temporary shoring alternatives. In most instances, the steel soldier pile is left in place following construction. Where ground anchors or deadman anchors are used, easements may be required if the anchors extend outside the right-of-way/property boundary. Where ground anchors are used and soft soils are present below the base of the excavation, the toe of the soldier pile should be designed to prevent excessive settlements.

### 15-F-8 Prefabricated Modular Block Walls

In general, modular blocks (see Section 15.6.6.1.2) for cut applications require the soil deposit to have adequate standup time such that the excavation can be made and the blocks placed without excessive caving. Otherwise large temporary backcuts and subsequent backfill placement may be required. A key advantage to modular block walls is that the blocks can be removed and reused after the temporary structure is no longer needed. One disadvantage to using modular blocks in cut applications is that the blocks are placed in front of an excavation and the soils are initially not in full contact with the blocks unless the areas is backfilled. Some movement of the soil mass is required prior to load being applied to the blocks—this movement can be potentially damaging to upslope improvements.
15-F-9  **Braced Cuts**

Braced cuts are used in applications where a temporary excavation is required that provides support to the retained soils in order to reduce excessive settlement or lateral movement of the retained soils. Braced cuts are generally used for trenches or other excavations where soil is present on both sides of the excavation and construction activities are not affected by the presence of struts extending across the excavation. A variety of techniques are available for constructing braced cuts; however, most include a vertical element, such as a sheet pile, metal plate, or a soldier pile, that is braced across the excavation by means of struts. Many of the considerations discussed below for soldier pile walls and sheet piling apply to braced cuts.

15-F-10  **Soil Nail Walls**

The soil nail wall system consists of drilling and grouting rows of steel bars or "nails" behind the excavation face as it is excavated and then covering the face with reinforced shotcrete. The placement of soil nails reinforces the soils located behind the excavation face and increases the soil's ability to resist a mass of soil from sliding into the excavation. Soil nail walls are typically used in dense to very dense granular soils or stiff to hard, low plasticity, fine-grained soils. Soil nail walls are less cost effective in loose to medium dense sands or soft to medium stiff/high plasticity fine-grained soils.

The soils typically are required to have an adequate standup time (to allow placement of the steel wire mesh and/or reinforcing bars to be installed and the shotcrete to be placed). Soils that have short standup times are problematic for soil nailing. Many techniques are available for mitigating short standup time, such as installation of vertical elements (vertical soil nails or light steel beams set in vertical drilled shafts placed several feet on center along the perimeter of the excavation), drilling soil nails through soil berms, use of slot cuts, and flash-coating with shotcrete. Easements may be required if the soil nails extend outside the right-of-way/property boundary.

15-F-11  **Uncommon Shoring Systems for Cut Applications**

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

15-F-11.1  **Diaphragm/Slurry Walls**

Diaphragm/slurry walls are constructed by excavating a deep trench around the proposed excavation. The trench is filled with a weighted slurry that keeps the excavation open. The width of the trench is at least as wide as the concrete wall to be constructed. The slurry trench is completed by installing steel reinforcement cages and backfilling the trench with tremied structural concrete that displaces the slurry. The net result is a continuous wall that significantly reduces horizontal ground water flow. Once the concrete cures, the soil is excavated from in front of the slurry wall. Internal bracing and/or ground anchors can be incorporated into slurry walls. Diaphragm/slurry walls can be incorporated into a structure as permanent walls.

Diaphragm/slurry walls are most often used where groundwater is present above the base of the excavation. Slurry walls are also effective where contaminated groundwater is to be contained. Slurry walls can be constructed in dense soils where the use of sheet piling is difficult. Other advantages of slurry walls include the ability to withstand significant
vertical and lateral loads, low construction vibrations, and the ability to construct slurry walls in low-headroom conditions. Slurry walls are particularly effective in soils where high groundwater and loose soils are present, and dewatering could lead to settlement related damage of adjacent improvements, assuming that the soils are not so loose or soft that the slurry is inadequate to prevent squeezing of the very soft soil.

In addition to detailed geotechnical design information, diaphragm/slurry walls require jobsite planning, preparation and control of the slurry, and contractors experienced in construction of slurry walls. For watertight applications, special design and construction considerations are required at the joints between each panel of the slurry wall.

15-F-11.2 Secant Pile Walls

Secant pile walls are another type of diaphragm wall that consist of interconnected drilled shafts. First, every other drilled shaft is drilled and backfilled with low strength concrete without steel reinforcement. Next, structural drilled shafts are installed between the low strength shafts in a manner that the structural shafts overlap the low strength shafts. The structural shafts are typically backfilled with structural concrete and steel reinforcement. The net result is a continuous wall that significantly reduces horizontal ground water flow while retaining soils behind the wall.

Secant pile walls are typically more expensive than many types of cut application temporary shoring alternatives; thus, the use of secant pile walls is limited to situations where secant pile walls are better suited to the site conditions than other shoring alternatives. Conditions where secant pile walls may be more favorable include high groundwater, the need to prevent migration of contaminated groundwater, sites where dewatering may induce settlements below adjacent improvements, sites with soils containing obstructions, and sites where vibrations need to be minimized.

15-F-11.3 Cellular Cofferdams

Sheet pile cellular cofferdams can be used for applications where internal bracing is not desirable due to interference with construction activities within the excavation. Cellular cofferdams are typically used where a dewatered work area or excavation is necessary in open water or where large dewatered heads are required. Cellular cofferdams consist of interlocking steel sheet piles constructed in a circle, or cell. The individual cells are constructed some distance apart along the length of the excavation or area to be dewatered. Each individual cell is joined to adjacent cells by arcs of sheet piles, thus providing a continuous structure. The cells are then filled with soil fill, typically granular fill that can be densified. The resulting structure is a gravity wall that can resist the hydrostatic and lateral earth pressures once the area within the cellular cofferdam is dewatered or excavated. As a gravity structure, cellular cofferdams need adequate bearing; therefore, sites where the cellular cofferdam can be founded on rock or dense soil are most suitable for these structures.

Cellular cofferdams are difficult to construct and require accurate placement of the interlocking sheet piles. Sites that require installation of sheet piles through difficult soils, such as through cobbles or boulders are problematic for cellular cofferdams and can result in driving the sheets out of interlock.
15-F-11.4 Frozen Soil Walls (Ground Freezing)

Frozen soil walls can be used for a variety of temporary shoring applications including construction of deep vertical shafts and tunneling. Frozen soil walls are typically used where conventional shoring alternatives are not feasible or have not been successful. Frozen soil walls can be constructed as gravity structures or as compressive rings. Ground freezing also provides an effective means of cutting of groundwater flows. Frozen soil has compressive strengths similar to concrete. Installation of a frozen soil wall can be completed with little vibration and can be completed around existing utilities or other infrastructure. Ground freezing is typically completed by installing rows of steel freeze pipes along the perimeter of the planned excavation. Refrigerated fluid is then circulated through the pipes at temperatures typically around -20°C to -30°C. Frozen soil forms around each freeze pipe until a continuous mass of frozen soil is present. Once the frozen soil reaches the design thickness, excavation can commence within the frozen soil.

Frozen soil walls can be completed in difficult soil and groundwater conditions where other shoring alternatives are not feasible. Frozen soil walls can provide an effective cutoff for groundwater and are well suited for containment of contaminated groundwater. Frozen soil walls are problematic in soils with rapid groundwater flows, such as coarse sands or gravels, due to the difficulty in freezing the soil. Flooding is also problematic to frozen soil walls where the flood waters come in contact with the frozen soil—a condition which can lead to failure of the shoring. Special care is required where penetrations are planned through frozen soil walls to prevent groundwater flows from flooding the excavation. Accurate installation of freeze pipes is required for deeper excavations to prevent windows of unfrozen soil. Furthermore, ground freezing can result in significant subsidence as the frozen ground thaws. If settlement sensitive structures are below or adjacent to ground that is to be frozen, alternative shoring means should be selected.

15-F-11.5 Deep Soil Mixing

Deep soil mixing (DSM) is an in-situ soil improvement technique used to improve the strength characteristics of panels or columns of native soils. DSM utilizes mixing shafts suspended from a crane to mix cement into the native soils. The result is soil mixed panels or columns of improved soils. Two types of DSM walls can be constructed: gravity walls and diaphragm-type walls. Gravity type DSM walls consist of columns or panels of improved soils configured in a pattern capable of resisting movement of soil into the excavation. Diaphragm-type DSM walls are constructed by improving the soil along the perimeter of the excavation and inserting vertical reinforcement into the improved soil immediately after mixing cement into the soil. The result is a low permeability structural wall that can be anchored with tiebacks, similar to a soldier pile wall, where the improved soil acts as the lagging.

Advantages with deep soil mixing gravity walls include the use of the native soils as part of the shoring system and reduced or no reinforcement. However, a significant volume of the native soils needs to be improved over a wide area to enable the improved soil to act as a gravity structure. Advantages with soil mixed diaphragm walls include the ability to control groundwater seepage, construction of the wall facing simultaneously with placement of steel soldier piles, and a thinner zone of improved soils compared to gravity DSM walls.
DSM walls can be installed top-down by wet methods where mechanical mixing systems combine soil with a cementitious slurry or through bottom up dry soil mixing where mechanical mixing systems mix pre-sheared soil with pneumatically injected cement or lime. DSM is generally appropriate for any soil that is free of boulders or other obstructions; although, it may not be appropriate for highly organic soils. DSM can be completed in very soft to stiff cohesive soils and very loose to medium dense granular soils.

15-F-11.6 Permeation Grouting

Permeation grouting involves the pressurized injection of a fluid grout to improve the strength of the in-situ soils and to reduce the soil’s permeability. A variety of grouts are available—micro-fine cement grout and sodium silicate grout are two of the more frequently used types in permeation grouting. To be effective, the grout must be able to penetrate the soil; therefore, permeation grouting is not applicable in cohesive soils or granular soils with more than about 20 percent fines. Disadvantages of permeation grouting is the expense of the process and the high risk of difficulties. Permeation grouting, like ground freezing or jet grouting, can be used to create gravity retaining walls consisting of improved soils or can be used to create compression rings for access shafts or other circular excavations.

In addition to characterizing the soils gradation and stratigraphy, it is important to characterize the permeability of the soils to evaluate the suitability of permeation grouting.

15-F-11.7 Jet Grouting

Jet grouting is a ground improvement technique that can be used to construct temporary shoring walls and groundwater cutoff walls. Jet grouting can also be used to form a seal or strut at the base of an excavation. Jet grouting is an erosion based technology where high velocity fluids are injected into the soil formation to break down the soil structure and to mix the soil with a cementitious slurry to form columns of improved soil. Jet grouting can be used to construct diaphragm walls to cutoff groundwater flow and can be configured to construct gravity type shoring systems or compressive rings for circular shafts. Jet grouting is applicable to most soil conditions; however, high plasticity clays or stiff to hard cohesive soils are problematic for jet grouting.

Advantages with jet grouting include the ability to use of the native soils as part of the shoring system. A significant volume of the native soils needs to be improved over a wide area to enable the improved soil to act as a gravity structure. The width of the improved soil column is difficult to control, thus the final face of a temporary shoring wall may be irregular or protrude into the excavation.

15-F-12 Factors Influencing Choice of Temporary Shoring

A multitude of factors will influence the choice of temporary shoring systems for a particular application. The most common considerations are cost, subsurface constraints (i.e. difficult driving conditions, the need to cutoff groundwater seepage, etc.), site constraints (i.e. limited access, impacts to adjacent infrastructure, etc.), and local practice. The sections below, while not all-inclusive, provide a brief discussion of several of the factors that influence selection of temporary shoring systems.
15-F-12.1 Application

The first screening criteria for alternative temporary shoring options will be the purpose of the shoring—will it retain an excavation or support a fill.

15-F-12.2 Cut/fill Height

Some retaining systems are more suitable for supporting deep excavations/fill thicknesses than others. Temporary modular block walls are typically suitable only for relatively short fill embankments (less than 15 feet), while MSE walls can be designed to retain fills several tens of feet thick.

In cut applications, the common cantilever retaining systems (sheet piling and soldier piles) are typically most cost effective for retained soil heights of 12 to 15 feet or less. Temporary shoring walls in excess of 15 feet typically require bracing, either external (struts, rakers, etc.) or internal (ground anchors or dead-man anchors).

15-F-13 Soil Conditions

15-F-13.1 Dense Soils and Obstructions

Dense subsurface conditions, such as presented by glacial till or bedrock, result in difficult installation conditions for temporary shoring systems that are typically driven or vibrated into place (sheet piling). Cobbles, boulders and debris within the soil also often present difficult driving conditions. It is often easier to use drilling methods to install shoring in these conditions. However, oversize materials and dense conditions may also hinder conventional auger drilling, resulting in the need for specialized drilling equipment. Methods such as slurry trenches and grouting may become viable in areas with very difficult driving and drilling conditions.

15-F-13.2 Caving Conditions

Caving conditions caused by a combination of relatively loose cohesionless soils and/or groundwater seepage may result in difficult drilling conditions and the need to use casing and/or drilling slurry to keep the holes open.

15-F-13.3 Permeability

Soil permeability is based primarily on the soil grain size distribution and density. It influences how readily groundwater flows through a soil. If soils are very permeable and the excavation will be below the water level, then some sort of groundwater control will be required as part of the shoring system; this could consist of traditional dewatering methods or the use of shoring systems that also function as a barrier to seepage, such as sheet piling and slurry trench methods.

15-F-13.4 Groundwater, Bottom Heave and Piping

The groundwater level with respect to the proposed excavation depth will have a substantial influence on the temporary shoring system selected. Excavations that extend below the groundwater table and that are underlain by relatively permeable soils will require either dewatering, shoring systems that also function as a barrier to groundwater seepage, or some combination thereof. If the anticipated dewatering volumes are high,
issues associated with treating and discharge of the effluent can be problematic. Likewise, large dewatering efforts can cause settlement of nearby structures if they are situated over compressible soils, or they may impact nearby contamination plumes, should they exist. Considerations for barrier systems include the depth to an aquitard to seal off groundwater flow and estimated flow velocities. If groundwater velocity is high, some barrier systems such as frozen ground and permeation grouting will not be suitable.

Bottom heave and piping can occur in soft/loose soils when the hydrostatic pressure below the base of the excavation is significantly greater than the resistance provided by the floor soils. In this case, temporary shoring systems that can be used to create a seepage barrier below the excavation, thus increasing the flow path and reducing the hydrostatic pressure below the base, may be better suited than those that do not function as a barrier. For example, sheet piling can be installed as a seepage barrier well below the base of the excavation, while soldier pile systems cannot. This is especially true if an aquitard is situated below the base of the excavation where the sheet piles can be embedded into the aquitard to seal off the groundwater flow path.

15-F-13.5 **High Locked in Lateral Stresses**

Glacially consolidated soils, especially fine-grained soils, often have high locked in lateral stresses because of the overconsolidation process (i.e. $K_o$ can be much greater than a typical normally consolidated soil deposit). The Seattle Clay is an example of this type of soil, and much has been written about the performance of cuts into this material made to construct Interstate 5 (Peck, 1963; Sherif, 1966; Andrews, et al., 1966; and Strazer, et al., 1974). When cuts are made into soils with high locked in lateral stresses, they tend to rebound upon the stress relief, which can open up joints and fractures. Hydrostatic pressure buildup in the joints and fractures can function as a hydraulic jack and move blocks of soil, and movement can quickly degrade the shear strength of the soil. Therefore, for excavations into virgin material suspected of having high locked in lateral stresses, temporary shoring methods that limit the initial elastic rebound are required. For example, anchored shoring systems that are loaded and locked-off before the excavation will likely perform better than passive systems that allow the soil move, such as soil nails.

15-F-13.6 **Compressible Soils**

Compressible soils are more likely to impact the selection of temporary walls used to retain fills. MSE walls are typically more settlement tolerant than other fill walls, such as modular block walls.

15-F-13.7 **Space Limitations**

Space limitations include external constraints, such as right-of-way issues and adjacent structures, and internal constraints such as the amount of working space required. If excavations are required near existing right-of-ways, then temporary construction easements may be required to install the shoring system. Permanent easements may be required if the shoring systems include support from ground anchors or dead-man anchors that may remain after construction is complete. To minimize the need for temporary and permanent easements, cantilever walls or walls with external bracing (e.g. struts or rakers) should be considered. However, if the work space in front of the excavation needs to be clear, then shoring systems with external support may not be appropriate.
Existing infrastructure, such as underground utilities that cannot be relocated, may have the same impact on the choice of temporary shoring system as nearby right-of-ways.

**15-F-13.8 Adjacent Infrastructure**

The location of infrastructure adjacent to the site and the sensitivity of the infrastructure to settlement and/or vibrations will influence the selection of temporary shoring. For example, it may be necessary to limit dewatering or incorporate recharge wells if the site soils are susceptible to consolidation if the water table is lowered. If the adjacent infrastructure is brittle or supported above potentially liquefiable soils, it may be necessary to limit vibrations, which may exclude the selection of temporary shoring systems that are driven or vibrated into place, such as sheet piling.

The shoring system itself could also be sensitive to adjacent soil improvement or foundation installation activities. For example, soil improvement activities such as the installation of stone columns in loose to medium dense sands immediately in front of a shoring structure could cause subsidence of the loose sands and movement, or even failure, of the shoring wall. In such cases, the shoring wall shall be designed assuming that the soil immediately in front of the wall could displace significantly, requiring that the wall embedment be deepened and ground anchors be added.

**15-F-14 References**


Appendix 15-G Testing and Acceptance Protocols for Tiebacks in Clay

Testing and Acceptance for Tiebacks Installed in Clay

The contents for this appendix are based on Allen (2020).

For tiebacks installed in intact glacially overconsolidated clay, paleolandslide deposits derived from the glacially overconsolidated clay, or otherwise disturbed glacial clay, a project specific protocol for tieback bond zone design, testing, and acceptance shall consist of the following:

- **Sacrificial pullout and sacrificial pullout with creep tests conducted on tiebacks in each soil unit in which tieback bond zones will be installed:**
  - To be able to extrapolate the pullout test results to longer bond zones, a minimum bond zone length of 4.6 m (15 ft) should be used for the test tiebacks to minimize the effect of load transfer rate nonlinearity along the bond zone soil-grout interface.
  - The testing protocol and analysis should be consistent with the protocol used for long-term tieback testing.
  - The pullout tests should be done in pairs. The first test is used to establish the values of $T_c$ and $T_{uw}$ that will be used for the second pullout test, loading the tieback incrementally until pullout is achieved, if possible. The sacrificial tieback testing schedule for this testing is provided in Tables 15-G-1 and 15-G-2.
  - The loading increments should be based on the Factored Design Load (FDL), using a load increment of 0.10FDL to 0.20FDL. A load factor of 1.35, consistent with required load factors in AASHTO (2020), should be used to determine the FDL.
  - The second pullout test is also loaded incrementally until $T_{uw}$ from the first test is achieved, at which point a 72-hour creep test is conducted. If in the second test the creep rate versus load level curve is starting to sweep upward sooner than expected, it may become necessary to use a lower value of $T_{uw}$ for the 72-hour load hold. In that case, the next increment of load increase above $T_{uw}$ may need to start lower than shown in Table 15-G-2.
  - Once the creep test is completed for the second tieback, the tieback load is increased incrementally above $T_{uw}$ until pullout is achieved, if possible.
  - At each load increment, the load level should be held for 60 minutes and creep measured.

- **Contractor designed tieback bond zone diameter and length:** The results from these sacrificial verification (pullout) tests described above, if they are successful, should be used to design the tieback bond zone length and diameter for the proposed production tieback installation method. To obtain the average bond zone soil-grout interface adhesion for the final design of the tieback bond zone, a resistance factor of 0.67 (i.e., the reciprocal of the safety factor, or $1/1.5 = 0.67$) should be applied to $T_{uw}$ determined from these pullout and extended creep tests. If the tiebacks are in disturbed glacially overconsolidated clay (e.g., paleolandslide or otherwise similar deposits), a resistance factor of 0.45 should be used to account for the increased variability of the deposit.
• **Production creep performance tests:** Five percent of the production tiebacks (or a minimum of 3 tiebacks per wall, whichever is greater) should be subjected to a creep performance test. The tieback testing schedule for this creep performance testing is provided Table 15-G-3.

• **Production cyclic performance tests, but with no longer-term creep testing:** Five percent of the production tiebacks (or a minimum of 3 tiebacks per wall, whichever is greater) should be subjected to a cyclic performance test. In cyclic performance tests the highest load tested is held for 60 minutes to determine the creep rate. The tieback testing schedule for this cyclic performance testing is provided Table 15-G-4.

• **Production proof tests conducted on all remaining tiebacks in each wall:** In proof tests the highest load tested is held for 60 minutes to determine the creep rate. The tieback testing schedule for proof testing is provided Table 15-G-5.

**Table 15-G-1** Sacrificial pullout test schedule for tiebacks in glacial clay soil units (first test)

<table>
<thead>
<tr>
<th>Load*</th>
<th>Hold Time (minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AL</td>
<td>--</td>
</tr>
<tr>
<td>0.20 FDL</td>
<td>60</td>
</tr>
<tr>
<td>0.40 FDL</td>
<td>60</td>
</tr>
<tr>
<td>0.50 FDL</td>
<td>60</td>
</tr>
<tr>
<td>0.60 FDL</td>
<td>60</td>
</tr>
<tr>
<td>0.70 FDL</td>
<td>60</td>
</tr>
<tr>
<td>0.80 FDL</td>
<td>60</td>
</tr>
<tr>
<td>0.90 FDL</td>
<td>60</td>
</tr>
<tr>
<td>1.0 FDL</td>
<td>60</td>
</tr>
<tr>
<td>1.2 FDL</td>
<td>60</td>
</tr>
<tr>
<td>1.4 FDL</td>
<td>60</td>
</tr>
<tr>
<td>1.6 FDL</td>
<td>60</td>
</tr>
<tr>
<td>1.8 FDL</td>
<td>60</td>
</tr>
<tr>
<td>2.0 FDL</td>
<td>60</td>
</tr>
</tbody>
</table>

*FDL = Factored Design Load. Failure is defined as the tieback being unable to hold the load without continued movement (pullout).
Table 15-G-2  Sacrificial pullout test schedule for tiebacks in glacial clay soil units with 72-hour creep test (second test)

<table>
<thead>
<tr>
<th>Load*</th>
<th>Hold Time (minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AL</td>
<td>--</td>
</tr>
<tr>
<td>0.20 $T_{uw}$</td>
<td>60</td>
</tr>
<tr>
<td>0.40 $T_{uw}$</td>
<td>60</td>
</tr>
<tr>
<td>0.50 $T_{uw}$</td>
<td>60</td>
</tr>
<tr>
<td>0.60 $T_{uw}$</td>
<td>60</td>
</tr>
<tr>
<td>0.70 $T_{uw}$</td>
<td>60</td>
</tr>
<tr>
<td>0.80 $T_{uw}$</td>
<td>60</td>
</tr>
<tr>
<td>0.90 $T_{uw}$</td>
<td>60</td>
</tr>
<tr>
<td>$T_{uw}$ from first test</td>
<td>4,320 (72 hrs)</td>
</tr>
<tr>
<td>1.2 FDL</td>
<td>60</td>
</tr>
<tr>
<td>1.4 FDL</td>
<td>60</td>
</tr>
<tr>
<td>1.6 FDL</td>
<td>60</td>
</tr>
<tr>
<td>1.8 FDL</td>
<td>60</td>
</tr>
<tr>
<td>2.0 FDL</td>
<td>60</td>
</tr>
</tbody>
</table>

*FDL = Factored Design Load. Failure is defined as the tieback being unable to hold the load without continued movement (pullout).

Table 15-G-3  Production tieback creep performance test schedule for tiebacks in glacial clay soil units

<table>
<thead>
<tr>
<th>*Load</th>
<th>Hold Time (minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AL</td>
<td>--</td>
</tr>
<tr>
<td>0.20 FDL</td>
<td>60</td>
</tr>
<tr>
<td>0.40 FDL</td>
<td>60</td>
</tr>
<tr>
<td>0.60 FDL</td>
<td>60</td>
</tr>
<tr>
<td>0.80 FDL</td>
<td>60</td>
</tr>
<tr>
<td>1.00 FDL</td>
<td>360 (6 hrs)</td>
</tr>
</tbody>
</table>

*Conduct on 5% of the production tiebacks in each wall, but no less than 3 tiebacks per wall.
### Table 15-G-4  Production tieback cyclic performance test schedule for tiebacks in glacial clay soil units

<table>
<thead>
<tr>
<th>Load</th>
<th>Hold Time (minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AL</td>
<td>--</td>
</tr>
<tr>
<td>0.25 FDL</td>
<td>--</td>
</tr>
<tr>
<td>AL</td>
<td>--</td>
</tr>
<tr>
<td>0.25 FDL</td>
<td>--</td>
</tr>
<tr>
<td>0.50 FDL</td>
<td>--</td>
</tr>
<tr>
<td>AL</td>
<td>--</td>
</tr>
<tr>
<td>0.25 FDL</td>
<td>--</td>
</tr>
<tr>
<td>0.50 FDL</td>
<td>--</td>
</tr>
<tr>
<td>0.75 FDL</td>
<td>--</td>
</tr>
<tr>
<td>AL</td>
<td>--</td>
</tr>
<tr>
<td>0.25 FDL</td>
<td>--</td>
</tr>
<tr>
<td>0.50 FDL</td>
<td>--</td>
</tr>
<tr>
<td>0.75 FDL</td>
<td>--</td>
</tr>
<tr>
<td>1.00 FDL</td>
<td>60</td>
</tr>
<tr>
<td>AL</td>
<td>--</td>
</tr>
<tr>
<td>Jack to lock-off load</td>
<td>--</td>
</tr>
</tbody>
</table>

*Conduct on 5% of the production tiebacks in each wall, but no less than 3 tiebacks per wall.

*If the anchor movement between 6 and 60 minutes exceeds 0.04 inches (0.03 inches for pressure- and post-grouted tiebacks), the maximum test load shall be held for an additional 300 minutes.

### Table 15-G-5  Production tieback proof test schedule for tiebacks in glacial clay soil units

<table>
<thead>
<tr>
<th>Load</th>
<th>Hold Time (minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AL</td>
<td>--</td>
</tr>
<tr>
<td>0.25 FDL</td>
<td>10</td>
</tr>
<tr>
<td>0.50 FDL</td>
<td>10</td>
</tr>
<tr>
<td>0.75 FDL</td>
<td>10</td>
</tr>
<tr>
<td>1.00 FDL</td>
<td>60</td>
</tr>
</tbody>
</table>

*Conduct on all remaining production tiebacks in each wall.

*If the anchor movement between 6 and 60 minutes exceeds 0.04 inches (0.03 inches for pressure- and post-grouted tiebacks), the maximum test load shall be held for an additional 300 minutes.
Creep test measurement times and tieback acceptance criteria shall be as provided in Table 15-G-6 for tiebacks in clay.

**Table 15-G-6**  Creep measurement times and creep criteria for tiebacks in clay soil units

<table>
<thead>
<tr>
<th>Hold Time (minutes)</th>
<th>Measurement Times (minutes)</th>
<th>Creep Criterion*</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>1, 2, 3, 4, 5, 6, and 10</td>
<td>0.75 mm/log cycle (0.03 inch per log cycle) of time</td>
</tr>
<tr>
<td>60</td>
<td>1, 2, 3, 4, 5, 6, 10, 20, 30, 40, 50, and 60</td>
<td>1.0 mm/log cycle (0.04 inch per log cycle) of time</td>
</tr>
<tr>
<td>360</td>
<td>1, 2, 3, 4, 5, 6, 10, 20, 30, 40, 50, 60, then every 30 minutes up to 360 minutes</td>
<td>1.0 mm/log cycle (0.04 inch per log cycle) of time</td>
</tr>
<tr>
<td>4320</td>
<td>1, 2, 3, 4, 5, 6, 10, 20, 30, 40, 50, 60, then every 30 minutes up to 4,320 minutes</td>
<td>*1.5 mm/log cycle (0.06 inch per log cycle) of time</td>
</tr>
</tbody>
</table>

*Adjust criterion based on test results from the sacrificial pullout tests, but no greater than shown in this table.

+ Limit to 1.0 mm/log cycle (0.04 in./log cycle) of time if testing pressure- or post-grouted tiebacks.

Use slope of creep curve (i.e., such as from a log linear regression) to determine creep rate for comparison to the creep criterion.

**Additional Implementation Requirements for Production Tieback Walls**

Based on the results of this study, the following are recommendations that should be considered when developing tieback testing programs for production walls:

1. The special testing requirements provided in this appendix should be considered applicable to tiebacks installed in overconsolidated clays, both in an intact condition and in a disturbed condition (e.g., partially reconsolidated paleolandslide deposits such as the Vashon Unsorted), in the central Puget Sound region. This testing is especially important when, for the soil surrounding the tieback bond zones, the soil consistency index is less than 0.9 and the liquid limit is greater than 50, but should also be considered for any clayey silt, silty clay, or clay. See the report conclusions (Allen 2020) for guidance regarding the soil data requirements needed to make this assessment.

2. Two sacrificial pullout/creep test tiebacks should be installed in each clay unit; one sacrificial test is for pullout and the other is for pullout and creep testing (see Tables 15-G-1 and 15-G-2). The load zone should be in the target soil unit. The verification (pullout) tests must be performed prior to production tieback installation.

3. A minimum 15-foot-long bond length is required for the sacrificial verification test tiebacks. Additional tendon steel should be added to the test tiebacks to make sure the tieback can be loaded to at least twice the FDL and high enough to achieve pullout, if possible.

4. Except for the load-cycled performance tests, no load cycling is allowed for tiebacks in glacial clay units. No retesting is allowed for tiebacks in glacial clay units.

5. If the verification (pullout) test results indicate good creep performance at the 4,320-minute hold time, a creep criterion up to 0.06 inch per log cycle of time could be considered by the Engineer for straight shafted tiebacks. For pressure-grouted and post-grouted tiebacks, a creep criterion up to 0.04 inch per log cycle of time could be considered.
6. If a tieback fails in creep, lock off the load at 50% of the load at creep failure. Additional tiebacks may be required to achieve the wall design load resistance.

7. The sacrificial verification test load-hold periods shall start as soon as the test load is applied and the anchor movement, with respect to a fixed reference, shall be measured and recorded in accordance with Table 15-G-6.

8. The maximum test load in a cyclic performance test shall be held for 60 minutes. The load-hold period shall start as soon as the maximum test load is applied and the tieback movement, with respect to a fixed reference, shall be measured and recorded in accordance with Table 15-G-6. If the anchor movement between 6 and 60 minutes exceeds 0.04 inches (0.03 inches for pressure- and post-grouted tiebacks), the maximum test load shall be held for an additional 300 minutes. If the load-hold is extended the anchor movement shall be recorded in accordance with Table 15-G-6.

9. The maximum test load in a proof test shall be held for 60 minutes. The load-hold period shall start as soon as the maximum test load is applied and the anchor movement, with respect to a fixed reference, shall be measured and recorded in accordance with Table 15-G-6. If the anchor movement between 6 and 60 minutes exceeds 0.04 inches, the maximum test load shall be held for an additional 300 minutes. If the load-hold is extended, the tieback movement shall be recorded in accordance with the 360-minute creep measurements listed in Table 15-G-6.

AL = Alignment Load, FDL = Factored Design Load

References

Appendix 15-H  Preapproved Wall Appendix: Specific Requirements and Details for Hilfiker Welded Wire Faced Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific design requirements shall be met:

No HITEC evaluation report is currently available for this wall system. Design procedures for specific elements of the wall system have been provided to WSDOT in a letter dated September 15, 2003. The design procedures used by Hilfiker Retaining Walls are in full conformance with the AASHTO Standard Specifications for Highway Bridges (2002). Interim approval is given for the continued use of the AASHTO Standard Specifications as the basis for design.

Regarding the soil reinforcement material, the minimum wire size acceptable for permanent walls is W4.5 for the longitudinal wires. For the transverse wires, the minimum wire size shall be W3.5. For all permanent walls, the welded wire shall be galvanized in accordance with the AASHTO LRFD specifications. For temporary walls, galvanization is not required, but the life of the wire shall be designed to be adequate for the intended life.

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

The maximum vertical spacing of soil reinforcement shall be 24 inches.

The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet. Larger diameter culverts or obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers.

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

The following standard details shall be used for the Hilfiker Welded Wire Faced Wall system:
Appendix 15-I  Preapproved Wall Appendix: Specific Requirements and Details for Eureka Reinforced Soil Concrete Panel Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply to the design of the Hilfiker Eureka Reinforced Soil concrete 5 feet × 5 feet panel faced retaining wall:

No HITEC evaluation report is currently available for this wall system. The design procedures used by Hilfiker Retaining Walls are based on the AASHTO Standard Specifications for Highway Bridges (2002). Therefore, for internal stability of the wall, the AASHTO Simplified Method shall be used. Interim approval is given for the continued use of the AASHTO Standard Specifications as the basis for design.

Note the connector shall be designed to have adequate life considering corrosion loss.

Furthermore, the connector loops embedded in the facing panels shall be lined up such that the steel grid reinforcement cross bar at the connection is uniformly loaded.

Therefore, regarding the alignment of the bearing surfaces of the embedded anchors, once the steel welded wire grid is inserted into the loops, no loop shall have a gap between the loop and the steel welded wire grid cross bar of more than 0.125 inch.

Reinforcement pullout shall be calculated based on the default values for steel grid reinforcement provided in the AASHTO Specifications. If, at some future time product and soil specific pullout data is provided to support use of non-default pullout interaction coefficients, it should be noted that LRFD pullout resistance design using these product and soil specific interaction coefficients has not been calibrated using product specific data statistics and reliability theory. Therefore, the specified resistance factors in the GDM and AASHTO LRFD Specifications should not be considered applicable to product specific pullout interaction coefficients.

Approved details for the Hilfiker Eureka Reinforced Soil concrete 5 feet × 5 feet panel faced retaining wall system are provided in the following plan sheets. Exceptions and additional requirements regarding the approved details are as follows:

• Regarding the filter fabric shown, WSDOT reserves the right to require the use of 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.
In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply to the design of the Reinforced Earth™ concrete 5 feet × 5 feet panel faced retaining wall:

No HITEC evaluation report is currently available for this wall system. Design procedures for specific elements of the wall system have been provided to WSDOT in a binder dated March 29, 2004. The design procedures used by RECO are based on the AASHTO Standard Specifications for Highway Bridges (2002). Internal stability is based on the use of the Coherent Gravity method per the other widely used and accepted methods clause in the AASHTO Standard Specifications. The Coherent Gravity Method should yield similar results to the AASHTO Simplified Method for this wall system. Interim approval is given for the continued use of the AASHTO Standard Specifications and the Coherent Gravity Method as the basis for design. Note the connector between the wall face panels and the soil reinforcement strips shall be designed to have adequate life considering corrosion loss as illustrated in the March 29, 2004 binder provided to WSDOT by RECO.

Reinforcement pullout shall be calculated based on the default values for steel grid reinforcement provided in the AASHTO Specifications. If, at some future time product and soil specific pullout data is provided to support use of non-default pullout interaction coefficients, it should be noted that LRFD pullout resistance design using these product and soil specific interaction coefficients has not been calibrated using product specific data statistics and reliability theory. Therefore, the specified resistance factors in the GDM and AASHTO LRFD Specifications should not be considered applicable to product specific pullout interaction coefficients.

Approved details for the Reinforced Earth™ concrete 5 feet × 5 feet panel faced retaining wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- Several plan sheets were submitted that detail panels with dimensions other than 5 feet × 5 feet. The cruciform shaped panels are also considered preapproved for use in WSDOT projects. However, unless otherwise shown in the contract, it should always be assumed that the 5 feet × 5 feet panels are intended for WSDOT projects. Other panel sizes may be used by special design (e.g., full height panels), with the approval of the State Bridge Design Engineer and the State Geotechnical Engineer, provided a complete wall design with detailed plans are developed and included in the construction contract (i.e., walls with larger facing panels shall not be submitted as shop drawings in design-bid-build projects).
• Where filter cloth or geotextile fabric is shown, WSDOT reserves the right to require the use Standard Specification materials as specified in Standard Specification Section 9-33 that are similar to those specified in this plan sheet.

• Where steel strips are skewed to avoid a backfill obstruction, the maximum skew angle shall be 15 degrees.

• The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet. Larger diameter culverts or obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers.
STANDARD PRECAST PANEL
SHOP DRAWINGS – SQUARE PANELS

**Panel General Notes**

1. Reinforcement shown shall comply to the AHS A505, Grade 60, repair layout will be detailed and shown on panel shop drawings.

2. 3/4" 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 1/4" nominal. This thickness must increase to accommodate any architectural, sculptured finish.

5. Actual location of rebar will be adjusted to accommodate panel casting.

6. Panel reinforcement shall be placed with a minimum 1 1/4" clearance from the tie strips. If mesh reinforcement is used, the tie strip location shall be adjusted to provide the minimum required clearance of 1 3/4".

7. Concrete for panels shall have a minimum compressive strength after 28 days of 4000 psi.

8. Tie strips shall be sized at least grade 50 (calculated per AHS A505).

9. Vertical reinforcement bars shall be placed 2" max. clear from the back face of the panel.

10. All rebar shores shall be stopped 7" clear from any edge of panel, unless noted on individual fabrication drawings.

11. All individual fabrication drawings are shown back face.

12. In the case of cut panels and panels with holes, additional reinforcement shown on the shop drawings shall be provided along with the news reinforcement.

13. All panels shall have two 2-ton lifting inserts, except for N and P panels. The M, N, and P panels shall have two 2-ton lifting inserts, except for N and P panels.

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Appendix 15-J  Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

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

Appendix 15-J

Geotechnical Design Manual M 46-03.13
December 2020

Page 15-J-13

The Reinforced Earth Company

DESCRIPTION
SLIP JOINT COVER PARTIAL ELEVATION W/ C.I.P. COPING SQUARE PANELS

DATE: 3/04

SHEET NO. 0020

Provide 3/4" expansion joint material
1/2" joint

See wall elevation at 1'-10"
C.I.P. TRAFFIC BARRIER
OVER SLIP JOINT COVER

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

* SEE WALL ELEVATION

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>DATE: 3/04</th>
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</thead>
<tbody>
<tr>
<td>SLIP JOINT COVER PARTIAL ELEVATION W/ BARRIER CRUCIFORM PANELS</td>
<td>SHEET NO. 0021</td>
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</table>
C.I.P. TRAFFIC BARRIER
OVER SLIP JOINT COVER

SCALE: 1/2” = 1' - 0"

* SEE WALL ELEVATION

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>DATE</th>
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<tbody>
<tr>
<td>SLIP JOINT COVER PARTIAL ELEVATION W/ BARRIER SQUARE PANELS</td>
<td>3/04</td>
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</table>

SHEET NO. 0022
TYPICAL LEVELING PAD STEP DETAIL
NOT TO SCALE

**DESCRIPTION**
SQUARE PANELS - LEVELING PAD STEP DETAIL

**DATE**
3/04

**SHEET NO.**
0024
5/8" DIA. HILTI HAS  
WITH HVA ADHESIVE ANCHOR  
6" LONG (GALV.) EMBEDED 4 1/2"  
ROD HAS 5/8" X 6"  
HVA ADHESIVE ANCHOR  
1 1/2" MIN. ±1/16"

2 PER CONNECTION ASSEMBLY  
11/16" BOLT HOLE IN ANGLE

50mm X 4mm  
REINF. STRIP  
4" x 3" x 3/8"  
3" LONG (GALV.) A36 STEEL  
2 PER CONNECTION

1/2" DIA. A325 BOLT 2" LONG  
W/WASHERS & NUT (GALVANIZED)  
9/16" Ø BOLT HOLE

CLIP ANGLE DETAIL  
SCALE: 3" = 1'-0"

<table>
<thead>
<tr>
<th>Description</th>
<th>Date</th>
<th>Sheet No.</th>
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</thead>
<tbody>
<tr>
<td>CLIP ANGLE DETAIL</td>
<td>3/04</td>
<td>0025</td>
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</tbody>
</table>
SPLICE CONNECTION DETAIL A

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

SECTION A-A

SPLICE CONNECTION DETAIL B

SCALE 1:2

NOTES:

1. SPLICE PLATE CONNECTIONS REQUIRED ON ALL REINFORCING STRIPS EXCEEDING LENGTH OF 40 FEET.

<table>
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<tr>
<th>The Reinforced Earth Company</th>
<th>DESCRIPTION</th>
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<tr>
<td></td>
<td>DOUBLE BOLTED SPLICE CONNECTION DETAIL W/ PLATES</td>
<td>SHEET NO. 0027</td>
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APPLY A LAYER OF ADHESIVE (DAP 4000) ON TOP OF LEVELING CONCRETE

CONCRETE FILL AS REQUIRED

#4 CONTINUOUS WITH 1'-0" MIN. LAP

1'-7 1/8"

8 1/8" 11'

CONTRACTOR TO FILL SPREAD ANCHOR RECESS WITH NON-SHRINK GROUT AFTER PLACEMENT

2" MIN. FINISHED GRADE AT REAR FACE OF WALL PLACE NON-SHRINK GROUT AS SHOWN

4 1/2" MIN. 3-#5 2'-5 1/2" LONG DOWELS PER PANEL. TRIM DOWELS WHERE REQUIRED TO CLEAR TOP OF LEVELING CONCRETE FILL

5 1/2"

FRONT FACE OF WALL PANEL AND HORIZ. CONTROL LINE

NOTE:
STANDARD COPING UNIT IS 10'-0" LONG.
Appendix 15-J

Geotechnical Design Manual  M 46-03.13
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PRECAST COPING FABRICATION DRAWING – TYPE 1

DATE: 3/04
SHEET NO. 0029
CONCRETE FILL AS REQUIRED

1 1/2" CLR.

2'-0"

CONTRACTOR TO FILL SPREAD ANCHOR RECESS WITH NON-SHRINK GROUT AFTER PLACEMENT

3#4 2'-0" LONG DOWELS PER PANEL. CONTRACTOR TO TRIM DOWELS WHERE REQUIRED TO CLEAR TOP OF LEVELING CONCRETE FILL

FRONT FACE OF WALL PANEL AND HORIZ. CONTROL LINE

5 1/2"

PRECAST COPING SECTION TYPE 2
SCALE: 1" = 1'-0"

NOTE:
STANDARD COPING UNIT IS 10'-0" LONG WITH SQUARE ENDS.

<table>
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<tr>
<th>The Reinforced Earth Company</th>
<th>DESCRIPTION</th>
<th>DATE :</th>
<th>SHEET NO.</th>
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<td>PRECAST COPING DETAIL - TYPE 2</td>
<td></td>
<td>3/04</td>
<td>0030</td>
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</table>
PRECAST COPING DETAIL TYPE 2

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

NOTE:
STANDARD COPING UNIT IS 10'-0"
LONG WITH SQUARE ENDS.
ALL SHORTER LENGTHS REQUIRING BEVELED ENDS SHALL BE FIELD
CUT BY THE CONTRACTOR.

The Reinforced Earth Company

DESCRIPTION
PRECAST COPING FABRICATION
DRAWING - TYPE 2

DATE: 3/04
SHEET NO. 0031
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.
1/2" x 1/2" CHAMFER

#4 AT DOWEL LOCATION

#4 FOLLOW SLOPE LINE

V-DITCH

#4 PARALLEL W/TOP OF PANEL

3 - #4 DOWELS 2'-0" LONG EMBEDDED IN PANEL. (SEE PARTIAL ELEVATION)

FRONT FACE OF WALL PANEL AND HORIZ. CONTROL LINE

5 1/2"

C.I.P. CONC. COPING W/DITCH

SCALE: 1" = 1'-0"

NOTES:
1. CONC. = 4000 psi
2. STEEL = GRADE 60
3. ALL LONGITUDINAL BARS ARE #4
4. EXPANSION JOINTS (1/2") EVERY 2 PANEL UNITS

<table>
<thead>
<tr>
<th>The Reinforced Earth Company</th>
<th>DESCRIPTION</th>
<th>C.I.P. COPING W/ DITCH DETAIL</th>
<th>DATE :</th>
<th>SHEET NO.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C.I.P. COPING W/ DITCH DETAIL</td>
<td>3/04</td>
<td>0010</td>
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December 2020
Appendix 15-J  Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls Appendix 15-L

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October 2013

Geotechnical Design Manual  M 46-03.13
December 2020

C.I.P. CONC. COPING W/FENCE

SCALE: 1" = 1'-0"

NOTES:
1. CONC. = 4000 psi
2. STEEL = GRADE 60
3. ALL LONGITUDINAL BARS ARE #4
4. EXPANSION JOINTS (1/2") EVERY 2 PANEL UNITS

The Reinforced Earth Company

DESCRIPTION
C.I.P. COPING W/ FENCE DETAIL
DATE: 3/04

SHEET NO. 0011
**NOTES:**
1. CONC. = 4000 psi
2. STEEL = GRADE 60
3. ALL LONGITUDINAL BARS ARE #4
4. EXPANSION JOINTS (1/2") EVERY 2 PANEL UNITS

**C.I.P. CONC. COPING**

**SCALE:** 1" = 1'-0"
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 8" 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.
CONC VERTICAL COPING DETAIL

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

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>DATE</th>
<th>SHEET NO.</th>
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<tbody>
<tr>
<td>C.I.P. VERTICAL COPING – TO MATCH PRECAST COPING</td>
<td>3/04</td>
<td>0014</td>
</tr>
</tbody>
</table>
CONNECTION DETAIL @ C.I.P. STRUCTURE

SCALE: 1" = 1'-0"

GEOTEXTILE FABRIC
18" WIDE
PLACED AS SHOWN
(TYPE FX-45HS
OR EQUAL)

REINFORCING STRIP

FRONT FACE OF
WALL PANEL

3/4" + 3/8" JOINT

4" MIN

9" MIN

C.I.P. STRUCTURE

VARIES

DESCRIPTION

CONN. DETAIL @ C.I.P. STRUCTURE
TYPE 1 - 4" LIP BEHIND PANEL

DATE: 3/04

SHEET NO. 0007

The Reinforced Earth Company

Geotechnical Design Manual M 46-03.13
December 2020
CONNECTION DETAIL @ C.I.P. STRUCTURE

DESCRIPTION
CONN. DETAIL @ C.I.P. STRUCTURE
TYPE 2 - 4" LIP IN FRONT OF PANEL

DATE: 3/04
SHEET NO. 0008
COPING ENCLOSURE DETAIL

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

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

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OBTUSE CORNER ELEMENT DETAIL

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

* THREE BEARING PADS PER UNIT, BASE STEM OF BEARING PAD SHALL BE FIELD CUT TO FIT FLAT ON TOP OF CORNER ELEMENT. FRONT PADS SHALL BE PLACED ON INSIDE EDGE OF LIP.
90° CORNER ELEMENT DETAIL

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

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

The Reinforced Earth Company

DESCRIPTION

90° CORNER ELEMENT DETAIL

DATE:

3/04

SHEET NO.

0005
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<td>3/04</td>
<td>0002</td>
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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.
PARTIAL WALL PLAN AT LIGHT POLE

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

The Reinforced Earth Company

PARTIAL PLAN AT LIGHT POLE

DATE: 3/04

SHEET NO. 0036
Proposed Obstruction

1" (±) Clearance (Typical)

Reinforcing Strip (Typical)

Skew Strip to Either Side of Obstruction as Shown. Keep Skew Angle to a Minimum.

Tie Strip (Typical)

Front Face of Wall Panels

Partial Wall Plan at Obstruction

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

The Reinforced Earth Company

Description: Partial Plan at General Obstruction

Date: 3/04

Sheet No.: 0037
PIECE PENETRATION AT WALL FACE DETAIL
(TREATMENT FOR CONCRETE PIPE SHOWN, CORRUGATED STEEL PIPE SIMILAR)

SCALE: 1" = 1'–0"
Appendix 15-K  Preapproved Wall Appendix: Specific Requirements and Details for Tensar ARES Walls

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

The detailed design methodology, design properties, and assumptions used by Tensar Earth Technologies for the ARES wall are summarized in the HITEC evaluation report for this wall system (HITEC, 1997, Evaluation of the Tensar ARES Retaining Wall System, ASCE, CERF Report No. 40301). The design methodology, which is based on the Standard Specifications for Highway Bridges (2002) is consistent with the general design requirements in Appendix 15-A, except as noted below. Interim approval is given for the continued use of the AASHTO Standard Specifications as the basis for design.

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications. For LRFD based design, while it is recognized that product and soil type specific pullout interaction coefficients obtained in accordance with the AASHTO LRFD Specifications for the Tensar products used with this wall system are provided in the HITEC report for the ARES Wall system, pullout resistance design using these product and soil specific interaction coefficients has not been calibrated using the available product specific data statistics and reliability theory. Therefore, the specified resistance factors in the GDM and AASHTO LRFD Specifications should not be considered applicable to the product specific pullout interaction coefficients provided in the HITEC report.

The reinforcement long-term tensile strengths (T_{al}) provided in the WSDOT Qualified Products List (QPL) for the Tensar Geogrid product series, which are based on the 2003 version of the product series, shall be used for wall design, until such time that they are updated, and the updated strengths approved for WSDOT use in accordance with WSDOT Standard Practice T 925. Until such time that the long-term reinforcement strengths are updated, it shall be verified that any material sent to the project site for this wall system is the 2003 version of the product. Furthermore, the short-term ultimate tensile strengths (ASTM D6637) listed in the QPL shall be used as the basis for quality assurance testing and acceptance of the product as shipped to the project site per the Standard Specifications for Construction.

The HITEC report provided details and design criteria for a panel slot connector to attach the geogrid reinforcement to the facing panel. Due to problems with cracking of the facing panel at the location of the slot, that connection system has been discontinued and replaced with a full thickness panel in which geogrid tabs have been embedded into the panel. For this new connection system, the geogrid reinforcement is connected to the geogrid tab through the use of a Bodkin joint. Construction and fabrication inspectors should verify that the panels to be used for WSDOT projects do not contain the discontinued slot connector.
The Bodkin connection test results provided by letter to WSDOT dated September 28, 2004, were performed on the 2003 version of the Tensar geogrid product line. In that letter, it was stated that UMESA6 (UX1700HS) will typically be used for the connector tabs, regardless of the product selected for the reinforcement. If a lighter weight product is used for the connector tabs, the connection strength will need to be reduced accordingly. Table 15-(Tensar ARES)-1 provides a summary of the connection strengths that are approved for use with the ARES wall system.

Table 15-K-1  Approved Connection Strength Design Values for Tensar Ares Walls

<table>
<thead>
<tr>
<th>Tensar Soil Reinforcement Geogrid Product</th>
<th>Tensar Panel Connector Tab Geogrid Product</th>
<th>( T_{\text{ult}} ) (MARV) for Geogrid Reinforcement per ASTM D6637 in WSDOT QPL (lbs/ft)</th>
<th>( CR_u )</th>
<th>RF</th>
<th>( T_{ac} ) (lbs/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UMESA3/UX1400HS</td>
<td>UMESA6/UX1700HS</td>
<td>4,820</td>
<td>1.0</td>
<td>3.6</td>
<td>1,340</td>
</tr>
<tr>
<td>UMESA4/UX1500HS</td>
<td>UMESA6/UX1700HS</td>
<td>7,880</td>
<td>1.0</td>
<td>3.5</td>
<td>2,250</td>
</tr>
<tr>
<td>UMESA5/UX1600HS</td>
<td>UMESA6/UX1700HS</td>
<td>9,870</td>
<td>1.0</td>
<td>3.4</td>
<td>2,900</td>
</tr>
<tr>
<td>UMESA6/UX1700HS</td>
<td>UMESA6/UX1700HS</td>
<td>12,200</td>
<td>0.91</td>
<td>3.3</td>
<td>3,360</td>
</tr>
<tr>
<td>UMESA3/UX1400HS</td>
<td>UMESA3/UX1400HS</td>
<td>4,820</td>
<td>0.85</td>
<td>3.6</td>
<td>1,140</td>
</tr>
<tr>
<td>UMESA4/UX1500HS</td>
<td>UMESA4/UX1500HS</td>
<td>7,880</td>
<td>0.79</td>
<td>3.5</td>
<td>1,780</td>
</tr>
<tr>
<td>UMESA5/UX1600HS</td>
<td>UMESA5/UX1600HS</td>
<td>9,870</td>
<td>0.87</td>
<td>3.4</td>
<td>2,530</td>
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<tr>
<td>UMESA6/UX1700HS</td>
<td>UMESA6/UX1700HS</td>
<td>12,200</td>
<td>0.91</td>
<td>3.3</td>
<td>3,360</td>
</tr>
</tbody>
</table>

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

\[
T_{ac} = \frac{T_{\text{MARV}} \cdot CR_u}{RF}
\]  \hspace{1cm} (15-(Tensar ARES)-1)

Where:
- \( RF = RF_{ID} \times RF_{CR} \times RF_{D} \)
- \( T_{\text{MARV}} \) = The minimum average roll value for the ultimate geosynthetic strength \( T_{\text{ult}} \)
- \( CR_u \) = The ultimate connection strength \( T_{\text{ultconn}} \) divided by the lot specific ultimate tensile strength, \( T_{\text{lot}} \) (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 10º.

- In plan sheets 10 and 14 of 19, regarding the filter fabric shown, WSDOT reserves the right to require the use Standard Specification materials as specified in Standard Specification Section 9-33 that are similar to those specified in this plan sheet.

- In plan sheet 15 of 19, the guard rail detail, the guard rail post shall either be installed through precut holes in the geogrid layers that must penetrated, or the geogrid layers shall be cut in a manner that prevents ripping or tearing of the geogrid.

- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 2 feet for culvert penetration through the face and up to 4 feet for obstruction avoidance. Larger diameter culverts or obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers.
STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS

<table>
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<td>Geogrid Panel Connection</td>
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<td>ARES Retaining Wall Construction Requirements</td>
<td>10.</td>
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<td>ARES Articulated Panel Levelling Pod</td>
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<tr>
<td>3.</td>
<td>ARES Articulated Panel Cross-Section</td>
<td>11.</td>
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<td>ARES Full Height Panel Levelling Pod</td>
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<tr>
<td>4.</td>
<td>ARES Full Height Panel Cross-Section</td>
<td>12.</td>
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<td>Panel Coping</td>
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<td>5.</td>
<td>ARES Articulated Panels</td>
<td>13.</td>
<td></td>
<td>Panel Coping</td>
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<td>6.</td>
<td>ARES Full Height Panels</td>
<td>14.</td>
<td></td>
<td>Obstructions</td>
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<td>Typical Details</td>
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<td>8.</td>
<td>Geogrid Panel Connection</td>
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<td></td>
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<tr>
<td></td>
<td></td>
<td>17.</td>
<td></td>
<td>32 Inch Type &quot;F&quot; Traffic Barrier Standard</td>
</tr>
<tr>
<td></td>
<td></td>
<td>18.</td>
<td></td>
<td>42 Inch Type &quot;F&quot; Traffic Barrier Standard</td>
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<tr>
<td></td>
<td></td>
<td>19.</td>
<td></td>
<td>42 Inch Type &quot;F&quot; Traffic Barrier Standard</td>
</tr>
</tbody>
</table>
Appendix 15-K Preapproved Wall Appendix: Specific Requirements and Details for Tensar ARES Walls

Panel Connection Detail - Section (A-A)

To form a panel body connection for connections to facing panel:

1. Bend the last aperture of reinforcing geogrid as shown.

2. Pass the bend of the last aperture through the ribs of the geogrid tail and under the bottom bar into the space between the two geogrid layers.

3. Pull reinforcing geogrid to tension connection, complete connection.

Panel Connection - Bookin Detail

Articulated Panel Connection Detail - Plan View

Full Height Panel Connection Detail - Plan View
Appendix 15-L  Preapproved Wall Appendix: Specific Requirements and Details for Tensar MESA Walls

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

The detailed design methodology, design properties, and assumptions used by Tensar Earth Technologies for the MESA wall are summarized in the HITEC evaluation report for this wall system (HITEC, 2000, Evaluation of the Tensar MESA Wall System, ASCE, CERF Report No. 40358). The design methodology, which is based on the Standard Specifications for Highway Bridges (2002) is consistent with the general design requirements in Appendix 15-A, except as noted below. Interim approval is given for the continued use of the AASHTO Standard Specifications as the basis for design.

Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2 feet. Regarding horizontal spacing of reinforcement strips (i.e., rolls), reinforcement coverage ratios of greater than 0.7 are acceptable for this wall system. This is based on having a maximum of one facing block between reinforcement rolls, as allowed by the AASHTO Specifications.

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications. For LRFD based design, while it is recognized that product and soil type specific pullout interaction coefficients obtained in accordance with the AASHTO LRFD Specifications for the Tensar products used with this wall system are provided in the HITEC report for the MESA Wall system, pullout resistance design using these product and soil specific interaction coefficients has not been calibrated using the available product specific data statistics and reliability theory. Therefore, the specified resistance factors in the GDM and AASHTO LRFD Specifications should not be considered applicable to the product specific pullout interaction coefficients provided in the HITEC report.

The reinforcement long-term tensile strengths ($T_{al}$) provided in the Qualified Products List (QPL) for the Tensar Geogrid product series, which are based on the 2003 version of the product series, shall be used for wall design, until such time that they are updated, and the updated strengths approved for WSDOT use in accordance with WSDOT Standard Practice T 925. Until such time that the long-term reinforcement strengths are updated, it shall be verified that any material sent to the project site for this wall system is the 2003 version of the product. Furthermore, the short-term ultimate tensile strengths (ASTM D6637) listed in the QPL shall be used as the basis for quality assurance testing and acceptance of the product as shipped to the project site per the Standard Specifications for Construction.
The HITEC report provided connection data for the DOT³ system and the HP System. Both systems provide partial connection coverage, with the DOT³ system only providing 14 teeth per 21 openings, and the HP System providing 17 teeth per 21 openings. The DOT³ 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, CRₜₚ, 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 Tac, as defined in the AASHTO LRFD Specifications, assuming a durability reduction factor of 1.1.

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<tbody>
<tr>
<td>UMESA3</td>
<td>4400</td>
<td>4820</td>
<td>0.79</td>
<td>0.72</td>
<td>2.6</td>
<td>0.28</td>
<td>1200</td>
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<tr>
<td>UMESA4</td>
<td>6850</td>
<td>7880</td>
<td>0.73</td>
<td>0.63</td>
<td>2.6</td>
<td>0.24</td>
<td>1720</td>
</tr>
<tr>
<td>UMESA5</td>
<td>9030</td>
<td>9870</td>
<td>0.80</td>
<td>0.73</td>
<td>2.6</td>
<td>0.28</td>
<td>2510</td>
</tr>
<tr>
<td>UMESA6</td>
<td>10,700</td>
<td>12200</td>
<td>0.75</td>
<td>0.66</td>
<td>2.6</td>
<td>0.25</td>
<td>2770</td>
</tr>
</tbody>
</table>

*i.e., to get same Tultconn value as in HITEC report.

Tₚₚ, the long-term connection strength, shall be calculated as follows:

\[ T_{ac} = \frac{T_{MARV} \cdot CR_{u}}{RF_{CR} \cdot RF_{D}} \]  \hspace{0.5cm} (15-L-1)

where,

- \( T_{MARV} \) = the minimum average roll value for the ultimate geosynthetic strength \( T_{ult} \),
- \( CR_{u} \) = the ultimate connection strength \( T_{ultconn} \) divided by the lot specific ultimate tensile strength, \( T_{lot} \) (i.e., the lot of material specific to the connection testing),
- \( RF_{CR} \) = creep reduction factor for the geosynthetic, and
- \( RF_{D} \) = the durability reduction factor for the geosynthetic.
Since the HITEC report was developed, Tensar Earth Technologies has developed a new connector that provides, for the most part, a full coverage connector, providing 19 teeth per 21 openings. Short-term connection tests on the strongest geogrid product in the series shows that connection strengths higher than those obtained with the HP System will be obtained with the new connector, which is called the DOT system (note that the 3 has been dropped – this is not the same as the DOT$^3$ system). This new DOT System may be used, provided that the values for $T_{ac}$ shown in Table 15-(Tensar MESA)-1 are used for design, which should be conservative, until a more complete set of test results are available. Photographs illustrating the new DOT connector system are provided in Figures 15-(Tensar MESA)-1 through 15-(Tensar MESA)-3.

The longitudinal (i.e., in the direction of loading) and transverse (i.e., parallel to the wall or slope face) ribs that make up the geogrid shall be perpendicular to one another. The maximum deviation of the cross-rib from being perpendicular to the longitudinal rib (skew) shall be manufactured to be no more than 1 inch in 5 feet of geogrid width. The maximum deviation of the cross-rib at any point from a line perpendicular to the longitudinal ribs located at the cross-rib (bow) shall be 0.5 inches.

The gap between the connector tabs and the bearing surface of the geogrid reinforcement cross-rib shall not exceed 0.5 inches. A maximum of 10% of connector tabs may have a gap between 0.3 inches and 0.5 inches. Gaps in the remaining connector tabs shall not exceed 0.3 inches.

Concrete for dry cast concrete blocks used in the Tensar MESA wall system shall meet the following requirements:

1. Have a minimum 28 day compressive strength of 4,000 psi.
2. Conform to ASTM C1372.
3. The lot of blocks produced for use in this project shall conform to the following freeze-thaw test requirements when tested in accordance with ASTM C 1262:
   • Minimum acceptable performance shall be defined as weight loss at the conclusion of 150 freeze-thaw cycles not exceeding one percent of the block's initial weight for a minimum of four of the five block specimens tested.
4. The concrete blocks shall have a maximum water absorption of one percent above the water absorption content of the lot of blocks produced and successfully tested for the freeze-thaw test specified in the preceding paragraph.

It is noted in ASTM C1372 that a dimensional tolerance for the height of the block of ¼ 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 ⅛ 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 ⅛ inch to reduce the risk of significant cracking of facing blocks.
Figure 15-L-1  MESA DOT System Connector and Block

Figure 15-L-2  MESA DOT System Connector and Block as Assembled
Block connectors for block courses with geogrid reinforcement shall be glass fiber reinforced high-density polypropylene conforming to the following minimum material specifications:

<table>
<thead>
<tr>
<th>Property</th>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polypropylene</td>
<td>ASTM D 4101 Group 1 Class 1 Grade 2</td>
<td>73 ± 2 percent</td>
</tr>
<tr>
<td>Fiberglass Content</td>
<td>ASTM D 2584</td>
<td>25 ± 3 percent</td>
</tr>
<tr>
<td>Carbon Black</td>
<td>ASTM D 4218</td>
<td>2 percent minimum</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>ASTM D 792</td>
<td>1.08 ± 0.04</td>
</tr>
<tr>
<td>Tensile Strength at yield</td>
<td>ASTM D 638</td>
<td>8,700 ± 1,450 psi</td>
</tr>
<tr>
<td>Melt Flow Rate</td>
<td>ASTM D 1238</td>
<td>0.37 ± 0.16 ounces/10 min.</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Property</th>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>HDPE</td>
<td>ASTM D 1248 Group 3 Class 1 Grade 5</td>
<td>68 ± 3 percent</td>
</tr>
<tr>
<td>Fiberglass Content</td>
<td>ASTM D 2584</td>
<td>30 ± 3 percent</td>
</tr>
<tr>
<td>Carbon Black</td>
<td>ASTM D 4218</td>
<td>2 percent minimum</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>ASTM D 792</td>
<td>1.16 ± 0.06</td>
</tr>
<tr>
<td>Tensile Strength at yield</td>
<td>ASTM D 638</td>
<td>8,700 ± 725 psi</td>
</tr>
<tr>
<td>Melt Flow Rate</td>
<td>ASTM D 1238</td>
<td>0.11 ± 0.07 ounces/10 min.</td>
</tr>
</tbody>
</table>
Approved details for the Tensar MESA wall system with the DOT System connector are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- In plan sheet 5 of 13, the guard rail detail, the guard rail post shall either be installed through precut holes in the geogrid layers that must penetrated, or the geogrid layers shall be cut in a manner that prevents ripping or tearing of the geogrid.

- In plan sheets 4, 6, and 8 of 13, regarding the geotextiles and drainage composites shown, WSDOT reserves the right to require the use Standard Specifications materials as specified in Standard Specification Section 9-33 that are similar to those specified in this plan sheet.

- In plan sheet 7 of 13, regarding the geogrid at wall corner detail, cords in the wall facing alignment to form an angle point or a radius shall be no shorter than the width of the roll to insure good contact between the connectors and the geogrid cross-bar throughout the width of the geogrid. Alternatively, the geogrid roll could be cut longitudinally in half to allow a tighter radius, if necessary.

- In plan sheet 7 of 13, regarding the typical geogrid percent coverage, the maximum distance X between geogrid strips shall be one block width. Therefore, the minimum percent coverage shall be 73 percent.

- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 2 feet for culvert penetration through the face and up to 4 feet for obstruction avoidance. Larger diameter culverts or obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers.
# STANDARD MESA DETAILS & CONSTRUCTION NOTES

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</table>

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

**STATE OF WASHINGTON**

**DEPARTMENT OF TRANSPORTATION**

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

**WSDOT Geotechnical Design Manual  M 46-03.08**

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

**WSDOT Geotechnical Design Manual  M 46-03.13**

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**October 2013**
Appendix 15-L
Preapproved Wall Appendix: Specific Requirements and Details for Tensar MESA Walls

**Fabric Drainage Specification**

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

All eight (8) oz per square yard non-woven needle punched geotextile, ACHTI W28B CLASS 1, shall be used on the filter blanket.

Unless otherwise permitted by Engineer, the geotextile shall be delivered to the project site in rolls that have been factory cut to the specified widths for the installation.

**Installation Procedure**

1. Install each course of the mesa facing units between the top of the last geosynthetic placed and the bottom of the next layer of geosynthetic drainage as shown on the approved construction drawings. The facing units shall be aligned and seated in accordance with the wall installation guide.

2. Prior to placing the select fill, the geotextile shall be placed behind the units such that a minimum of six (6) inches of material is placed into the fill at the top and the bottom. The geotextile shall then be adjusted to prevent a visibly smooth surface.

3. The select fill shall then be placed and compacted in accordance with the approved construction drawings and project specifications.

4. After the select fill has been compacted and properly graded for the installation of the next layer of geosynthetic reinforcement, the geosynthetic on the top units will remain in position on the facing unit (see Stage 6 detail) or be pushed back onto the bottom (contractor's option).

5. Install the geosynthetic reinforcement and repeat the process commencing with item 1.

6. After the last level of primary geosynthetic reinforcement has been placed, install the remaining courses of mesas facing units except for the last standard course and the cap units, in accordance with the details on the approved construction drawings.

7. Place a line of an approved construction adhesive along the top of the mesa standard units approximately one (1) inch behind the face as shown in Detail 51.

8. Place the eight (8) oz geotextile such that the leading edge of the material is approximately 1/2 inch behind the face and press into the adhesive. The bottom of the geotextile shall extend a minimum of six (6) inches into the select fill. Allow adhesive to obtain an initial set for approximately 20 minutes (Detail 51).

9. Install the geosynthetic connectors in the area as shown in the wall installation guide. Connectors will push the faces into the side as shown in the Typical Section, this sheet.

10. Install the last course of mesa standard units and cap and level as required in the installation guide.

11. Place a line of adhesive in the depressed areas between the connector slots and the face of the unit per Detail 52.

12. Install the eight (8) oz geotextile such that the leading edge of the material just contacts the edge of the depressed area as shown in the Typical Section, this sheet. The bottom of the geotextile shall extend a minimum of six (6) inches beyond the geotextile in the mesa standard unit.

13. Place a line of adhesive along the top of the standard units just behind the face per Detail 53.

14. Install the cap units as shown in Detail 54.

Geotextile widths required for the details:

**ACHTI W28B Class 1:** 35 inch
Appendix 15-M  Preapproved Wall Appendix: Specific Requirements and Details for Tensar Welded Wire Form Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific design requirements shall be met:

No HITEC evaluation report is currently available for this wall system. Design procedures for specific elements of the wall system have been provided to WSDOT in a submittal dated May 20, 2005, and final Wall Details submitted May 26, 2005. The design procedures used by Tensar Earth Technologies (TET) are in full conformance with the AASHTO LRFD Bridge Design Specifications (2004).

This wall system consists of Tensar geogrid reinforcement that is connected to a welded wire facing panel. Regarding the welded wire facing panel, the minimum wire size acceptable for permanent walls is W4.5, and the welded wire shall be galvanized in accordance with the AASHTO LRFD specifications. The actual wire size submitted is W4.0. The exception regarding the wire size is allowed. Due to the smaller wire size, there is some risk that the welded wire form will not provide the full 75 year life required for the wall. Therefore, to insure internal stability of the wall, the geogrid reinforcement shall be wrapped fully behind the face to add the redundancy needed to insure the wall face system is stable for the required design life. The galvanization requirement for the welded wire form still applies, however, as failure of the welded wire form at some point during the wall design life could allow some local sagging of the wall face to occur. The minimum clear opening dimension of the facing panel, or backing mat if present, shall not exceed the minimum particle size of the wall facing backfill. The maximum particle size for the wall facing backfill shall be 4 inches. The maximum vertical spacing of soil reinforcement shall be 18 inches for vertical and battered wall facings.

The geogrid tensile strengths used for design for this wall system shall be aslisted in the WSDOT Qualified Products List (QPL).

The Bodkin connection shown in the typical cross-section (page 15-(Tensar WW)-1) may be used subject to the following conditions:

- No more than one Bodkin connection may be used within any given layer, and 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.
Table 15-M-1 Approved Bodkin Connection Strength Reduction Factors for Tensar Welded Wire Form Walls

<table>
<thead>
<tr>
<th>Tensar Primary Soil Reinforcement Geogrid Product</th>
<th>Tensar Product to Which Soil Reinforcement is Connected</th>
<th>Connection Strength Reduction Factor, CR_u</th>
</tr>
</thead>
<tbody>
<tr>
<td>UMESA3/UX1400HS</td>
<td>UMESA6/UX1700HS</td>
<td>1.0</td>
</tr>
<tr>
<td>UMESA4/UX1500HS</td>
<td>UMESA6/UX1700HS</td>
<td>1.0</td>
</tr>
<tr>
<td>UMESA5/UX1600HS</td>
<td>UMESA6/UX1700HS</td>
<td>1.0</td>
</tr>
<tr>
<td>UMESA6/UX1700HS</td>
<td>UMESA6/UX1700HS</td>
<td>0.91</td>
</tr>
<tr>
<td>UMESA3/UX1400HS</td>
<td>UMESA3/UX1400HS</td>
<td>0.85</td>
</tr>
<tr>
<td>UMESA4/UX1500HS</td>
<td>UMESA4/UX1500HS</td>
<td>0.79</td>
</tr>
<tr>
<td>UMESA5/UX1600HS</td>
<td>UMESA5/UX1600HS</td>
<td>0.87</td>
</tr>
<tr>
<td>UMESA6/UX1700HS</td>
<td>UMESA6/UX1700HS</td>
<td>0.91</td>
</tr>
</tbody>
</table>

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

- Though not shown in the approved plan sheets, if guard rail is to be placed at the top of the wall, the guard rail post shall either be installed through precut holes in the geogrid layers that must penetrated, or the geogrid layers shall be cut in a manner that prevents ripping or tearing of the geogrid.

- In plan sheets on pages 3, 4, 5, and 13, regarding the geotextiles shown, WSDOT reserves the right to require the use Standard Specification materials as specified in Standard Specification Section 9-33 that are similar to those specified in this plansheet.

- Regarding the plantable face alternate plan details on page 6, this 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.
TYPICAL CROSS-SECTION

DESCRIPTION: TYPICAL CROSS-SECTION
FILE NAME: WWFSS6.DWG

Tensar Earth Technologies Inc.
NOTES:
1. SEE WELDED WIRE FACING UNIT DETAIL FOR MATERIAL AND DIMENSIONS.
2. ALL FACING UNITS SHALL BE GALVANIZED AS PER ASTM A123 AFTER FABRICATION.
3. OPTIONAL THIN LAYER OF FINER STONE MAY BE PLACED AT THE TOP OF EACH UNIT TO PROVIDE A LEVEL SURFACE FOR THE UNIT ABOVE.

ALTERNATE WELDED WIRE FACING DETAIL
NOT TO SCALE

Tensar Earth Technologies Inc.
Appendix 15-M

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

NOTES:
1. See Welded Wire Facing Unit Detail for material and dimensions.
2. All facing units shall be galvanized as per ASTM A123 after fabrication.

ALTERNATE WELDED WIRE FACING DETAIL (1” - 2” FACE FILL)

DESCRIPTION: ALTERNATE SIERRASCAP FACING DETAIL (1”-2” FACE FILL)
FILE NAME: WWFS52x023035.DWG

Tensar Earth Technologies Inc.

TYPICAL DETAIL
LIMIT OF PLANTABLE FILL SHALL NOT EXTEND BENEATH THE WELDED WIRE FORM FACING UNIT ABOVE.

WELDED WIRE FORM FACING UNIT

36" (MIN.) TOP AND BOTTOM

3" (MIN. TRM LENGTH EXTENDING BENEATH THE WELD WIRE FORM FACING UNIT ABOVE)

NORTH AMERICAN GREEN P300 TRM

6" MIN. BOTTOM WRAP OF TRM

PLANTABLE FILL (TOP SOIL) (SEE NOTE 3)

TENSAR UNIAXIAL GEORGRID IN ACCORDANCE WITH ELEVATION VIEW

TENSAR BX1120 GEORGRID

SUPPORT STRUT

REINFORCED FILL

NOTES:
1. SEE WELDED WIRE FORM FACING UNIT DETAIL FOR FACING MATERIAL AND DIMENSIONS.
2. FACING UNITS SHALL BE CONSTRUCTED FROM BLACK STEEL.
3. PLANTABLE FILL OR TOP SOIL MAY BE PLACED AT THE FACE TO SUPPORT VEGETATION GROWTH.

ALTERNATE WELDED WIRE FORM FACING DETAIL (PLANTABLE FACE FILL)

NOT TO SCALE

DESCRIPTION: SIERRASCAPES FACING DETAIL
FILE NAME: WWFSS.3a2030A.DWG

Tensar Earth Technologies Inc.

TYPICAL DETAIL
NOTES:
1. FACING TO CONSIST OF PREFABRICATED WWF 4x4-W4.0xW4.0 FORMS.
2. ALL FORMS SHALL BE GALVANIZED PER ASTM A123 AFTER BENDING WHEN REQUIRED.
3. OVERALL LENGTH OF WIRE FORMS IS 10'-0". EFFECTIVE CONSTRUCTED WIDTH IS 9'-8" WITH 4" OVER LAPPING AT ENDS.
4. STRUT LENGTH AND CROSS-SECTIONAL FORM DIMENSIONS TO BE PROVIDED IN FABRICATORS SHOP DRAWINGS.

NOT TO SCALE

FILE NAME: WWF.DWG
NOTES:
BEND OR CUT BASKETS TO FIT FIELD CONDITIONS
ENSURE THAT GEOTEXTILE AND BIAXIAL GEGRID OVERLAP 1’ MINIMUM

90° OUTSIDE CORNER DETAIL
NOT TO SCALE
90° INSIDE CORNER DETAIL

NOTE:
BEND, BUTT OR CUT BASKETS TO FIT FIELD CONDITIONS

TIE BASKETS

90°

TENSAR

TYPICAL DETAIL

DESCRIPTION: INSIDE CORNER DETAIL
FILE NAME: WWFD02.DWG
MINIMUM 3" OF SOIL BETWEEN OVERLAPPING LAYERS OF GEOGRID

FRONT FACE

TRIM GEOGRID AT FACE WHERE NECESSARY

GEOGRID PLACEMENT ON CURVES
NOT TO SCALE

Tensar Earth Technologies Inc.

TYPICAL DETAIL

DESCRIPTION: GEOGRID PLACEMENT ON CURVES
FILE NAME: TENSAR2.DWG
WALL CORNER DETAIL
NOT TO SCALE

3" SOIL FILL REQUIRED BETWEEN OVERLAPPING GEOGRIDS FOR PROPER ANCHORAGE

FRONT FACE OF WALL
GEOGRID 90° CORNER DETAIL
NOT TO SCALE

DESCRIPTION: GEOGRID 90° CORNER DETAIL
FILE NAME: GPW93.DWG
ELEVATION VIEW

PLAN VIEW

NOTES:
1. CUT WIRE FACING AS CLOSE AS POSSIBLE TO PIPE PENETRATION.
2. CUT OR TERMINATE GEOGRIDS 3 INCHES OR LESS FROM PIPE.
3. WRAP ENTIRE PIPE WITH AASHTO M288 CLASS 3 NON-WOVEN DRAINAGE GEOTEXTILE. ENSURE THAT WRAP EXTENDS AT LEAST 12 INCHES BEHIND WIRE FACING AT PENETRATION TO ENSURE NO LOSS OF FILL.
4. FOR GEOGRID LAYOUT REFER TO ELEVATION VIEW FOR LENGTH, TYPE AND LOCATION

PIPE PENETRATION DETAIL AT WELDED WIRE FACE SYSTEM
NOT TO SCALE

Tensar Earth Technologies Inc.

TYPICAL DETAIL
GEOGRID PLACEMENT AT PIPE

NOT TO SCALE

PIPE

2x PIPE DIAMETER (MIN.)

SPACE EQUAL DISTANCE APART 3" (MIN.)

SOIL COVER BETWEEN GEOGRIDS

GEOGRID (TYP.)

3" (MIN.) SOIL COVER BETWEEN PIPE & GEOGRID

DESCRIPTION: GEOGRID PLACEMENT AT PIPE

FILE NAME: GPP1.DWG

Tensar Earth Technologies Inc.

TYPICAL DETAIL
CUT OPENING IN GEOGRID A MAX. OF 2” LARGER THAN VERTICAL STRUCTURES

30.0” (MAX.)

3.0’ (MIN.)

FRONT FACE OF WIRE FORM WALL

NOTE:
FOR OTHER CONDITIONS APPLY THE PROVISIONS OF ARTICLE 11.10.10.4 OF AASHTO LRFD SPECIFICATIONS.

GEOGRID PENETRATION
NOT TO SCALE
TO FORM A BODKIN CONNECTION:

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


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
Appendix 15-N Preapproved Wall Appendix: Specific Requirements and Details for SSL Concrete Panel Walls

In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply to the design of the SSL MSE Plus™ Retaining Wall:

The welded wire steel soil reinforcement shall be comprised of W11, W20, or W24 smooth wire as shown and noted in the preapproved SSL MSEPlus wall system drawings. Deformed bars shall not be used for soil reinforcement. As SSL has committed to always supply soil reinforcement steel with a minimum yield strength of 75 ksi, the soil reinforcement steel shall be designed for a yield strength, $F_y$, 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 $C_{Ru}$ shall be used:

<table>
<thead>
<tr>
<th>Welded Wire Soil Reinforcement Wire Size</th>
<th>Short-Term Connection Strength Ratio, $C_{Ru}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>W11</td>
<td>0.98</td>
</tr>
<tr>
<td>W20</td>
<td>0.87</td>
</tr>
<tr>
<td>W24</td>
<td>0.96</td>
</tr>
</tbody>
</table>

Minimum bend radii for the welded wire soil reinforcement shall be as shown in the preapproved plans (sheet 4 of 15 titled “Standard Details 3 of 3”).

Reinforcement pullout shall be calculated based on the default values for steel grid reinforcement provided in the AASHTO Specifications. If, at some future time product and soil specific pullout data is provided to support use of non-default pullout interaction coefficients, it should be noted that LRFD pullout resistance design using these product and soil specific interaction coefficients has not been calibrated using product specific data statistics and reliability theory. Therefore, the specified resistance factors in the GDM and AASHTO LRFD Specifications should not be considered applicable to product specific pullout interaction coefficients.
Approved details for the SSL MSE PlusTM wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- In plan sheet 4 of 10, regarding the filter fabric shown, the use of 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.
Appendix 15-N: Preapproved Wall Appendix: Specific Requirements and Details for SSL Concrete Panel Walls

**Panel Reinforcement Table**

<table>
<thead>
<tr>
<th>Panel Type</th>
<th>Minimum Vert. Area</th>
<th>Minimum Horz. Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.88 in²</td>
<td>0.99 in²</td>
</tr>
<tr>
<td>A2</td>
<td>0.99 in²</td>
<td>0.53 in²</td>
</tr>
<tr>
<td>X</td>
<td>1.40 in²</td>
<td>1.40 in²</td>
</tr>
<tr>
<td>X2</td>
<td>1.60 in²</td>
<td>0.77 in²</td>
</tr>
<tr>
<td>Y</td>
<td>1.76 in²</td>
<td>1.10 in²</td>
</tr>
<tr>
<td>Y2</td>
<td>1.76 in²</td>
<td>0.55 in²</td>
</tr>
</tbody>
</table>

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.

**Section A-A**

**Panel Tolerances:**

- Overall Dimensions:
  - Standard Panel: ± 3/16" Vertical, ± 3/16" Horizontal
  - Top and Special Panels: ± 3/16" Vertical, ± 3/16" Horizontal

- Connection Device Locations:
  - Embeds:
    - ± 1" Vertical
    - ± 1" Horizontal

- Panel Squareness:
  - 90° Panel Corners: ± 3/16" using 2" square (measure 3 panel corners)

- Panel Diagonal:
  - Panels with 90° Corners: ± ½" max. difference between diagonals

- Surface Finish:
  - Finish at front face: ± ¼" in 5'

**Certified Only with Respect to Internal Stability of Reinforced Earth Structures**
MATERIAL PROPERTIES NOTES:

1. PANEL REINFORCEMENT BARS SHALL BE DEFORMED BILLET STEEL BARS FOR CONCRETE REINFORCEMENT CONFORMING TO THE SPECIFICATIONS OF ASTM DESIGNATION A615, GRADE 60, INCLUDING SUPPLEMENTARY REQUIREMENT S1 OR LOW ALLOY STEEL DEFORMED BARS CONFORMING TO THE SPECIFICATIONS OF ASTM DESIGNATION A705. STRUCTURAL WELDED WIRE REINFORCEMENT THAT CONFORMS TO ASTM A185/A497 OR SMOOTH SPECIFICATIONS MAY BE SUBSTITUTE FOR ASTM DESIGNATION A615.


3. THE LOOP EMBEEDS, SOIL REINFORCEMENT AND CONNECTION PINS SHALL BE GALVANIZED IN ACCORDANCE WITH ASTM DESIGNATION A123 AFTER SENDING.

4. CONCRETE PANELS TO HAVE A 28-DAY COMpressive STRENGTH OF 4000 PSI.

5. ALL PANEL REINFORCEMENT MUST HAVE A MINIMUM OF 1/2" COVERAGE WITH CONCRETE ON ALL SIDES.

6. FOR PANELS WITH W11 SOIL REINFORCEMENT USE "A" PANEL REINFORCEMENT. PANELS WITH W20 SOIL REINFORCEMENT USE "X" PANEL REINFORCEMENT. PANELS WITH W24 SOIL REINFORCEMENT USE "Y" PANEL REINFORCEMENT.

7. THE MOLDED PLASTIC PANEL PADS SHALL BE COMPOSED OF A HIGH DENSITY POLYETHYLENE MATERIAL, SHALL HAVE A MINIMUM TENSILE STRENGTH OF 4 KSI AND A MINIMUM TENSILE ELONGATION OF 600 PERCENT.

8. FILTER FABRIC IS A NON-WOVEN GEOTEXTILE COMPOSED OF Polypropylene FIBERS, WHICH ARE FORMED INTO A STABLE NETWORK THAT THE FIBERS RETAIN THEIR RELATIVE POSITION. FILTER FABRIC IS INERT TO BIOLOGICAL DEGRADATION AND RESISTS NATURALLY ENCOUNTERED CHEMICALS, ALKALI AND ACIDS.

9. THE MINIMUM INSIDE BEND DIAMETER FOR W11 AND W20 WIRE USED FOR SOIL REINFORCEMENT SHALL BE NO LESS THAN TWICE THE NOMINAL DIAMETER OF THE WIRE SIZE AND IN NO INSTANCE LESS THAN 1 INCH. THE INSIDE BEND DIAMETER FOR W24 WIRE USED FOR SOIL REINFORCEMENT SHALL BE 2 1/2 INCHES.

10. THE BEARING BAR AND THE CROSS WIRE SHALL BE THE SAME SIZE.

FILTER FABRIC DETAIL AND PANEL PLACEMENT

VERTICAL PANEL JOINT DETAIL

HORIZONTAL PANEL JOINT DETAIL
TYPICAL "OUTSIDE" CORNER LAYOUT

TYPICAL C CORNER PANEL
SHOWN FROM BACK FACE

SECTION A-A

BEARING PAD PLACEMENT DETAIL

NOTE: TYPICAL PLACEMENT SHOWN; PANELS WITH DIFFERENT ANGLES WILL HAVE DIFFERENT BEARING PAD LAYOUTS
DRAINAGE INLET DETAILS
TYPICAL CROSS SECTION A-A

NOTES:
1. Obstruction shall be constructed before wall installation or, void former shall be installed during backfill placement. Void former not supplied by SSL.

DRAINAGE DETAILS: 1 OF 2

WELDED WIRE SOIL REINFORCEMENT

1" DIAMETER HOLE
4 PLACES

6" 1" 1" 1" 6"
ASTM A351 GALVANIZED

3/4" EXPANSION BOLT (GALV.)
FIELD DRILL AND INSTALL
(SUPPLIED BY OTHERS)

CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF RENIERCED EARTH STRUCTURES

STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION

DANIEL MITCHELL
05/13/13

SD 06

REVISION DESCRIPTION

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TYPICAL DRAINAGE DETAILS
(CENTER LINE OF INLET TO BE RELOCATED TO PANEL JOINT)
SCALE: 1"=20"

PLACE OBSTRUCTED ROWS OF MESH TO OUTSIDE CONNECTION OPENINGS TO AVOID OBSTRUCTION.

BRAINAGE INLET SHALL BE CENTERED ON THE CLOSEST OBSTRUCTED PANEL JOINT.
SEE ELEVATIONS FOR INLET STATION

MESH BELOW THE OBSTRUCTION SHALL BE PLACED IN THE STANDARD CONFIGURATION

OBSTRUCTED PANEL

OBSTRUCTED PANEL

A-

A-

SEE SHEET SD-06 FOR SECTION A-A

MESH CONNECTOR FOR ALL PANEL TYPES
SEE DETAILS ON SHEET SD-06

PREAPPROVED WALL APPENDIX: SPECIFIC REQUIREMENTS AND DETAILS FOR SSL CONCRETE PANEL WALLS
APPENDIX 15-N

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Appendix 15-N Preapproved Wall Appendix: Specific Requirements and Details for SSL Concrete Panel Walls

STANDARD "A" PANEL WITH A FORMED HOLE FOR PENETRATIONS THROUGH THE WALL FACE SHOWN FROM BACK FACE

TYPICAL PENETRATION DETAIL

NOTES:
1. WOTAR MAY BE SUBSTITUTED FOR CONCRETE, SEE PROJECT SPECIFICATIONS FOR STRENGTH.
2. 1" THICK EXPANSION JOINT MATERIAL MAY BE OMITTED IF THE DIAMETER IS LESS THAN 8".

WRAP AND SECURE FILTER CLOTH AROUND PIPE OVER JOINT PRIOR TO BACKFILLING BEHIND WALL
FORM CONCRETE AROUND PIPE OVER EXPANSION JOINT MATERIAL AS SHOWN. ALLOW CONCRETE TO SET PRIOR TO BACKFILLING BEHIND WALL
TROWEL CONC. SMOOTH AGAINST FACE OF MSE WALL PANEL AROUND PIPE

CERTIFIED ONLY WITH RESPECT TO INTERNAL STABILITY OF REINFORCED EARTH STRUCTURES

SD-08 DANIEL MITCHELL 05/13/13
STATE OF WASHINGTON DEPARTMENT OF TRANSPORTATION
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 embedment, 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 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.

PROPER STORAGE AND HANDLING OF WELDED WIRE SOIL REINFORCEMENT

1. Soil reinforcement arrives to the site on a flatbed truck with Dunnage separating the different bundles of soil reinforcement (see Figure 3).

2. Off-load the soil reinforcement carefully, using at least two balanced pick points spaced no more than 7 feet apart (see Figure 3).

3. Place soil reinforcement on the Dunnage before setting on the ground, making sure that the soil reinforcement does not contact the ground. Do not place the soil reinforcement 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 classification purposes only. Dunnage may need to be added or removed based on the length of the soil reinforcement being placed into storage.

Figure 1

Figure 2

Figure 3

This drawing contains information proprietary to SSL and is furnished for the benefit of SSL. This information shall not be transmitted to any other person or entity without written consent of SSL.

Handing and unloading details:

5x5 Panel with Tongue and Groove

State of Washington, Department of Transportation

Preapproved Wall Appendix: Specific Requirements and Details for SSL Concrete Panel Walls
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December 2013
5.0 ERECTION SEQUENCE

5.1 OVERVIEW

The side faces of the supporting walls shall be faced with a masonry panel. The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.2 INSTALLATION

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.3 QUALITY CONTROL

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.4 SAFETY

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.5 ABANDONMENT

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.6 MASONRY PANELS

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.7 BUSINESSES

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.8 DISTANCE

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.9 INERTIALS

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.10 RESPONSE

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.11 REPAIR

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.12 TERMINATION

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.13 PRECAST PANELS

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.14 SOURCE

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.15 PRECAST CONCRETE PANELS

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.

5.16 ERECTION SEQUENCE

The masonry panels shall be placed and fastened in accordance with the specifications provided in this Appendix. The masonry panels shall be placed and fastened in a manner that ensures the stability of the structure. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart. The masonry panels shall be fastened with a minimum of two bolts per panel, and the bolts shall be placed at a minimum distance of 12 inches apart.
# Appendix 15-N Preapproved Wall Appendix: Specific Requirements and Details for SSL Concrete Panel Walls

## Type "TS" Panels with 2-3 Embeds

Shown from back face

<table>
<thead>
<tr>
<th>Qt.</th>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>VERTICAL BAR</td>
<td>W15 WIRE – GRADE 80</td>
</tr>
<tr>
<td>VAR</td>
<td>HORIZONTAL BAR</td>
<td>W15 WIRE – GRADE 60</td>
</tr>
<tr>
<td>3</td>
<td>LIFTING INSERT</td>
<td>1 TON INSERT</td>
</tr>
<tr>
<td>VAR</td>
<td>LOOP EMBEDS</td>
<td>6 CONNECTION EMBED</td>
</tr>
</tbody>
</table>

Use W11 SSL reinforcement per layer

**Note:**
- For rebar panel reinforcement:
  - Top panels above 68” need 12 horizontal W15 bars
  - Top panels 85” to 115” need 14 horizontal W15 bars
  - Top panels 75” 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 48” 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.
In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply:

**Facing Blocks** – Blocks acceptable for use are the Landmark tapered and straight blocks. These blocks can form facing batters of vertical (0 degrees) to 4 degrees. Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2.5 feet.

**Soil Reinforcement** – Only geosynthetic reinforcement listed in the QPL and which has been evaluated for connection strength with the Landmark wall system shall be used. Therefore, the following specific QPL geosynthetic reinforcement products are approved for use with this wall system:

- Miragrid 5XT
- Miragrid 8XT
- Miragrid 10XT

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications.

**Reinforcement/Facing Block Connection Requirements** – The connection between Landmark facing units and the geosynthetic reinforcement is essentially a mechanical connection, with the possible exception of the connection when Miragrid 10XT is used. For mechanical connections, the connection resistance is generally not dependent on the normal force between blocks. The connection testing conducted for this wall system demonstrates that the connection is behaving as a mechanical connection for the Miragrid 5XT and 8XT. For the 10XT, the connection strength increases as normal stress increases. Therefore, it is likely that the connection with Miragrid 10XT is at least partially depending on frictional resistance. The design facing/reinforcement connection strength shall be as specified in the following table.

**Table 15-O-1** Approved Connection Strength Design Values for Landmark Walls

<table>
<thead>
<tr>
<th>Block</th>
<th>Geogrid Product</th>
<th>$T_{ultconn}$ (lbs/ft)</th>
<th>$T_{lot}$ (lbs/ft)</th>
<th>$CR_u$</th>
<th>Creep Reduction Factor applicable to the Connection (use for $RF_{CR}$ in Eq. 1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight Block</td>
<td>Miragrid 5XT</td>
<td>2800*</td>
<td>3844</td>
<td>0.73</td>
<td>1.45*</td>
</tr>
<tr>
<td></td>
<td>Miragrid 10XT</td>
<td>3948+N*Tan 16°</td>
<td>9456</td>
<td>$T_{ultconn}/9456$</td>
<td>1.2</td>
</tr>
<tr>
<td>Tapered Block</td>
<td>Miragrid 5XT</td>
<td>2837 – N*Tan 7°</td>
<td>3844</td>
<td>$T_{ultconn}/3844$</td>
<td>1.45*</td>
</tr>
<tr>
<td></td>
<td>Miragrid 8XT</td>
<td>4250 – N*Tan 5°</td>
<td>6564</td>
<td>$T_{ultconn}/6564$</td>
<td>1.45*</td>
</tr>
<tr>
<td></td>
<td>Miragrid 10XT</td>
<td>3770+N*Tan 30° to N = 2850 lbs/ft, and 5400 lbs/ft at N &gt; 2850 lbs/ft</td>
<td>9456</td>
<td>$T_{ultconn}/9456$</td>
<td>1.2</td>
</tr>
</tbody>
</table>

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

+ This is a lower bound value – see connection test results in report by Bathurst, Clarabut Geotechnical Testing, Inc., Project report No. BCGT9930, 9/1/2000.

*Same as the value of RFCR reported in the QPL, Appendix D for these geogrid products.
T_{ac}, the long-term connection strength, shall be calculated as follows:

\[ T_{ac} = \frac{T_{MARV} \times CR_U}{RF_{CR} \times RF_D} \]

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 \( \frac{1}{8} \) inch is allowed, but that Section 15.5.3.8 recommends a tighter dimensional tolerance of \( \frac{1}{16} \) 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 \( \frac{1}{16} \) inch to reduce the risk of significant cracking of facing blocks.

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

- In plan sheet 5 of 6, the guard rail detail, the guard rail post shall either be installed through precut holes in the geogrid layers that must penetrated, or the geogrid layers shall be cut in a manner that prevents ripping or tearing of the geogrid.

- In plan sheet 3 of 6, regarding the geogrid at wall corner detail, cords in the wall facing alignment to form an angle point or a radius shall be no shorter than the width of the roll to insure good contact between the connectors and the geogrid cross-bar throughout the width of the geogrid. Alternatively, the geogrid roll could be cut longitudinally in half to allow a tighter radius, if necessary.
PROPOSED SEGMENTAL RETAINING WALL PLANS FOR:

PROJECT NAME (1)

PROJECT NAME (2)

CITY, WASHINGTON

PART 1 GENERAL

1.01 SUMMARY

A. Section includes:
   Work includes furnishing and installing Anchor Landmark modular retaining wall units to the lines and grades designated on the construction drawings and as specified herein.

1.02 REFERENCES

A. American Society of Testing and Materials
   1. ASTM C 146 Standard Test Methods of Sampling and Testing Concrete Masonry Units
   2. ASTM D 448 Standard Classification for Sizes of Aggregate for Road and Bridge Construction
   3. ASTM C 568 Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.46
      Rammer and 134c, Drop (Standard Proctor)
   4. ASTM C 1292 Standard Test Method for Evaluating Freeze-Thaw Durability of Manufactured Concrete Masonry
      Units and Related Concrete Units
   5. ASTM C 1372 Standard Specification for Segmental Retaining Wall Units
   6. ASTM D 1556 Standard Test Method for Density of Soil in Place by the Sand Cone Method
      Rammer and 134c, Drop (Modified Proctor)
      Unified Soil Classification System
   9. ASTM D 2522 Standard Test Method for Density of Soil and Soil-Aggregate In Place by Nuclear Methods (Shallow
      Depth
   10. ASTM D 4354 Practice for Sampling of Geosynthetics for Testing
   11. ASTM D 4596 Test Method for Tensile Properties of Geosynthetics by the Wide Width Tensile Test Method
   12. ASTM D 4769 Practice for Determining Specification Conformance of Geosynthetics
   14. ASTM D 5292 Geosynthetic Strength of Geosynthetics

B. American Association of State Highway Officials
   2. AASHTO M 278 Standard Specification for Geotextile Reinforcement (GFRP) Pipe
   3. AASHTO M 304 Standard Specification for Poly (Vinyl Chloride) (PVC) Profile Wall Drain Pipe and Fittings Based
      on Controlled Inside Diameter
   5. Geosynthetic Research Institute
      1. GRI G44: Allowable Design Strength of Geosynthetics

1.03 SUBMITTALS

A. Submit the following in accordance with Section 1.01:
   1. Manufacturer's Literature: Materials description
   2. Shop drawings/Retaining wall system design, including wall heights, geosynthetic reinforcement layout, drainage
      provisions and other pertinent details. The shop drawings shall be signed by a registered professional engineer
      licensed in the state of wall installation.
   3. Certificate of Compliance letter in accordance with Section 1.02 (Earth Retaining Structures, Proprietary Earth
      Retaining Systems) and Section 1.05 of the Standard Specifications
   4. Design calculations demonstrating satisfaction safety factors for:
      1) Overall stability
      2) Internal stability
      3) Bearing capacity
      4) Sliding

1.04 DELIVERY, STORAGE AND HANDLING

A. The contractor shall check the materials upon delivery to assure that proper material has been received.
B. Deliver and handle materials in such a manner as to prevent damage, Stores above ground on wood pallets or skids,
   Remove damaged or otherwise unsatisfactory material, when so determined from the site;
C. The contractor shall prevent excessive mud, wet cement, spaws and the material, from coming in contact with the modular
   units and reinforcement;
D. Geosynthetic materials shall remain in protective wrapping until placed in the wall. Follow manufacturer's recommenda-
   tions regarding protection from direct sunlight.
E. Lock bar material shall remain in boxes until placed in the modular units. Lock bar exposed to direct sunlight for a period
   exceeding 2 months should not be used in the constructed wall.

1.05 DEFINITIONS

A. Geosynthetic Reinforcement is a material specifically fabricated for use as soil reinforcement
B. Landmark modular retaining wall units are machine made from portland cement, water, mineral aggregates and potentially
   fly ash and various admixtures
C. Permeable Materials are fine draining materials used as a drainage media
D. Reinforced Backfill is the soil used as fill within the geosynthetic reinforced soil mass
E. Foundation Soil is the soil mass supporting the leveling pad and reinforced soil zone of the modular retaining wall system
F. Wall Backfill is a perforated PVC pipe, generally of 4 inch (100 m) diameter, used to drain water from soil
G. Filter Fabric is a non-woven geosynthetic material used for filtration and to allow for the long-term passage of water
   into a subsurface drain system while retaining the illus soil
H. The Landmark Lock Bar is a specifically manufactured polymer based material, supplied by Anchor, and used to
   mechanically connect the reinforcement products to the Landmark units.

1.06 DISCREPANCIES

Should discrepancies exist between the plans and specifications, plans shall take precedence over the specifications.
PART 2 PRODUCTS

2.01 MATERIALS

A. Modular Block Facing Unit: "Landmark" as manufactured under license from Anchor Wall Systems, No substitutions allowed. Landmark units shall meet requirements of ASTM C1572 except as modified by HWNA-NH-00-043, section 14b.

1. Unit height dimensions shall not vary more than +/-0.16 inch (1.6 mm) from that specified.
2. Unit width dimensions shall not vary more than +/-0.20 inch (3.2 mm) from that specified.
3. Minimum required compressive strength shall be 4,000 psi (28 MPa).
4. Maximum water absorption shall be 5%.
5. Color: Choose a stock sample for approval by the resident engineer.
7. The concrete slabs shall include an integral concrete shear connection device.
8. Geosynthetic reinforcement shall be placed at 5% in SRT, 1% in SRTX, No substitutions allowed.
9. Connections: The Landmark block bar as supplied by Anchor Wall Systems shall be manufactured from CPVC and PVC to the dimensions shown on the plans.
10. Geosynthetic material shall conform to the provisions in Section (Earth Retaining Structures) of the Special Provisions for Permeable Materials.
12. With substrate: Shall conform to the provisions in Section (Substrate) of the Standard Specifications and the Special Provisions for this project.
13. Filter fabric: Shall conform to the provisions in Section (Filter Fabric) of the Standard Specifications and the Special Provisions for this project.

H. Leveling Pad: A leveling pad of unreinforced concrete shall be placed to facilitate first course placement. Concrete leveling pads shall have a minimum thickness of 6 inches (150 mm) and a minimum width of 24 inches (600 mm) and shall have a minimum 28-day compressive strength of 3,000 psi (20.7 MPa).

PART 3 EXECUTION

3.01 EXAMINATION

A. The contractor shall examine the areas and conditions under which the retaining walls are to be erected and notify the owner's representative in writing of conditions detrimental to the proper and timely completion of the work. Contractor shall not proceed with the work until satisfactory conditions have been corrected. The contractor shall promptly notify the wall design engineer and owners representative of site conditions which may affect work performance or require a reevaluation of the wall design.

B. Foundation soil and cutback materials shall be examined by the project geotechnical engineer or technician to ensure that the actual soil is consistent with the foundation soil strength, meets or exceeds the strength required, as shown on the construction drawings. Geotechnical cutback slabs for precast or water sewage.

3.02 EXCAVATION

A. The contractor shall excavate to the lines and grades shown on the construction drawings. Overexcavation not approved by the owner or owner's representative shall not be paid for and replacement with approved compacted fill and/or wall system components will be required at the Contractor's expense. Do not disturb base beyond the lines shown on the plans.

3.03 FOUNDATION PREPARATION

A. Foundation soil shall be excavated as required for the base reinforcement dimension shown on the construction drawings, or as directed by the engineer.
B. The project geotechnical engineer shall examine the foundation and related soils to ensure that the soil strength and types meet or exceed those required, as shown on the construction drawings. Foundation soil not meeting the strength required for bearing capacity or settlement shall be remediated per the direction of the engineer.

3.04 BASE COURSE PREPARATION

A. Leveling pad materials shall be placed as shown on the construction drawings on the pre-approved foundation soils.
B. Leveling pad materials shall be installed upon undisturbed soils, or foundation soils prepared in accordance with Section 3.03.
C. Concrete leveling pads shall be allowed to cure for 12-hours prior to placement of the first course of modular units.
D. Leveling pad materials shall be placed to provide intimate contact with the modular wall units.
E. Leveling pad materials shall be to the depths and widths shown on the plans.

3.05 WALL ERECTION

A. Foundation units shall be placed on the prepared leveling pad. Units shall be checked for horizontal alignment with a straight line placed at the back of the units and vertical alignment from top and bottom of wall units. The top of all units in the base course shall be at the same elevation.
B. Ensure that concrete wall units are in full contact with base. A 1/8 inch (3.2 mm) gap between foundation units is allowed, provided a suitable filter fabric is placed behind the foundation units.
C. The foundation course of modular units shall be backfilled and compacted, front and back, then checked for level and alignment prior to placing the next course of wall units.
D. Wall subbase shall be installed at the lowest elevation possible to maintain gravity flow of water to out of the reinforced zone. The wall backfill shall be established to an appropriate location away from the wall system at each low point and at 50-foot (15 m) intervals along the wall.
E. Remove all excess fill from top of units and from the back channels of the top of the unit and install next course.
F. Subsequent courses of modular units shall be placed side by side for full length of wall alignment. A maximum gap of 1/8 inch (3.2 mm) is allowed between units. Alignment should be checked by using a stringing the back of the units. Adjust units as necessary to maintain horizontal alignment.
G. If required, a maximum of 12 inches (300 mm) of angular permeable material shall be placed behind the modular units.
H. A Filter fabric may be required between the permeable material and reinforced soil wall depending on the compatibility of the permeable material and reinforced soil zone materials.
I. Ensure permeable materials and basaltic soil are compacted before installation of each succeeding course.
J. Install each succeeding course. Basaltic soil must be compacted and prior to placement of the next course. Pull the units forward until the back of the unit contacts the backfilling surface of the units in the preceding course.
K. Check vertical alignment with a string on each course. Adjust units as necessary with reinforcement struts to maintain proper alignment and setback conditions.
L. Permeable material or reinforcement wall shall be placed level with the top of the modular units at courses where reinforcement is required.
M. Remove all excess fill from top of units and from the back channels of the top of the units prior to backfill placement.
N. Install geosynthetic reinforcement at locations and elevations shown on the design drawings.

1. The geosynthetic reinforcement has a primary direction. The primary direction must be placed perpendicular to the wall face.
2. Reinforcement panels shall be continuous. Seams or connections are not permitted. Adjacent panels shall be abutted with less than 4 inch gap between adjacent panels. 100 percent reinforcement coverage is required.
3. Panels of geosynthetic reinforcement shall be tensioned such that all fabric and wire is removed before reinforced units placed. Panels shall be stapled or anchored as necessary to maintain test conditions.
4. Track车辆 may not operate on geosynthetic reinforcement without less than 6 inches of compacted soil between the reinforcement and the track. Operating speeds must be kept to a minimum to prevent damage and disturbance to the reinforcement.
5. Rubber-tired vehicles may operate at less than 10 mph between each pair of reinforcement. Alignment backfill is not required on courses where geosynthetic reinforcement is not placed. Gaps between adjacent sections of backfill shall be no more than 3 inches (75 mm). The backfill bar shall be placed flat side up, with the angled sides to the back of the unit, as shown on the construction drawings. The reinforcement must be maintained within 1/8 inch (20 mm) of the face of the smaller Landmark units below.
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 loosely level to an elevation approximately 1" (25mm) above
the level of the footing unit before placing the soil reinforcement.
C. Any defects already on the top of the units and from within the channel in the top of the units and the backfill be 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 pilings 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 slacks or voids from forming in the reinforcement.
G. Place a minimum of 8" (150mm) of backfill prior to operating equipment above the reinforcement. Avoid sudden brailing 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 8 to 10 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.

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
setting 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.
In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply:

**Facing Blocks** – Blocks acceptable for use with this wall system include, AB Classic, and AB Vertical. These blocks are for a facing batter of 1°, 3°, and 6° degrees. Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2 ft.

**Soil Reinforcement** – Only geosynthetic reinforcement listed in the WSDOT QPL and which has been evaluated for connection strength with the Allan Block wall system shall be used. For walls with a face batter of 1 degrees or more (i.e., facing blocks, AB Classic, and AB Vertical), this includes the following specific products that are approved for use with this wall system:

<table>
<thead>
<tr>
<th>Geosynthetic Product</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Miragrid 3XT</td>
<td>Stratagrid SG200</td>
</tr>
<tr>
<td>Miragrid 5XT</td>
<td>Stratagrid SG350</td>
</tr>
</tbody>
</table>

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications.

**Reinforcement/Facing Block Connection Requirements** – Connection testing was done for the range of blocks and geosynthetic reinforcements preapproved for this wall system. The connection between Allan Block facing units and the geosynthetic reinforcement is essentially a frictional connection. That being the case, the connection resistance is strongly dependent on the normal force between blocks and in the gravel in-fill inside the blocks and less dependent on the roll or lot specific tensile strength, \( T_{\text{lot}} \), as well as the long-term effect of creep on the connection strength. However, neither \( T_{\text{lot}} \) for each test (only \( T_{\text{MARV}} \) values for the tested geogrids were provided), nor connection creep tests, were provided. Since no connection creep tests were provided, as required in the AASHTO LRFD Bridge Design manual, \( RF_{CR} \) must be used to obtain \( T_{ac} \). Therefore, the long-term connection strength (i.e., \( T_{ac} \)) equation provided in the AASHTO LRFD Bridge Design Manual will need to be simplified to the equation shown below:

\[
T_{uc} = \frac{T_{\text{ultconn}}}{RF_{CR} \times RF_{D}} \tag{15-P-1}
\]

where,

- \( T_{\text{ultconn}} \) is the ultimate connection strength from the product specific connection strength tests, the results of which are provided in Table 15-S-1,
- \( RF_{CR} \) = creep reduction factor for the geosynthetic, and
- \( RF_{D} \) = the durability reduction factor for the geosynthetic.
RF_{CR} \text{ and } RF_D \text{ shall be as provided in the WSDOT QPL, Appendix D.}

### Table 15-P-1  Approved connection strength design values for Allan Block walls

<table>
<thead>
<tr>
<th>Applicable Facing Blocks</th>
<th>Geogrid Product</th>
<th>Normal Load, N (lbs/ft)</th>
<th>T_{ultconn} (lbs/ft) Face Batter = 1° or 3°</th>
<th>T_{ultconn} (lbs/ft) Face Batter = 6°</th>
</tr>
</thead>
<tbody>
<tr>
<td>AB Classic and AB Vertical</td>
<td>Miragrid 3XT</td>
<td>N ≤ 2474</td>
<td>1239 + N*Tan 26° 2,450</td>
<td>1193 + N*Tan 29° 2,560</td>
</tr>
<tr>
<td></td>
<td>Miragrid 5XT</td>
<td>N ≤ 3713</td>
<td>1320 + N*Tan 27° 3,210</td>
<td>1287 + N*Tan 29° 3,350</td>
</tr>
<tr>
<td></td>
<td>Stratagrid SG200 &amp; SG350</td>
<td>N ≤ 2474</td>
<td>890 + N*Tan 34° 2,560</td>
<td>1383 + N*Tan 18° 2,190</td>
</tr>
<tr>
<td></td>
<td>Stratagrid SG350</td>
<td>N ≤ 3713</td>
<td>1079 + N*Tan 19° 2,360</td>
<td>1257 + N*Tan 12° 2,050</td>
</tr>
</tbody>
</table>

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

The connection strengths provided in the table assume that crushed rock is used to fill the interior of the blocks. Allan Block also provides the option to grout the interior of the blocks, creating a full mechanical connection. This connection approach is not preapproved, as connection strength data for this situation was not provided, and furthermore, the elevated pH that could be caused by the grout could accelerate chemical degradation. This has not been evaluated.

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

- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet.
- In plan sheet 7 of 12, the guard rail detail, the guard rail post shall either be installed through precut holes in the geogrid layers that must be penetrated, or the geogrid layers shall be cut in a manner that prevents ripping or tearing of the geogrid.
- In plan sheet 5 of 12, regarding the geogrid at wall corner detail, cords in the wall facing alignment to form an angle point or a radius shall be no shorter than the width of the roll to insure good contact between the connectors and the geogrid cross-bar throughout the width of the geogrid. Alternatively, the geogrid roll could be cut longitudinally in half to allow a tighter radius, if necessary.
- It is noted in ASTM C1372 that a dimensional tolerance for the height of the block of 1/8 inch is allowed, but that WSDOT GDM Section 15-5.3.8 recommends a tighter dimensional tolerance of 1/16 inch. Based on WSDOT experience, for walls greater than 25 ft in height, some cracking of facing blocks due to differential vertical stresses tends to occur in the bottom portion of the wall. Therefore, blocks placed at depths below the wall top of 25 ft or more should be cast to a vertical dimensional tolerance of 1/16 inch to reduce the risk of significant cracking of facing blocks.
CONSTRUCTION DRAWINGS
PREPARED FOR
WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION

ALLAN BLOCK DETAIL AND CONSTRUCTION NOTES

SHEET INDEX

<table>
<thead>
<tr>
<th>SHEET NO.</th>
<th>SHEET TITLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TITLE SHEET</td>
</tr>
<tr>
<td>2</td>
<td>WALL CONSTRUCTION DETAILS (1 OF 2)</td>
</tr>
<tr>
<td>3</td>
<td>WALL CONSTRUCTION DETAILS (2 OF 2)</td>
</tr>
<tr>
<td>4</td>
<td>CORNER AND CAP DETAILS</td>
</tr>
<tr>
<td>5</td>
<td>CURVE WALL AND GEORED GEODRED OBSTRUCTION DETAILS</td>
</tr>
<tr>
<td>6</td>
<td>LARGE GEORED GEODRED OBSTRUCTION DETAILS</td>
</tr>
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<td>7</td>
<td>RAILING AND BARRIER DETAILS</td>
</tr>
<tr>
<td>8</td>
<td>TRAFFIC BARRIER AND COPING DETAILS</td>
</tr>
<tr>
<td>9</td>
<td>WALL PENETRATION DETAILS (1 OF 2)</td>
</tr>
<tr>
<td>10</td>
<td>WALL PENETRATION DETAILS (2 OF 2)</td>
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<tr>
<td>11</td>
<td>CULVERT PENETRATION DETAILS</td>
</tr>
<tr>
<td>12</td>
<td>TRANSITION AND SLIP JOINT DETAILS</td>
</tr>
</tbody>
</table>
Appendix 15-P
Preapproved Wall Appendix: Specific Requirements and Details for Allan Block Walls

2.2: ALLAN BLOCK AB VERTICAL (1") - BLOCK PROFILE

NOTE: Actual dimensions and setbacks will vary slightly by manufacturer. Check with your local Allan Block manufacturer for exact specifications.

2.3: ALLAN BLOCK AB CLASSIC - BLOCK PROFILE
3.1: ALLAN BLOCK TYPICAL SECTION - REINFORCED WALL

3.2: STEP UP AT BASE COURSE

3.3: CONCRETE LEVELING PAD
5.1: INSIDE CURVE GEOGRID OVERLAP

5.2: OUTSIDE CURVE GEOGRID OVERLAP

5.3: ALLAN BLOCK TYPICAL DETAIL - GRID OBSTRUCTION

5.4: GEOGRID PLACEMENT AT PAVEMENT / OBSTRUCTION SECTION
8.1: ALLAN BLOCK TYPICAL SECTION - CIP CONCRETE COPING

WSDOT Conditional Pre-approval: The details showing grout in the interior cells of the blocks are not pre-approved when the grout is in direct contact with the geogrid reinforcement.

8.2: ALLAN BLOCK TYPICAL SECTION - TRAFFIC BARRIER
9.1: ALLAN BLOCK TYPICAL SECTION - CULVERT THROUGH WALL

9.2: CULVERT THROUGH WALL DETAIL
10.1: ALLAN BLOCK TYPICAL SECTION - SMALL PIPE PENETRATION (LESS THAN 6 in DIAMETER)

*REFER TO DESIGN DETAILS SECTION 1:
ALLAN BLOCK TYPICAL SECTION FOR
ALL OTHER NOTES, DETAILS AND
SPECIFICATIONS.

ALLAN BLOCK UNIT
CUT NOTCH IN ALLAN BLOCK TO ALLOW FOR
DRAIN PIPE. ROBBER SCREEN AS REQUIRED.
NON-SHINING GROUT CONCRETE TO FILL
VOIDS.

14 STEEL REINFORCEMENT BAR
POURED CONCRETE

3 in (75 mm)
MINIMUM COVER

PIPE - UP TO 4 in (1.2 m) DIAMETER
EXTEND REBAR A MINIMUM OF 2 ft (600 mm) PAST EDGE
OF PIPE OR EITHER SIDE

ALLAN BLOCK ELEVATION
SCALE: NOT TO SCALE

10.2: ALLAN BLOCK TYPICAL SECTION (UP TO 4 ft DIAMETER)

*REFER TO DESIGN DETAILS:
WSDOT CONSTRUCTION DRAWINGS.

*WHEN POSSIBLE, INSTALL ALL PIPES
PERPENDICULAR TO FACE OF WALL. IF
SHEW IS REQUIRED, SHEW MAY NOT
EXCEED 45 DEGREES.

ALLAN BLOCK UNIT
8 in (203 mm)
PIPE VENTED THROUGH THE
FACE OF THE WALL

ALLAN BLOCK DETAIL
SCALE: NOT TO SCALE
In addition to the general design requirements provided in Appendix 15-A, the following specific requirements apply:

**Facing Blocks** – Blocks acceptable for use with this wall system are the 28-inch Positive Connection blocks. The 41-inch blocks shown in the drawings are not considered part of the approved system.

**Soil Reinforcement** – Only geosynthetic reinforcement listed in the WSDOT QPL and which has been evaluated for connection strength with the Redi-Rock Positive Connection wall system shall be used. The following products are approved for use with this wall system:

- Miragrid 5XT
- Miragrid 8XT
- Miragrid 10XT
- Miragrid 20XT
- Miragrid 24XT

All Miragrid products for the Redi-Rock Positive Connection system will be 12-inch wide rolls consisting of 11 longitudinal ribs. TenCate Geosynthetics will provide certification of the wide width tensile strength of the 12-inch wide rolls.

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications.

**Reinforcement/Facing Block Connection Requirements** – The connection between the facing units and the geosynthetic reinforcement is essentially independent of the normal force between the blocks (i.e., not a frictional connection), as the reinforcement strips wrap around the internal wall of the block as a continuous layer. The design facing/reinforcement connection strength shall be as specified in the following table:

<table>
<thead>
<tr>
<th>Geogrid Product</th>
<th>$T_{\text{ultconn}}$ (lbs/ft)</th>
<th>$T_{\text{tot}}$ (lbs/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Miragrid 5XT</td>
<td>4,460</td>
<td>5,334</td>
</tr>
<tr>
<td>Miragrid 8XT</td>
<td>7,928</td>
<td>8,055</td>
</tr>
<tr>
<td>Miragrid 10XT</td>
<td>8,681</td>
<td>10,635</td>
</tr>
<tr>
<td>Miragrid 20XT</td>
<td>13,447</td>
<td>16,397</td>
</tr>
<tr>
<td>Miragrid 24XT</td>
<td>20,199</td>
<td>29,130</td>
</tr>
</tbody>
</table>
Appendix 15-Q

Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls

\[ T_{ac} \text{, the long-term connection strength, shall be calculated as follows:} \]

\[ T_{ac} = \frac{T_{MARV} \cdot CR_u}{RFCR \cdot RFD} \]  \hspace{1cm} (15-Q-1)

where,

- \( T_{MARV} \) = the minimum average roll value for the ultimate geosynthetic strength \( T_{ult} \),
- \( CR_u = T_{ultconn} / T_{lot} \) in which \( T_{ultconn} \) is the ultimate connection strength and \( T_{lot} \) is the lot specific ultimate tensile strength, (i.e., the lot or roll of material specific to the connection testing),
- \( RFCR \) = creep reduction factor for the geosynthetic,
- \( RFD \) = the durability reduction factor for the geosynthetic.

 RFCR and RFD shall be as provided in the WSDOT QPL, Appendix D.

Approved details for the Redi-Rock Positive Connection wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- Retaining wall heights up to a maximum of 33 feet.
- Retaining walls having a wall face batter of one degree to five degrees.
- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet.
- The pipe penetration details for pipes oriented up to a 45 degree skew angle as measured from perpendicular to the wall face are preapproved for pipe diameters of 18 inches or less.
- The cast-in-place concrete to be constructed around pipes that are protruding through the wall face is considered non-preapproved. Detailed stamped drawings and stamped engineering calculations are to be submitted for approval on a project specific basis.
- Reinforcement pullout design shall be calculated based on the default values for geogrid reinforcement provided in the latest edition of the AASHTO LRFD Bridge Design Specifications.
**Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls**

**Appendix 15-Q**

---

**Side View**

- **28" Positive Connection (PC) Middle Block**
- **Block Setbacks**

**Top View**

- **28" Positive Connection (PC) Middle Block**

**Front View**

- **28" Positive Connection (PC) Middle Block**

---

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Appendix 15-Q Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls

Typical Reinforced Wall Plan with 28” PC (Positive Connection) Blocks

No Scale

Typical Reinforced Wall Profile with 28” PC (Positive Connection) Blocks

No Scale

Legend

Station
0+00

Station
0+25

Station
0+50

Station
0+75

Station
0+100

- This drawing is for reference only.
- Final designs & construction must be prepared by a registered Professional Engineer using the actual conditions of the proposed site.
- Final wall design must address both internal and external drainage and be evaluated by the Professional Engineer who is responsible for the wall design.
Typical Reinforced Wall with 28" PC (Positive Connection) Blocks

- No Scale
- Special Items
- Perforated Sock Drain
- As Specified by Engineer
- 12" Wide Strip of Geogrid Wrapped Through Block and Extending Full Length (Typical)
- 28" PC Bottom Block
- Move Blocks Forward During Installation to Engage Shear Knobs (Typical)
- 28" PC Middle Block (Typical)
- 41" Bottom Block (No Center Slot)
- 6" Concrete Leveling Pad (Per WSDOT Standard Specifications)
- Perforated Sock Drain As Specified by Engineer
- SETBACK = 1 5/8"

This drawing is for reference only.

Final designs for construction must be prepared by a registered Professional Engineer using the actual conditions of the proposed site.

Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.

NOTE: One Degree or Zero Degree Batter Angle Walls are also available (Specialty Items)

NOTE: One Degree or Zero Degree Batter Angle Walls are also available (Specialty Items)

NOTE: This structural wall section may be used only for gravity applications at the ends of reinforced walls where the required wall height is 3 blocks or less.

 Glück auf, o Befürworter der Wahrheit, in eurer Sache seid Ihr gewiss, nur denke an die ewige Unwissenheit der Menschen!
Positive Connection (PC) Details

- Fill Slot and Wedge between Blocks with Crushed No. 57 Stone per WSDOT 9-03.1(4)C
- Free Draining Backfill Crushed No. 57 Stone per WSDOT 9-03.1(4)C
- To Extend at Least 12" Behind Wall

See www.redi-rock.com for Geogrid Connection and Interface Shear Test Reports.

12" Wide Strip of Geogrid Wrapped Through Block and Extending Full Length (L) Back Into Reinforced Fill Zone

Isometric View of Back of Blocks

No Scale

Section View Through Blocks

No Scale

Notes:
- This drawing is for reference only.
- Final design and construction must be performed by a registered Professional Engineer. If design is utilized for site not shown or described in this drawing, the site should be evaluated by the Professional Engineer responsible for the design.

Volume and Center of Gravity (C of G) calculations are based on the block as shown. Center of Gravity is measured from the back of the block. Heights may include standing seam or side. All weights and volumes are approximate. Final weights and volumes may vary.

Top - 28" PC Block
- Volume = 8.38 cft
- Weight = ±1200 lbs
- C of G = 15.5"

Middle - 28" PC Block
- Volume = 10.77 cft
- Weight = ±1540 lbs
- C of G = 14.4"

Bottom - 28" PC Block
- Volume = 11.96 cft
- Weight = ±1645 lbs
- C of G = 14.5"

Middle - 41" PC Block
- Volume = 15.34 cft
- Weight = ±2195 lbs
- C of G = 20.7"

Bottom - 41" Block
- Volume = 17.37 cft
- Weight = ±2483 lbs
- C of G = 21.3"

Half - 28" Middle Block
- Volume = 5.34 cft
- Weight = ±764 lbs
- C of G = 14.2"
**MOMENT SLAB AND TRAFFIC BARRIER**

INSTALLATION FOR LEVEL GRADE (0° SLOPE)

- #5 BARS AT 9" O.C.

INSTALLATION FOR SLOPING GRADE

- #5 BARS AT 8" O.C., TOP AND BOTTOM

**CAST-IN-PLACE MOMENT SLAB**

- 30'-0" SECTIONS

- 2" COVER

- 3" COVER

- 8'-0" MINIMUM.

**PAVEMENT**

- 1'-0" MIN

**REFER TO CONTRACT DOCUMENTS FOR**

- C.I.P. LEVEL-UP

- CONCRETE STRIP

- TRANSVERSE REINFORCEMENT #4 BARS

- DOWELS AT CONTRACTION AND EXPANSION JOINTS

- 1" EXPANDED POLYSTYRENE FOAM

- EXPANSION CAP

MOMENT SLAB SHOWN IS DIMENSIONED BASED ON AN EQUIVALENT STATIC LOAD OF 10,000 LBS PER NCHRP REPORT 663.

MOMENT SLAB REINFORCEMENT SHOWN IS BASED ON AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 5TH EDITION, TL-4, LOADING DETAILED IN TABLE A13.2.1. AND REQUIREMENTS SET FORTH IN WSDOT MOMENT SLAB DESIGN MEMORANDUM DATED 12/9/2011 AS PREPARED BY THE BRIDGE AND STRUCTURES OFFICE.

THE SELECTION AND USE OF THIS DETAIL, WHILE DESIGNED IN ACCORDANCE WITH GENERALLY ACCEPTED ENGINEERING PRINCIPLES AND PRACTICES, IS THE SOLE RESPONSIBILITY OF THE REGISTERED PROFESSIONAL ENGINEER IN CHARGE OF THE PROJECT.
**90° Outside Corner Detail with Specialty Corner Block**

- **Specialty Corner Block (No Scale)**
  - 4" x 6" x 2" High Oval Knob centered on block.
  - 4.6 x 0.8 x 0.16 High Oval Knob centered on block or 8" Dia. Knob.

**Isometric View of Corner (No Scale)**
- 1/4" wide groove, near end of block.
- The top row of blocks are shown in red, they have been cut out in line with their bottom grooves to show how they fit with the knobs on the bottom row of block.
- 10" Knob is fully engaged.

**Non-Woven Geotextile in All Joints Between Blocks (Typ)**

**Top View of Bottom Two Rows (No Scale)**
- 4" or 28" PC Blocks (41" shown)

**Overlap Blocks at Corner to Provide Full Engagement of Shear Knobs, Typical**

**90° Inside Corner Detail**

- 28" Redi-Rock PC Middle Blocks
- 28" Redi-Rock PC Bottom Blocks

**Note:**
- This drawing is for reference only.
- Final designs for construction must be prepared by a registered Professional Engineer using the actual conditions of the proposed site.
- Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.

**M. Walz 05-16-11**
**C. Hines 05-07-15**

**Typical Details PC - Ledger 100814.dwg**

**Redi-Rock PC Series**
**Inside & Outside 90° Corners**

---

*This drawing is for reference only.*
*Final designs for construction must be prepared by a registered Professional Engineer using the actual conditions of the proposed site. Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.*
Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls

Appendix 15-Q

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Note: 90° Corners are the only corners that can be constructed. All other corners must be converted to radius segments.
**CONVEX CURVES**

<table>
<thead>
<tr>
<th>NUMBER OF COURSES</th>
<th>HEIGHT OF BLOCKS (FROM FACE OF BLOCK)</th>
<th>BOTTOM ROW MIN. RADIUS</th>
<th>DISTANCE BETWEEN BLOCKS PER SKETCH</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1'-6&quot;</td>
<td>14'-6&quot;</td>
<td>0.13&quot;</td>
</tr>
<tr>
<td>2</td>
<td>2'-0&quot;</td>
<td>14'-6&quot;</td>
<td>0.21&quot;</td>
</tr>
<tr>
<td>3</td>
<td>3'-0&quot;</td>
<td>14'-6&quot;</td>
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</tr>
<tr>
<td>4</td>
<td>4'-0&quot;</td>
<td>15'-0&quot;</td>
<td>0.36&quot;</td>
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<td>14</td>
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<td>1.03&quot;</td>
</tr>
</tbody>
</table>

**CONCAVE CURVES**

- Place blocks tight together.
- No minimum radius - based only on block geometry.
- Smaller radii will result in more exposed untextured block surface.

**45° OUTSIDE CORNER RADIAL SOLUTION (REDI-ROCK PC BLOCKS)**

- Place bottom row of blocks according to minimum radius requirements.
- Rotate blocks and move forward to fully engage both knobs below (typical).

**ISOMETRIC VIEW**

- Place 18" high piece of non-woven geotextile fabric (WSDOT 9-33.2(2) - Table 7) in joint between blocks (typical).

**TOP VIEW**

- Place stone in joint between blocks.

**NOTES:**

- Place blocks tight together.
- No minimum radius - based only on block geometry.
- Smaller radii will result in more exposed untextured block surface.

**MINIMUM RADIUS AND OFFSET FOR BOTTOM ROW**

<table>
<thead>
<tr>
<th>NUMBER OF COURSES</th>
<th>HEIGHT OF BLOCKS (FROM FACE OF BLOCK)</th>
<th>OFFSET</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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</tr>
<tr>
<td>2</td>
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<td>18'-0&quot;</td>
<td>±1/4&quot;</td>
</tr>
<tr>
<td>20</td>
<td>19'-0&quot;</td>
<td>±1/4&quot;</td>
</tr>
</tbody>
</table>

**COMPLETED CORNER**

- Running bond shifts ±1/2" further with every row.
- Offset from theoretical corner (see chart).

**EXPOSED UNTEXTURED BLOCK SURFACE**

- Top blocks tight together.
- No minimum radius - based only on block geometry.
- Smaller radii will result in more exposed untextured block surface.

**EXPOSED UNTEXTURED BLOCK SURFACE**

- Top blocks tight together.
- No minimum radius - based only on block geometry.
- Smaller radii will result in more exposed untextured block surface.
PIPES 18" DIA. OR SMALLER INSTALLED AT A SKewed ANGLE TO THE WALL

CAST-IN-PLACE CONCRETE COLLAR AROUND PIPE 24" WIDE MIN.

CONTOUR GEOGRID ABOVE AND BELOW PIPE (1V : 4H MAX.)

28" PC BLOCK

SECTION A-A

PROFILE VIEW

PIPES LARGER THAN 18" DIA. MAY NOT BE INSTALLED AT A SKewed ANGLE TO THE WALL

3D VIEW FROM BACK

GEOTEXTILE DESIGN MANUAL M 46-03.13
Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls
December 2020

Appendix 15-Q

REDI-ROCK PC SERIES

CULVERT (0° TO 45° SKEW)

18" REINFORCED CONCRETE PIPE SHOWN

28" PC BLOCKS

REDI-ROCK PC SERIES

17/01/15

REDI-ROCK PC SERIES LEDGER 10014.pdf
REINFORCEMENT PLACEMENT AROUND VERTICAL OBSTRUCTIONS
With the Redi-Rock PC Series

LARGE OBSTRUCTION - CONCEPTUAL DETAIL

NOTE:
(1) ALL STRUCTURAL STEEL ELEMENTS TO BE HOT-DIPPED GALVANIZED IN ACCORDANCE WITH ASTM A123.
(2) THE ABOVE DETAIL IS VALID ONLY FOR OBSTRUCTIONS 84" AND SMALLER AND/OR PC WALL SECTIONS THAT DO NOT REQUIRE GEOGRID REINFORCEMENT WITH TENSILE STRENGTH HIGHER THAN 8XT.

PREAPPROVED WALL APPENDIX: SPECIFIC REQUIREMENTS AND DETAILS FOR REDI-ROCK POSITIVE CONNECTION WALLS

Page 15-Q-12

Geotechnical Design Manual M 46-03.13
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Redi-Rock International follows the recommendations of FHWA GEC 011 and discourages placing pipes or other horizontal obstructions behind the wall in the reinforced soil zone. Placing pipes in this zone could lead to maintenance problems and potential wall failure.

**Utilities in the Reinforced Soil Zone**

- Keep sufficient separation to meet maximum geogrid slope and clearance requirements.
- See contract documents for WSDOT approved backfill around pipe.
- Wrap pipe joints with non-woven geotextile fabric.
- Maintain 3” min. between geogrid and pipe.
- Storm or sanitary sewer pipe installed parallel to wall.
- Wrap pipe joints with non-woven geotextile fabric.
- Maintain 3” min. between geogrid and pipe.

**Dry Utilities (Electric, Gas, Telecommunications)**

- Install geogrid strips above and below pipe.
- Keep sufficient separation to meet maximum geogrid slope and clearance requirements.

**Concrete (Cast-In-Place Around Pipe)**

- Steel reinforcement shall be submitted based upon project specific requirements.

**Control Joint (If Needed).**

- Line up joints between units to create control joints.

**Pipe Protruding Through Wall**

- (48” Dia Concrete Pipe Shown) Leveling pad or lower courses of Redi-Rock blocks.

**Culvert Penetration Detail - Plan View**

- No Scale.
- Remove minimum number of blocks required to fit pipe through wall.

**Redi-Rock PC Block Wall**

- Concrete (Cast-In-Place Around Pipe)
CAST-IN-PLACE COPING DETAIL

NO SCALE

SECTION A-A
(JUST BEFORE STEP DOWN ON TOP OF WALL)

SECTION B-B
(JUST AFTER STEP DOWN ON TOP OF WALL)

LENGTH OF COPING SECTIONS VARIES

ELEVATION VIEW

NOTE:
- This drawing is for reference only.
- Final designs for construction must be prepared by a registered Professional Engineer using the actual conditions of the proposed site.
- Final wall design must address both internal and external drainage and shall be evaluated by the Professional Engineer who is responsible for the wall design.

RAILING DESIGNED TO PROJECT REQUIREMENTS

SIDEWALK OR GRASS SURFACE ON TOP OF WALL PER PROJECT DESIGN

FREESTANDING BLOCKS USED WHERE BLOCK IS EXPOSED AND TEXTURED SURFACE IS REQUIRED ON BOTH SIDES OF WALL

EXPANSION JOINT MATERIAL BETWEEN COPING SECTIONS. LOCATE AT ELEVATION CHANGES IN WALL (20.0 FT. MAX.)

HEIGHT VARIES ALONG WALL 14" (MIN) TO 32" (MAX)

#4 STIRRUP @ 12" O.C.
#6 BARS

6" CAP BLOCK - 2 SIDED
6" CAP BLOCK - 3 SIDED
6" PRECAST CAP UNIT

ATTACH WITH EXTERIOR GRADE CONCRETE BONDING ADHESIVE THAT WILL PROVIDE MINIMUM TENSIILE BOND STRENGTH OF 300 PSI @ 7 DAYS PER ASTM C1583 (QUIKRETE CONCRETE BONDING ADHESIVE NO. 9182 OR EQUAL)

SETBACK = 0'

POSTS @ 46-1/8" O.C. (MAX) SEE SHEET 15 OF 17 FOR POST DETAIL

POSTS @ 46-1/8" O.C. (MAX) SEE SHEET 15 OF 17 FOR POST DETAIL

SETBACK = 2 7/8" (15" KNOB)
SETBACK = 1 5/8" (7 1/2" KNOB)

#6 BARS

GROUND

5" (TYP)

1 3/4" FACE (TYP.)

20" (MIN) TO 32" (MAX)

SECTION A-A
(JUST BEFORE STEP DOWN ON TOP OF WALL)

SECTION B-B
(JUST AFTER STEP DOWN ON TOP OF WALL)

LENGTH OF COPING SECTIONS VARIES

ELEVATION VIEW

FREESTANDING BLOCKS USED WHERE BLOCK IS EXPOSED AND TEXTURED SURFACE IS REQUIRED ON BOTH SIDES OF WALL

EXPANSION JOINT MATERIAL BETWEEN COPING SECTIONS. LOCATE AT ELEVATION CHANGES IN WALL (20.0 FT. MAX.)

HEIGHT VARIES ALONG WALL 14" (MIN) TO 32" (MAX)

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#6 BARS

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ATTACH WITH EXTERIOR GRADE CONCRETE BONDING ADHESIVE THAT WILL PROVIDE MINIMUM TENSIILE BOND STRENGTH OF 300 PSI @ 7 DAYS PER ASTM C1583 (QUIKRETE CONCRETE BONDING ADHESIVE NO. 9182 OR EQUAL)

SETBACK = 0'

POSTS @ 46-1/8" O.C. (MAX) SEE SHEET 15 OF 17 FOR POST DETAIL

POSTS @ 46-1/8" O.C. (MAX) SEE SHEET 15 OF 17 FOR POST DETAIL

SETBACK = 2 7/8" (15" KNOB)
SETBACK = 1 5/8" (7 1/2" KNOB)

#6 BARS

GROUND

5" (TYP)

1 3/4" FACE (TYP.)

20" (MIN) TO 32" (MAX)

SECTION A-A
(JUST BEFORE STEP DOWN ON TOP OF WALL)

SECTION B-B
(JUST AFTER STEP DOWN ON TOP OF WALL)

LENGTH OF COPING SECTIONS VARIES

ELEVATION VIEW

FREESTANDING BLOCKS USED WHERE BLOCK IS EXPOSED AND TEXTURED SURFACE IS REQUIRED ON BOTH SIDES OF WALL

EXPANSION JOINT MATERIAL BETWEEN COPING SECTIONS. LOCATE AT ELEVATION CHANGES IN WALL (20.0 FT. MAX.)

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6" CAP BLOCK - 3 SIDED
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SETBACK = 0'

POSTS @ 46-1/8" O.C. (MAX) SEE SHEET 15 OF 17 FOR POST DETAIL

POSTS @ 46-1/8" O.C. (MAX) SEE SHEET 15 OF 17 FOR POST DETAIL

SETBACK = 2 7/8" (15" KNOB)
SETBACK = 1 5/8" (7 1/2" KNOB)

#6 BARS

GROUND

5" (TYP)

1 3/4" FACE (TYP.)

20" (MIN) TO 32" (MAX)

SECTION A-A
(JUST BEFORE STEP DOWN ON TOP OF WALL)

SECTION B-B
(JUST AFTER STEP DOWN ON TOP OF WALL)

LENGTH OF COPING SECTIONS VARIES

ELEVATION VIEW

FREESTANDING BLOCKS USED WHERE BLOCK IS EXPOSED AND TEXTURED SURFACE IS REQUIRED ON BOTH SIDES OF WALL

EXPANSION JOINT MATERIAL BETWEEN COPING SECTIONS. LOCATE AT ELEVATION CHANGES IN WALL (20.0 FT. MAX.)

HEIGHT VARIES ALONG WALL 14" (MIN) TO 32" (MAX)

#4 STIRRUP @ 12" O.C.
#6 BARS

6" CAP BLOCK - 2 SIDED
6" CAP BLOCK - 3 SIDED
6" PRECAST CAP UNIT

ATTACH WITH EXTERIOR GRADE CONCRETE BONDING ADHESIVE THAT WILL PROVIDE MINIMUM TENSIILE BOND STRENGTH OF 300 PSI @ 7 DAYS PER ASTM C1583 (QUIKRETE CONCRETE BONDING ADHESIVE NO. 9182 OR EQUAL)

SETBACK = 0'

POSTS @ 46-1/8" O.C. (MAX) SEE SHEET 15 OF 17 FOR POST DETAIL

POSTS @ 46-1/8" O.C. (MAX) SEE SHEET 15 OF 17 FOR POST DETAIL

SETBACK = 2 7/8" (15" KNOB)
SETBACK = 1 5/8" (7 1/2" KNOB)

#6 BARS

GROUND

5" (TYP)

1 3/4" FACE (TYP.)

20" (MIN) TO 32" (MAX)
**REDI-ROCK PC SERIES**

**TOP BLOCK GRADING OPTIONS**

**FINISH GRADE TO EDGE OF BLOCKS**

**TOP OF WALL TREATMENT USING REDI-ROCK TOP AND GARDEN BLOCKS INSTEAD OF COPING**

**MIDDLE BLOCK**

**BOTTOM BLOCK**

**TOP BLOCK**

**HALF GARDEN CORNER BLOCK AT END OF EACH ROW (TYPICAL)**

**SARCUT AND REMOVE INSIDE EDGE OF HALF GARDEN CORNER BLOCK AND FILL WITH TOPSOIL (RECOMMENDED)**

**GRADE DROPS ALONG SIDE OF HALF GARDEN BLOCK**

**GRADE DROPS ALONG BACK AND END OF GARDEN BLOCK**

**ALTERNATE GARDEN BLOCK PLACEMENT**

**DRAINAGE SWALE BEHIND WALL**

**SLOPE VARIES - SEE PLANS**

**3'-0" VARIES WITH CROSS-SLOPE**

**GRADE ON SWALE (2'-10" MINIMUM)**

**SLOPE VARIES - SEE PLANS**

**8" VARIES WITH SLOPE**

**24" 6" MIN.**

**30 mil PVC OR EPDM GEOMEMBRANE**

**GEOGRID STRIPS (TYPICAL)**

**GRASSED SWALE**

**GRADE SWALE CROSS-SLOPE AS NECESSARY TO PROVIDE MINIMUM 1% TO 2% FALL PARALLEL TO WALL**

**GRADE SWALE AROUND BLOCKS IN STEP DOWN AREAS**

**ROCK CHECK DAMS AS REQUIRED**

**PLACE GEOMEMBRANE OR PROVIDE MIN. OF 3" OF SOIL BETWEEN CIP CONCRETE AND GEOGRID STRIPS**

**CONCRETE SWALE**

**HALF GARDEN CORNER BLOCKS**

**TOP OF BLOCK PLACEMENT PC - LEDGER 100814.dwg**
HAND RAIL FOR POSITIVE CONNECTION (PC) BLOCKS

- Hand rail post spacing 46-1/8" O.C.
- Core drill blocks 1 in larger than diameter of post grout annulus with non-shrink grout.
- 4" concrete walk (typ.)
- Hand rail for positive connection (PC) blocks

CUSTOM WEEP HOLE PIPE CAST IN BLOCK

- Solid PVC or HDPE drain pipe cast into block
- DIA. = 3" or 4" as specified on plans
- Locate center of pipe 6” left and 6.75” down from top right corner of block
- Pipe to extend 6” to 8” from back of block for connection to perforated wall drain

FIELD INSTALLED WEEP HOLE PIPE

- Notch ± 2.5" x 5" hole in side of a REDI-ROCK block
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place
- Connect to perforated wall drain

- 4" CONCRETE WALK (TYP.)
- NO SCALE
- HAND RAIL POST SPACING 46-1/8" O.C.
- CORE DRILL BLOCKS 1 IN LARGER THAN DIAMETER OF POST GROUT ANNULUS WITH NON-SHRINK GROUT.
- 4" PC MODULE UNIT NO GEOGRID REINFORCEMENT STRIP ATTACHED TO THIS UNIT
- 2" PC MODULE UNIT GEOGRID REINFORCEMENT
- FREE-STANDING UNIT
POST AND BEAM GUARDRAIL INSTALLATION

SECTION VIEW

POST AND BEAM GUARDRAIL

3' MIN. (FROM BACK OF BLOCK)

GEOGRID STRIPS

SECTION VIEW

UPPER LEG OF STRIP (INSTALLED AT TOP OF BLOCK ELEVATION)

LOWER LEG OF STRIP (INSTALLED AT BOTTOM OF BLOCK ELEVATION)

INSTALL POSTS OR SLEEVE WHILE CONSTRUCTING THE WALL

GEOGRID INSTALLED ON BLOCK ONE LAYER DOWN (TYPICAL)

DIVIDE GEOGRID STRIPS AS NEEDED TO INSTALL AT LEAST 3" BELOW PAVEMENT AND BASE CONCRETE CURB AND GUTTER (PITCH-OUT CURB SHOWN)

PAVEMENT SECTION BELOW ELEVATION OF TOP GEOGRID LAYER

CONCRETE CURB AND GUTTER (PITCH-OUT CURB SHOWN)

PAVEMENT

GUARDRAIL

GEOTEXTILE GEONET 2000

GEOTEXTILE GEONET 2000
## REDI-ROCK PC CAP & CORNER UNIT BLOCKS

<table>
<thead>
<tr>
<th>Block Type</th>
<th>Volume</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Two-Sided Cap</td>
<td>4.68 ft³</td>
<td>±669 lbs</td>
</tr>
<tr>
<td>Two-Sided Curve Cap</td>
<td>4.81 ft³</td>
<td>±688 lbs</td>
</tr>
<tr>
<td>Three-Sided Cap</td>
<td>4.68 ft³</td>
<td>±669 lbs</td>
</tr>
<tr>
<td>Four-Sided Cap</td>
<td>4.81 ft³</td>
<td>±688 lbs</td>
</tr>
<tr>
<td>Two-Sided Half</td>
<td>2.25 ft³</td>
<td>±322 lbs</td>
</tr>
<tr>
<td>Three-Sided Half</td>
<td>2.43 ft³</td>
<td>±347 lbs</td>
</tr>
<tr>
<td>Four-Sided Cap</td>
<td>4.81 ft³</td>
<td>±688 lbs</td>
</tr>
<tr>
<td>Garden Corner</td>
<td>8.26 ft³</td>
<td>±1182 lbs</td>
</tr>
<tr>
<td>Half Garden Corner</td>
<td>4.25 ft³</td>
<td>±607 lbs</td>
</tr>
</tbody>
</table>

### Notes:
- Volumes listed are similar to those in the blocks as shown.
- Actual weights can differ, as may vary.
- Weight shown is based on 143 pcf concrete.

---

**REDI-ROCK PC SERIES CAP AND CORNER COMPONENT DETAILS**

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Geotechnical Design Manual
M 46-03.13

[Company Logo]

Dimensions Updated 11/20/13
Appendix 15-R  Preapproved Wall Appendix: Specific Requirements and Details for Lock and Load Walls

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

Facing System – The wall shall be designed as a wrapped face wall system. The concrete counterfort that attaches to the facing panel shall penetrate through the geogrid reinforcement by only cutting transverse ribs as necessary to allow the counterfort to connect to the facing panel, as shown in the preapproved plans. The wall facing design shall demonstrate that the facing panel plus counterfort is stable for all limit states in accordance with the AASHTO LRFD Bridge Design Specifications, the Bridge Design Manual M 23-50, and the Geotechnical Design Manual.

Soil Reinforcement – Only geosynthetic reinforcement listed in the QPL shall be used. The ultimate and long-term design strengths specified in Appendix D of the QPL shall be used.

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications.

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

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

• WSDOT standard materials, including backfill used for the wall, shall be used where possible. With regard to the wall backfill, the entire reinforced zone for the wall shall be backfilled with WSDOT Gravel Borrow, not just the area shown in the plans (i.e., sheet 2). Where “filter fabric” is specified in the preapproved plans, it shall be a WSDOT Standard Specification Construction Geotextile for Underground Drainage material (Section 9-33).
(1) SET COUNTERFORT AND PANEL ON COMPLETED LIFT.

(2) UNROLL GEOGRID REINFORCEMENT AND POSITION IT SO THAT THE TAIL DRAPES OVER THE FACING PANEL. CUT GEOGRID ON BOTH SIDES OF EACH COUNTERFORT ONLY, AND TUCK IN SLACK GEOGRID (SEE AXON VIEW). CUT ONLY GEOGRID STRANDS THAT RUN PARALLEL TO WALL FACE AND CROSS OVER COUNTERFORT.

   PLACE BACK FILL IN A WINDDROW SLIGHTLY GREATER THAN FULL LIFT AGAINST LOCK + LOAD PANEL TO HOLD GEOGRID IN PLACE.

   ROLL GRID INTO PLACE. STAKE OR HOLD THE GEOGRID TAUT AND FREE OF WRINKLES WHILE PLACING BACKFILL. 12 INCHES AT BACK OF PANEL AND 8 INCHES OVER THE TAIL OF THE COUNTERFORT BACK.

(3) PLACE THE GEOGRID TAIL OVER THE WINDDROW, SLOPE TAIL TO TOP OF REAR OF COUNTERFORT. LOCK INTO PLACE WITH BACKFILL.

   ENTIRE REINFORCED ZONE TO BE COMPACTED TO WSDOT STANDARD SPECIFICATION FOR MSE REINFORCED ZONE.

(4) FINISHED LOCK+LOAD COURSE INSTALLATION

LOCK+LOAD LIFT ASSEMBLY DETAIL
GRID DETAIL

CUT GEOGRID ON BOTH SIDES OF COUNTERFORT TO RELIEVE TENSION ON GRID.

CUT ONLY PERPENDICULAR TO WALL PANEL

FULL UNIFORMED COMPACTION IS MANDATORY. LOCK + LOAD CANTILEVER SYSTEM MAKES IT POSSIBLE TO ACHIEVE 95% PROCTOR AT WALL FACE.
* ALL REINFORCEMENT PRODUCTS FROM WSDOT QPL LIST

ASSEMBLY DETAIL

GAVEL BARROW, WSDOT SPEC 9-03.14(4) / MAX PARTIAL SIZE

GEO GRID WRAP (MINIMUM 4 FEET)

GAVEL BARROW, WSDOT SPEC 9-03.14(4) / MAX PARTIAL SIZE

PROJECT SPECIFIED REINFORCED BACKFILL ZONE.

TYPICAL SECTION AT WALL FACE

NOTE: ENTIRE REINFORCED ZONE TO BE COMPACTED TO WSDOT STANDARD SPECIFICATION FOR MSE REINFORCED ZONE.
NOTE:
1) MINIMUM CONCRETE COMPRESSIVE STRENGTH AT 28 DAYS 5500 PSI
2) 6% AIR ENTRAINMENT
3) 3 LBS STRUCTURAL FIBER PER CUBIC YARD

NOTE:
1) MATERIAL IS 1/4" DIA. T304 115 KSI STAINLESS STEEL WIRE ASTM A580

CENTER LINE OF WIRE

LOCATION

PICTORIAL VIEW

TOP VIEW

REAR VIEW

SIDE VIEW

BAR BENDING DETAIL

4" MIN. COVER

3.5"

3.0"

6.0"

80°

90°

1/4" DIA. T304 115 KSI STAINLESS STEEL WIRE ASTM A580

7.65"

3/4" MIN. COVER

4"

3.5"

6.75" CONNECTING LOOP

5"

27"

4"

3.85"
Preapproved Wall Appendix: Specific Requirements and Details for Lock and Load Walls

Appendix 15-R

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Appendix 15-P Preapproved Wall Appendix: Specific Requirements and Details for Lock and Load Walls

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October 2013

NOTE:
1) MINIMUM CONCRETE COMPRESSIVE STRENGTH 5000 psi AT 28 DAYS
2) 6% AIR ENTRAINMENT
3) 2 LBS STRUCTURAL FIBER PER CUBIC YARD
4) FACING TEXTURE AS SPECIFIED IN CONTRACT

TOP VIEW
LEFT OUTSIDE CORNER

SIDE VIEW

CONCRETE STIFFENER FOR 90 DEGREE CORNER

STANDARD LOOP HEIGHT TOLERANCE -0.25

CONNECTING LOOP .25 DIA. 6.75' -1.40'

-11'-1.25'

6.75' -0.25'

12'-1'

16'

10.7'

5'

2.7'

1.5'

6.75' -0.25'

NOTE:
MATERIAL IS 1/4" DIA. T304 STAINLESS STEEL WIRE 115 KSI ASTM A 580
ALL DIMENSIONS ARE TO OUTSIDE TO OUTSIDE INCLUDING RADIUS
WELDING IN ACCORDANCE WITH AASHTO/ANSI D1.5M/D1.5

TOP VIEW
RIGHT OUTSIDE CORNER

TOP VIEW
LEFT OUTSIDE CORNER

TOP VIEW
FRONT VIEW

CORNER REINFORCEMENT DETAILS

SIDE VIEW

REV. DESCRIPTION DATE

LOCK+LOAD™
THE SMARTEST SYSTEM FOR SECURING AND SECURING
LOCK+LOAD™

ACCOUNTING NUMBER

TRADE NAME

QUALITY CONTROL

CERTIFICATION NUMBER

CUTTING ORDER

APPENDIX 15-R
ALL GEOGRID MUST BE 100% OVER COUNTERFORT AND EXTEND TO BACK OF PANEL AND VERTICAL FOR 4" GEOGRID 100% COVERAGE UNLESS SPECIFIED OTHERWISE.

PLAN VIEW ACUTE OUTSIDE CORNER 71-89 DEGREES

PLAN VIEW OBTUSE INSIDE CORNER 90-180 DEGREES

PLAN VIEW ACUTE INSIDE CORNER 90 DEGREES AND LESS (MINIMUM ANGLE 45°)

"3" OF BACK FILL BETWEEN LAPPED GEOGRID

PLAN VIEW ORTHOGONAL OUTSIDE CORNER 90 DEGREE

PLAN VIEW OBTUSE OUTSIDE CORNER 90-180 DEGREES

PLAN VIEW ORTHOGONAL INSIDE CORNER 90 DEGREE
OVERHANGING MOMENT SLAB BARRIER DETAIL "F" SHAPE BARRIER
MOMENT SLAB BARRIER DETAIL - STANDARD "F" SHAPE BARRIER
CROSS SECTION 24" MAX
HORIZONTAL OBSTRUCTION

NOTE:
CONCRETE FOOTING MINIMUM COMPRESSIVE STRENGTH AFTER 28 DAYS 3000 PSI
CROSS SECTION 48" MAX
HORIZONTAL OBSTRUCTION

NOTE:
CONCRETE FOOTING MINIMUM COMpressive STRENGTH AFTER 28 DAYS 3000 PSI
NOTE:
DRAWING 7 SERIES IS FOR A VERTICAL OBSTRACTION
UP TO 48" DIAMETER BUT STILL WITHIN REINFORCED SOIL ZONE.
DRAWING 7 A/B ARE VERTICAL OBSTRUCTIONS CLOSER
THAN 4' FEET FROM FACE OF WALL.

GEO GRID SOIL REINFORCEMENT WRAP BEHIND VERTICAL OBSTRUCTION
FINISH GRADE

SECTION OF VERTICAL OBSTRUCTION

SAME LENGTH AS DESIGN SPECIFIED GRIDS OR 8' MIN

GEO GRID PLACEMENT AT VERTICAL OBSTRUCTION

48' MAX
2' MIN
8' MIN
Preapproved Wall Appendix: Specific Requirements and Details for Lock and Load Walls

Appendix 15-R

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December 2020

NOTE:
MATERIAL IS 1/4" DIA. T304 STAINLESS STEEL WIRE ASTM A580
ALL DIMENSIONS OUTSIDE TO OUTSIDE INCLUDING RADIUS

ANGLE IRON, WASHERS, NUTS, BOLTS ARE ALL STAINLESS STEEL T304 ASTM
DETAIL FOR SPECIAL PANELS ONLY AROUND VERTICAL OBSTRUCTION

ASTM CODES:
A276 - WASHERS
F983 - BOLTS
F594 - NUTS
A276 - WASHERS

NOTE:
MATERIAL IS 1/4" DIA. T304 STAINLESS STEEL WIRE ASTM A580
ALL DIMENSIONS OUTSIDE TO OUTSIDE INCLUDING RADIUS

GTAW

TOP VIEW

SIDE VIEW

FRONT VIEW

PICTORIAL

PANEL CLIP DETAIL

NOTE:
MATERIAL IS 1/4" DIA. T304 STAINLESS STEEL WIRE ASTM A580
ALL DIMENSIONS OUTSIDE TO OUTSIDE INCLUDING RADIUS

SECTION

TOP VIEW

SIDE VIEW

FRONT VIEW

PICTORIAL

PANEL CLIP DETAIL

NOTE:
MATERIAL IS 1/4" DIA. T304 STAINLESS STEEL WIRE ASTM A580
ALL DIMENSIONS OUTSIDE TO OUTSIDE INCLUDING RADIUS
PARALLEL APPURtenANCE CONNECTION DETAIL

90 DEGREE APPURtenANCE CONNECTION DETAIL
Appendix 15-S  Preapproved Wall Appendix: Specific Requirements and Details for KeyGrid Walls

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

Facing Blocks – Blocks acceptable for use with this wall system are Keystone Compac II and Compac III units (block width into the wall \( W_u = 1 \) ft for both units). These blocks are for a facing batter of 1:64. Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2 ft.

Soil Reinforcement – Only geosynthetic reinforcement listed in the WSDOT QPL and which has been evaluated for connection strength with the KeyGrid wall system shall be used. The following products are approved for use with this wall system:

- Miragrid 3XT
- Miragrid 5XT
- Miragrid 7XT
- 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 the Compac II and III 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. The design facing/reinforcement connection strength shall be as specified in the following tables:

**Table 15-S-1**  Approved connection strength design values for KeyGrid walls, Compac II blocks

<table>
<thead>
<tr>
<th>Geogrid Product</th>
<th>Wall Height, H (ft)</th>
<th>Normal Load, N (lbs/ft)</th>
<th>( T_{ulcno} ) (lbs/ft)</th>
<th>( T_{lo} ) (lbs/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Miragrid 3XT</td>
<td>H &lt; 9</td>
<td>N &lt; 1074</td>
<td>915 + N tan 45°</td>
<td>3484</td>
</tr>
<tr>
<td></td>
<td>9 &lt; H &lt; 18.9</td>
<td>1074 &lt; N &lt; 2268</td>
<td>1465 + N tan 26° 2571</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 18.9</td>
<td>N &gt; 2268</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Miragrid 5XT</td>
<td>H &lt; 15.3</td>
<td>N &lt; 1837</td>
<td>1706 + N tan 20° 2686</td>
<td>4927</td>
</tr>
<tr>
<td></td>
<td>15.3 &lt; H &lt; 28.5</td>
<td>1837 &lt; N &lt; 3424</td>
<td>2020 + N tan 11°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 28.5</td>
<td>N &gt; 3424</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Miragrid 7XT</td>
<td>H &lt; 14.2</td>
<td>N &lt; 1711</td>
<td>959 + N tan 42° 3637</td>
<td>6317</td>
</tr>
<tr>
<td></td>
<td>14.2 &lt; H &lt; 28.5</td>
<td>1711 &lt; N &lt; 3417</td>
<td>1970 + N tan 26° 3845</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 28.5</td>
<td>N &gt; 3417</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Miragrid 8XT</td>
<td>H &lt; 12.5</td>
<td>N &lt; 1500</td>
<td>1064 + N tan 43° 3892</td>
<td>7897</td>
</tr>
<tr>
<td></td>
<td>12.5 &lt; H &lt; 28.2</td>
<td>1500 &lt; N &lt; 3389</td>
<td>1361 + N tan 37° 3459</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 28.2</td>
<td>N &gt; 3389</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Miragrid 10XT</td>
<td>H &lt; 8.1</td>
<td>N &lt; 970</td>
<td>1335 + N tan 49.5° 10795</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8.1 &lt; H &lt; 28.2</td>
<td>970 &lt; N &lt; 2903</td>
<td>1980 + N tan 27° 3459</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 28.2</td>
<td>N &gt; 2903</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\( N \) = normal load at reinforcement layer at facing, in lbs/ft of width parallel to face.
Table 15-S-2  Approved ultimate connection strength design values $T_{ultconn}$ for KeyGrid walls, Compac III blocks

<table>
<thead>
<tr>
<th>Geogrid Product</th>
<th>Approx. Wall Height, H (ft)</th>
<th>Normal Load, N (lbs/ft)</th>
<th>$T_{ultconn}$ (lbs/ft)</th>
<th>$T_{lot}$ (lbs/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Miragrid 3XT</td>
<td>H &lt; 9</td>
<td>N &lt; 1030</td>
<td>937 + N tan 44°</td>
<td>3484</td>
</tr>
<tr>
<td></td>
<td>H &lt; 20</td>
<td>1030 &lt; N &lt; 2268</td>
<td>1500 + N tan 22°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 20</td>
<td>N &gt; 2268</td>
<td>2416</td>
<td></td>
</tr>
<tr>
<td>Miragrid 5XT</td>
<td>H &lt; 16</td>
<td>N &lt; 1724</td>
<td>1305 + N tan 36°</td>
<td>4927</td>
</tr>
<tr>
<td></td>
<td>H &lt; 30</td>
<td>1724 &lt; N &lt; 3424</td>
<td>2045 + N tan 16°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 30</td>
<td>N &gt; 3424</td>
<td>3030</td>
<td></td>
</tr>
<tr>
<td>Miragrid 7XT</td>
<td>H &lt; 16</td>
<td>N &lt; 1681</td>
<td>1221 + N tan 37°</td>
<td>6109</td>
</tr>
<tr>
<td></td>
<td>H &lt; 30</td>
<td>N &lt; 3479</td>
<td>1642 + N tan 26°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 30</td>
<td>N &gt; 3479</td>
<td>3339</td>
<td></td>
</tr>
<tr>
<td>Miragrid 8XT</td>
<td>H &lt; 16</td>
<td>N &lt; 1695</td>
<td>1146 + N tan 42°</td>
<td>7897</td>
</tr>
<tr>
<td></td>
<td>H &lt; 30</td>
<td>N &lt; 3380</td>
<td>1657 + N tan 31°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 30</td>
<td>N &gt; 3380</td>
<td>3688</td>
<td></td>
</tr>
<tr>
<td>Miragrid 10XT</td>
<td>H &lt; 16</td>
<td>N &lt; 1695</td>
<td>1094 + N tan 45°</td>
<td>10795</td>
</tr>
<tr>
<td></td>
<td>H &gt; 16</td>
<td>1695 &lt; N &lt; 3373</td>
<td>1640 + N tan 33°</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H &gt; 30</td>
<td>3373</td>
<td>3830</td>
<td></td>
</tr>
</tbody>
</table>

$N = \text{normal load at reinforcement layer at facing, in lbs/ft of width parallel to face.}$

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

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

where,

- $T_{MARV}$ = the minimum average roll value for the ultimate geosynthetic strength $T_{ult}$
- $CR_u = T_{ultconn}/T_{lot}$, in which $T_{ultconn}$ is the ultimate connection strength and $T_{lot}$ is the lot specific ultimate tensile strength, (i.e., the lot or roll of material specific to the connection testing),
- $RF_{CR}$ = creep reduction factor for the geosynthetic, and
- $RF_D$ = the durability reduction factor for the geosynthetic.

$RF_D$ shall be as provided in the WSDOT QPL, Appendix D (i.e., $RF_D = 1.3$). $RF_{CR}$ for the connection strength shall be equal to 1.2 for connections with Compac II and Compac III blocks based on long-term connection strength tests conducted for some of the block/geogrid combinations tested.
Approved details for the KeyGrid wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details and the wall system in general are as follows:

- Drawings 5A and 5B: Cords in the wall facing alignment to form a radius shall be no shorter than the roll width of the geosynthetic reinforcing.
- Applies to retaining wall heights up to a maximum of 33 feet.
- Applies to retaining walls having a wall face batter of 1H:64V.
- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet.
- The specifications for the fiberglass pins shall match the technical requirements submitted during the preapproval process.
- It is noted in ASTM C1372 that a dimensional tolerance for the height of the block of $\frac{1}{8}$ inch is allowed, but that WSDOT GDM Section 15.5.3.8 recommends a tighter dimensional tolerance of $\frac{1}{16}$ inch. Based on WSDOT experience, for walls greater than 25 ft in height, some cracking of facing blocks due to differential vertical stresses tends to occur in the bottom portion of the wall. Therefore, blocks placed at depths below the wall top of 25 ft or more should be cast to a vertical dimensional tolerance of $\frac{1}{16}$ inch to reduce the risk of significant cracking of facing blocks.
Appendix 15-T  
Preapproved Wall Appendix: Specific Requirements and Details for Basalite GEOWALL

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 the GEOWALL Pro, GEOWALL Max, and GEOWALL Max II. Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2.7 ft (i.e., up to four 8 in. thick blocks). Blocks are set at a near vertical 1H:64V batter. Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2 ft for the GEOWALL Pro (i.e., 2*W_u), and 2.7 ft for the GEOWALL Max and GEOWALL Max II.

**Soil Reinforcement** – Only geosynthetic reinforcement listed in the WSDOT QPL and which has been evaluated for connection strength with the Basalite GEOWALL system shall be used. The following specific products that are approved for use with this wall system:

- Miragrid 3XT  Stratagrid SG200
- Miragrid 5XT  Stratagrid SG350
- Miragrid 7XT  Stratagrid SG500
- Miragrid 8XT  Stratagrid SG550
- Miragrid 10XT  Stratagrid SG600

**Reinforcement/Facing Block Connection Requirements** – The connection between Basalite GEOWALL 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:

<table>
<thead>
<tr>
<th>Reinforcement/Facing Block Connection Requirements</th>
<th>Specific Requirements and Details for Basalite GEOWALL</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>T_{ac}, the long-term connection strength, shall be calculated as follows:</strong></td>
<td></td>
</tr>
<tr>
<td>$T_{ac} = \frac{T_{MARV} \cdot CR_u}{RF_{CR} \cdot RF_D}$</td>
<td>(15-T-1)</td>
</tr>
</tbody>
</table>

where,

- $T_{MARV}$ = the minimum average roll value for the ultimate geosynthetic strength $T_{ult}$
- $CR_u = \frac{T_{ultconn}}{T_{lot}}$ in which $T_{ultconn}$ is the ultimate connection strength and $T_{lot}$ is the lot specific ultimate tensile strength, (i.e., the lot or roll of material specific to the connection testing),
- $RF_{CR}$ = creep reduction factor for the geosynthetic, and
- $RF_D$ = the durability reduction factor for the geosynthetic.

$RF_{CR}$ and $RF_D$ shall be as provided in the WSDOT QPL, Appendix D.
### Table 15-T-1  Approved connection strength design values for Basalite GEOWALL

<table>
<thead>
<tr>
<th>SRW Facing Unit</th>
<th>Geogrid Product Line</th>
<th>Geogrid Product Designation</th>
<th>Normal Load, N (lbs/ft)</th>
<th>$\sigma_{ultconn}$ (lbs/ft)</th>
<th>$T_{lot}$ (lbs/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GEOWALL Pro $W_u = 12$ in.</td>
<td>StrataGrid</td>
<td>SG200</td>
<td>$\sigma_N &lt; 1427$ $\sigma_N &gt; 1427$</td>
<td>$\sigma_N^*\tan(38^\circ) + 756$</td>
<td>3724</td>
</tr>
<tr>
<td>GEOWALL Pro $W_u = 12$ in.</td>
<td>StrataGrid</td>
<td>SG350</td>
<td>$\sigma_N &lt; 2967$ $\sigma_N &gt; 2967$</td>
<td>$\sigma_N^*\tan(23^\circ) + 1077.5$</td>
<td>5211</td>
</tr>
<tr>
<td>GEOWALL Pro $W_u = 12$ in.</td>
<td>StrataGrid</td>
<td>SG500</td>
<td>$\sigma_N &lt; 2983$ $\sigma_N &gt; 2983$</td>
<td>$\sigma_N^*\tan(30^\circ) + 1060$</td>
<td>6751</td>
</tr>
<tr>
<td>GEOWALL Pro $W_u = 12$ in.</td>
<td>StrataGrid</td>
<td>SG550</td>
<td>$\sigma_N &lt; 3100$ $\sigma_N &gt; 3100$</td>
<td>$\sigma_N^*\tan(36^\circ) + 1076$</td>
<td>8247</td>
</tr>
<tr>
<td>GEOWALL Pro $W_u = 12$ in.</td>
<td>StrataGrid</td>
<td>SG600</td>
<td>$\sigma_N &lt; 3000$ $\sigma_N &gt; 3000$</td>
<td>$\sigma_N^*\tan(33^\circ) + 1252$</td>
<td>9553</td>
</tr>
<tr>
<td>GEOWALL Pro $W_u = 12$ in.</td>
<td>Mirafi</td>
<td>3XT</td>
<td>$\sigma_N &lt; 1975$ $\sigma_N &gt; 1975$</td>
<td>$\sigma_N^*\tan(38^\circ) + 1060$</td>
<td>3994</td>
</tr>
<tr>
<td>GEOWALL Pro $W_u = 12$ in.</td>
<td>Mirafi</td>
<td>5XT</td>
<td>$\sigma_N &lt; 3062$ $\sigma_N &gt; 3062$</td>
<td>$\sigma_N^*\tan(29^\circ) + 1339$</td>
<td>5334</td>
</tr>
<tr>
<td>GEOWALL Pro $W_u = 12$ in.</td>
<td>Mirafi</td>
<td>7XT</td>
<td>$\sigma_N &lt; 2776$ $\sigma_N &gt; 2776$</td>
<td>$\sigma_N^*\tan(35^\circ) + 1087$</td>
<td>6442</td>
</tr>
<tr>
<td>GEOWALL Pro $W_u = 12$ in.</td>
<td>Mirafi</td>
<td>8XT</td>
<td>$\sigma_N &lt; 3100$ $\sigma_N &gt; 3100$</td>
<td>$\sigma_N^*\tan(38^\circ) + 1178$</td>
<td>7898</td>
</tr>
<tr>
<td>GEOWALL Pro $W_u = 12$ in.</td>
<td>Mirafi</td>
<td>10XT</td>
<td>$\sigma_N &lt; 3003$ $\sigma_N &gt; 3003$</td>
<td>$\sigma_N^*\tan(36^\circ) + 1130$</td>
<td>10973</td>
</tr>
<tr>
<td>GEOWALL Max $W_u = 21$ in</td>
<td>StrataGrid</td>
<td>SG200</td>
<td>$1.75\sigma_N &lt; 1643$ $1.75\sigma_N &gt; 1643$</td>
<td>$(1.75\sigma_N^*\tan(37^\circ) + 1246$</td>
<td>3724</td>
</tr>
<tr>
<td>GEOWALL Max $W_u = 21$ in</td>
<td>StrataGrid</td>
<td>SG350</td>
<td>$1.75\sigma_N &lt; 2777$ $1.75\sigma_N &gt; 2777$</td>
<td>$(1.75\sigma_N^*\tan(31^\circ) + 1471$</td>
<td>5211</td>
</tr>
<tr>
<td>GEOWALL Max $W_u = 21$ in</td>
<td>StrataGrid</td>
<td>SG500</td>
<td>$1.75\sigma_N &lt; 2674$ $1.75\sigma_N &gt; 2674$</td>
<td>$(1.75\sigma_N^*\tan(33^\circ) + 1605$</td>
<td>6751</td>
</tr>
<tr>
<td>GEOWALL Max $W_u = 21$ in</td>
<td>StrataGrid</td>
<td>SG550</td>
<td>$1.75\sigma_N &lt; 2796$ $1.75\sigma_N &gt; 2796$</td>
<td>$(1.75\sigma_N^*\tan(41^\circ) + 1580$</td>
<td>8427</td>
</tr>
<tr>
<td>GEOWALL Max $W_u = 21$ in</td>
<td>StrataGrid</td>
<td>SG600</td>
<td>$1.75\sigma_N &lt; 2799$ $1.75\sigma_N &gt; 2799$</td>
<td>$(1.75\sigma_N^*\tan(44^\circ) + 1768$</td>
<td>9553</td>
</tr>
<tr>
<td>GEOWALL Max $W_u = 21$ in</td>
<td>Mirafi</td>
<td>3XT</td>
<td>$1.75\sigma_N &lt; 1651$ $1.75\sigma_N &gt; 1651$</td>
<td>$(1.75\sigma_N^*\tan(45^\circ) + 1314$</td>
<td>3994</td>
</tr>
<tr>
<td>GEOWALL Max $W_u = 21$ in</td>
<td>Mirafi</td>
<td>5XT</td>
<td>$1.75\sigma_N &lt; 1941$ $1.75\sigma_N &gt; 1941$</td>
<td>$(1.75\sigma_N^*\tan(64^\circ) + 23$</td>
<td>5334</td>
</tr>
<tr>
<td>GEOWALL Max $W_u = 21$ in</td>
<td>Mirafi</td>
<td>7XT</td>
<td>$1.75\sigma_N &lt; 2700$ $1.75\sigma_N &gt; 2700$</td>
<td>$(1.75\sigma_N^*\tan(44^\circ) + 1611$</td>
<td>6442</td>
</tr>
<tr>
<td>GEOWALL Max $W_u = 21$ in</td>
<td>Mirafi</td>
<td>8XT</td>
<td>$1.75\sigma_N &lt; 2763$ $1.75\sigma_N &gt; 2763$</td>
<td>$(1.75\sigma_N^*\tan(52^\circ) + 1294$</td>
<td>7898</td>
</tr>
<tr>
<td>GEOWALL Max $W_u = 21$ in</td>
<td>Mirafi</td>
<td>10XT</td>
<td>$1.75\sigma_N &lt; 2226$ $1.75\sigma_N &gt; 2226$</td>
<td>$(1.75\sigma_N^*\tan(53^\circ) + 1240$</td>
<td>10973</td>
</tr>
<tr>
<td>GEOWALL Max II, $W_u = 18$ in</td>
<td>StrataGrid</td>
<td>SG200</td>
<td>$1.5\sigma_N &lt; 2700$ $1.5\sigma_N &gt; 2700$</td>
<td>$(1.5\sigma_N^*\tan(15^\circ) + 1540$</td>
<td>3724</td>
</tr>
</tbody>
</table>
Table 15-T-1  Approved connection strength design values for Basalite GEOWALL

<table>
<thead>
<tr>
<th>SRW Facing Unit</th>
<th>Geogrid Product Line</th>
<th>Geogrid Product Designation</th>
<th>Normal Load, N (lbs/ft)</th>
<th>$T_{\text{ultconn}}$ (lbs/ft)</th>
<th>$T_{\text{lot}}$ (lbs/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GEOWALL Max II, $W_u = 18$ in</td>
<td>StrataGrid</td>
<td>SG350</td>
<td>$(1.5\sigma_N) &lt; 3600$ $(1.5\sigma_N) &gt; 3600$</td>
<td>$(1.5\sigma_N) \tan(16^\circ) + 1650$</td>
<td>5211</td>
</tr>
<tr>
<td>GEOWALL Max II, $W_u = 18$ in</td>
<td>StrataGrid</td>
<td>SG500</td>
<td>$(1.5\sigma_N) &lt; 4500$ $(1.5\sigma_N) &gt; 4500$</td>
<td>$(1.5\sigma_N) \tan(24^\circ) + 1570$</td>
<td>6751</td>
</tr>
<tr>
<td>GEOWALL Max II, $W_u = 18$ in</td>
<td>StrataGrid</td>
<td>SG600</td>
<td>$(1.5\sigma_N) &lt; 6300$ $(1.5\sigma_N) &gt; 6300$</td>
<td>$(1.5\sigma_N) \tan(26^\circ) + 2125$</td>
<td>9553</td>
</tr>
</tbody>
</table>

Notes:

1. MSEW's input is in lb/ft$^2$ of surface area. Testing reports lb/ft of wall face.
2. Input is based on $W_u * N$, where $W_u$ is the width of the block into the wall in ft, to get the correct input values. Using $N$ as the normal load in the connection test (ASTM D6638), then for MSEW, $\sigma_N$ is determined as:
   a. $N$ for Geowall Pro is $1.0\sigma_N$,
   b. $N$ for GEOWALL Max is $1.75\sigma_N$,
   c. $N$ for GEOWALL Max II is $1.5\sigma_N$.
   d. The regressions used to generate the $T_{\text{ultconn}}$ equations relate the normal force on the facing blocks in lbs/ft of reinforcement width to the connection strength, in lbs/ft. For example, for (b) above, $N$ is carried by the surface area of the block and therefore $\sigma_N$ is $(N \text{ lbs/ft})/(1.75 \text{ ft})$ to get stress in psf. Therefore, to get $N$ from $\sigma_N$, use $N = 1.75\sigma_N$. 

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications.

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

- It is noted in ASTM C1372 that a dimensional tolerance for the height of the block of ¼ inch is allowed, but that WSDOT GDM Section 15.5.3.8 recommends a tighter dimensional tolerance of ⅛ inch. Based on WSDOT experience, for walls greater than 25 ft in height, some cracking of facing blocks due to differential vertical stresses tends to occur in the bottom portion of the wall. Therefore, blocks placed at depths below the wall top of 25 ft or more should be cast to a vertical dimensional tolerance of ⅛ inch to reduce the risk of significant cracking of facing blocks.

- Applies to retaining wall heights up to a maximum of 33 feet.
- Applies to retaining walls having a wall face batter of 1:64.
- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet.
- The cast-in-place concrete collar to be constructed around pipes that are protruding through the wall face is considered non-preapproved. Detailed stamped drawings and stamped engineering calculations are to be submitted for approval on a project specific basis.
- The specifications for the fiberglass pins shall match the technical requirements submitted during the preapproval process.
- The geosynthetic reinforcement strength calculations shall be based on the values provided in the latest version of the WSDOT Qualified Products List, Appendix D.
The 3 Plane units are not approved for use on WSDOT projects. See WSDOT Standard Specification 6-13.3(4).

This restriction applies to all subsequent plan sheets that show the 3 Plane unit shape.
Geogrid is to be placed on level backfill over the fiberglass pins. Place next unit, pull grid taut and backfill. Stake as required.

**Typical Reinforced Wall Section**

Near Vertical Setback

1/4" = 1'

**Strength Direction**

**Retained Zone**

Approximate Limits of Excavation

**Reinforced Soil**

Foundation Soil

Concrete Leveling Pad

**UNIT FILL TO BE WSDOT STANDARD SPECIFICATION SECTION 9.03.0(3), CRUSHED SURFACING TOP COURSE**

2" unit fill as measured from the face of units

Retained Zone

**GEOWALL™ Unit**

A back drain is suggested in a cut situation or where groundwater is present. For fill walls, the source of groundwater may not exist.

A back drain could be a geocomposite, 2 ft above the slope height; 1/3 coverage on the slope.

**Shouldered Pin**

**Section A-A**
Geowall is to be placed on level backfill over the fiberglass pins. Place next unit. Pull grid taut and backfill. Stake as required.

Typical Reinforced Wall Section
Near Vertical Setback

Section A-A

Leveling Pad Detail

8" GEOWALL™ Unit

8" step

7.5" front

A back drain is suggested in a cut condition or where groundwater is present. For fill walls, the source of ground water may not exist.

A back drain could be a geocomposite, 2/3 the slope weight, 1/3 coverage on the slope.

7.5" front

0" - 1/8"

8"

Shouldered Pin

Geostrength Direction

Retained Zone

Approximate Limits of Excavation

Permeable Soil

Reinforced Soil

Foundation Soil

Concrete Leveling Pad

Cap Unit

12" unit fill

8" Min. Low

UNIT FILL TO BE WSDOT STANDARD SPECIFICATION SECTION 5(03,15), CRUSHED SURFACING TOP COURSE
Geogrid is to be placed on level backfill over the fiberglass pins. Place next unit. Pull grid taut and backfill. Stake as required.

Typical Reinforced Wall Section
Near Vertical Setback

Leveling Pad Detail

Section A-A
NOTE:
1. FOR PIPES LARGER THAN 24", A CONCRETE COLLAR MAY BE CAST AROUND PIPE FOR EASE OF CONSTRUCTION AND APPEARANCE.
2. SAW CUT UNITS TO FIT WITHIN 1/4" OF PIPE.

SCOUR PROTECTION AS REQUIRED, USE RIP RAP OR CONCRETE SLAB IN OUTLET AREA.

CONCRETE COLLAR (IF APPLICABLE)

SCOUR PROTECTION AS REQUIRED, USE RIP RAP OR CONCRETE SLAB IN OUTLET AREA.

CONCRETE COLLAR (IF APPLICABLE)

For culverts oriented up to a 45 degree skew angle as measured from perpendicular to the wall face.

Typical Pipe Outlet Detail

Pipe Penetration

Angled Pipe Outlet Detail

SCALE: N.T.S.

SCALE: N.T.S.

www.basalite.com
SECTION B-B

9.8' (TYP)

EXP JOINT (TYP)

SECTION A-A

9.8' (TYP)

0.8' EXP JOINT (TYP)

C.I.P. SIDE COPING ELEVATION

C.I.P. TOP COPING ELEVATION

The information on this drawing is for conceptual design and should not be used without the signature of a professional engineer. Details should be specific to the project and project site requirements.
Appendix 15-T Preapproved Wall Appendix: Specific Requirements and Details for Basalite GEOWALL

Geotechnical Design Manual M 46-03.13
December 2020

Geogrid Installation on Curves and Obtuse Corners
Minimum Radius: 4 ft (PRO); 6 ft (MAX, MAX II)

Geogrid Installation at 90 deg and Acute Corners
Maximum Outside Angle: 90° (zero ft)

These details apply to the GEOWALL, GEOWALL MAX AND GEOWALL MAX II Units

GEOWALL CORNER DETAILS

The information on this drawing is for conceptual design and should not be used without the signature of a professional engineer. Details should be specific to the project and project site requirements.

www.basalite.com
Outside and Obtuse Corners

Inside and Acute Corners

Inside and Obtuse Corners

GEODGRID INSTALLATION: Place geogrid strips perpendicular to the wall face and pull back to snug the connection. Where the geogrids overlap, place 3 inches of fill between layers as noted above.

These details apply to the GEOWALL, GEOWALL MAX, AND GEOWALL MAX II Units.

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Appendix 15-T Preapproved Wall Appendix: Specific Requirements and Details for Basalite GEOWALL

Appendix 15-T

Preapproved Wall Appendix: Specific Requirements and Details for Basalite GEOWALL

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December 2020

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General Notes:


2. Concrete shall have a minimum specified compressive strength (f'c) of 4000 PSI at 28 days.

3. Deformed steel reinforcing bars shall be: ASTM A 615 or a 705, Grade 60.

Scale: 1/8" = 1'-0"

NOTE: Two geocell layers shall be used in the upper portion of the wall. Combinations shall be one of the following: R1-R3 or R2-R3

Basalite GEOWALL Pro Straight Face (shown), GEOWALL Max II & Max Similar.

GENERAL NOTES
22-1 Overview

This chapter describes the geotechnical support needed for projects where WSDOT intends to use the Design-Build (DB) method of contract delivery and the geotechnical policies that govern that support.

DB differs from traditional Design-Bid-Build (DBB) projects in that the DB team is responsible for the final design, and the means and methods needed to successfully construct the project compatible with the design. In the DBB contract method of delivery there can be a reasonable anticipation of potential means and methods that may be selected by a contractor. Hence, provided a 100% design, establishing a geotechnical baseline with respect to the subsurface and site conditions that may be encountered can be more objectively established. Of significance to the preparation of geotechnical documents for DB is that foundation types and how they are constructed may change, retaining walls may move affecting both height and wall types considered during the development of the project concept, size and location of cuts and fills may change, and any effects on adjacent sensitive structures and utilities may be significantly different than anticipated in the Conceptual Design. Right of way (ROW) lines may also be affected as well as temporary construction easements (TCE).

In DB, the Design-Build team is the responsible Engineer of Record (EOR) and has the latitude in completing the majority of the project design such that it meets the performance requirements and is in compliance with the contract documents. While the WSDOT will always retain primary ownership of the project and its long-term operations and maintenance, the DB contract delivery method allocates the majority of the responsibility and risk for project design and construction to the Design-Builder to foster innovation and creativity.

These differences relative to DBB have a fundamental effect on the type of geotechnical support needed and how it is carried out. The geotechnical support provided by the Headquarters Geotechnical Office or the department’s geotechnical consultants includes:

- A geotechnical investigation to identify site geotechnical conditions and to gather the geotechnical information needed to provide a common and consistent basis for bidding.
- Verification of the feasibility of the project Conceptual Design and identification of areas of geotechnical risk.
- The development of geotechnical Technical Requirements to be included in the Request for Proposals (RFP) as well as the Geotechnical Data Report (GDR) and Geotechnical Baseline Report (GBR) to be included as part of the contract.
- The development of Geotechnical Reference and other reference documents.
- Once the contract advertisement begins, a review of proposals, if requested by the project management; this will depend on the importance and complexity of the project geotechnical issues.
• A review of geotechnical Alternative Technical Concepts (ATCs) for consistency with the contract design requirements and WSDOT design policy.
• Review of geotechnical designs, plans, and other geotechnical submittals after award and execution.
• Project office assistance when geotechnical problems occur during the life of the project.

The chapter sections that follow address each of these areas to provide the guidance needed by the Headquarters Geotechnical Office staff and department geotechnical consultant staff to successfully develop and support department DB projects. Since this chapter is for internal geotechnical staff and internal consultant staff, and the department offices who interact with these staff, to develop and carry out DB projects, this chapter should be excluded from the Mandatory Standards that are included in the contract documents.

22-2 Definitions

Geotechnical documents provided as part of or in support of a DB project include the Geotechnical Data Report (GDR), the Geotechnical Baseline Report (GBR), Geotechnical Reference documents, and other related Reference Documents. A GDR only presents factual geotechnical and geological information obtained through site and subsurface investigation, and laboratory testing, for the project, and should not include interpretive information. The GDR is a contract document. The Geotechnical Baseline Report (GBR) is a contract document and a risk allocation document provided to Proposers of DB projects that provides the primary contractual interpretation of geotechnical conditions, in addition to the factual data provided in the GDR, for Proposers to use as the basis for their proposals. The GBR interpretation of geotechnical conditions is based on the factual information in the GDR plus interpretation of the geotechnical conditions that is not strictly based on the available factual information in the GDR. The GBR is also used after contract award for evaluating differing site conditions claims.

This GBR should not refer to any part of a reference document, as doing so will make the reference document contractual and negate its reference document status. Geotechnical Memoranda and other reference documents include other geotechnical information, interpretations, and conceptual designs that were used as the basis for evaluating the feasibility of the project Conceptual Design, and possibly alternatives to the final project Conceptual Design, and to assess areas of geotechnical risk for the project. The Geotechnical Reference documents are not included as Contract Documents, but are made available to Proposers in an appendix of the RFP for information only, not to be used as the basis for their proposal.

The geotechnical information to be included in RFP is project-specific and can include all or only some of the documents identified above. For example, during concept development for the project, it may be determined that the overall geotechnical risks are minimal, warranting only the inclusion of a GDR as a contract document and incorporating a financial allowance to manage any unforeseen risks. The level of the potential financial allowance is a decision made by the project management with input from the HQ Geotechnical and Construction Offices and the project geotechnical team.


22-2.1 Field Investigation Requirements for Pre-Advertisement Design-Build Project Documents

Past experience has demonstrated that an inadequate project geotechnical investigation can lead to excessive risk both in terms of schedule and cost. Therefore, it is important to do the right amount of geotechnical investigation to provide the subsurface information needed to help mitigate those risks. This data can then be used to develop contract information that will provide potential Proposers with a consistent understanding of the site geotechnical conditions and the impact those conditions may have on the project design and the constructability of that design. This section summarizes the level of geotechnical investigation and analysis that should be considered prior to contract advertisement for DB projects. Decisions regarding the level of geotechnical investigation needed should be developed as early in the project as possible with region project office input, including the development of a geotechnical risk profile for the project that is mutually agreed upon by both region and headquarters offices. These early efforts will also be useful to develop a strategy for establishing geotechnical baselines.

The level of geotechnical field investigation necessary for assessment of potential geotechnical risks, with consideration to the baseline configuration for the project, and for preparation of the GDR and GBR should be conducted as early in the project as possible. The goal is to leave enough time in the project development schedule for the Geotechnical Office, the region project office, and possibly others such as the HQ Construction Office and region management, to identify and come to agreement on the level of geotechnical risk WSDOT should be taking and how to allocate that risk. The project baseline configuration geotechnical investigation shall be approved by the State Geotechnical Engineer, or an approved designee. The State Geotechnical Engineer, Region/Headquarters management, and the region project team will review and agree upon the short-term (i.e., during the contract) and long-term (i.e., after the contract is completed to the end of the design life of the facility) project performance risks when determining the initial level of investigation required. During the execution of the field exploration program, field findings may significantly alter those risks and require changes to the field investigation program. The level of geotechnical investigation shall consider the amount of information necessary to develop the Conceptual Design for the DB project and also to provide the appropriate level of confidence in baseline statements and thereby reduce the risk of differing site condition claims. If there is a disagreement regarding the level of geotechnical investigation required, the issue(s) may be escalated to the next higher management level to resolve the disagreement.

The amount of geotechnical investigation needed is project specific, and shall be determined based on the guidelines provided herein.
The goals of the typical geotechnical investigation for DB projects are to:

1. Identify the distribution of soil and rock types for the Conceptual Design, and assess how the material properties will affect the design and construction of the project elements.

2. Define the ground water and surface water regimes for the project concept design. It is especially important to determine the depth, and seasonal and spatial variability, of groundwater or surface water. The locations of confined water bearing zones, artesian pressures, and seasonal or tidal variations should also be identified. The geotechnical investigation will not be sufficient to fully define these groundwater issues, but should be enough to identify potential groundwater problems and risks.

3. Identify and consider any impacts to adjacent facilities that could be caused by the construction of the Conceptual Design.

4. Identify and characterize any geologic hazards that are present within or adjacent to the project limits (e.g., landslides, rockfall, debris flows, liquefaction, soft ground or otherwise unstable soils, seismic hazards) that are already known or discovered during the baseline configuration geotechnical investigation that could affect the Conceptual Design as well as adjacent facilities that could be impacted by the construction of the Conceptual Design.

5. Assess the feasibility of the proposed alignments, including the feasibility and conceptual evaluation of retaining walls and slope angles for cuts and fills, and the effect the construction of the Conceptual Design could have on adjacent facilities.

6. Assess potential project stormwater infiltration or detention sites with regard to their feasibility, and to gather at least one year of ground water data in accordance with storm water regulations if possible within the project development schedule.

7. Identify potential suitability of on-site materials as fill, and/or the usability of nearby materials sources.

8. For structures including, but not limited to, bridges and cut-and-cover tunnels, large culverts, walls, bored tunnels, trenchless technology, provide adequate subsurface information to assess feasibility of the Conceptual Design and to help quantify risks.

9. For projects that may include ground improvement to achieve the project Concept Design, provide adequate information to assess feasibility and to assess the potential impacts to adjacent facilities due to the ground improvement.

10. For projects that may include landslides, rockfall areas, and debris flows, provide adequate information to evaluate the feasibility of various stabilization or containment techniques.
To accomplish these goals, the typical geotechnical investigation should consist of the following:

- A review of historical records of previous investigations and construction of existing facilities.
- A geological site reconnaissance of the proposed alignment, focusing on all key project features, and identification of potential hazards within and adjacent to the alignment.
- A subsurface investigation consisting of an appropriate combination of borings, cone probes, field testing, field instrumentation (such as piezometers or inclinometers), geophysical surveys, and laboratory testing.

As a starting point, utilize existing subsurface information from records and augment that information with additional borings, cone probes and/or geophysical surveys to fill in gaps in the existing information.

Typically, to produce a GDR and GBR to support a 15 to 30% project design, a 50 percent or greater level geotechnical subsurface field investigation (including any existing (historical) borings that can be relied upon) is typically needed relative to a full PS&E level geotechnical investigation for final design as defined elsewhere in the GDM and referenced documents. The actual subsurface investigation conducted for a specific project may vary significantly from this target, however, depending on the uncertainty in the details of the Conceptual Design, the potential for variations in alignments and structure locations, the complexity of the site and project, the availability of preexisting subsurface information, and the potential for risk. As stated above, the level of geotechnical investigation undertaken should be developed collaboratively with the Region Project Office, as well as managers in the Region and in Headquarters as needed, based on the level of risk WSDOT should be taking.

Any new boring logs produced shall be consistent with the requirements in Chapter 4.

The geotechnical investigation may also include an assessment of the potential to encounter hazardous waste, since that potential and its location may be strongly tied to the subsurface stratigraphy and ground water regime. However, Environmental Services, and/or the region, or their consultants, have the lead in such investigations, working as a team with the Headquarters Geotechnical Office to complete that work. From a contract standpoint, it is desirable to "baseline" the hazardous/contaminated materials/water in the same manner that the geotechnical project attributes are baselined. It is also desirable from a contract standpoint that this hazardous/contaminated materials/water information be consolidated in one place in the contract. The decision of whether this is captured in the GBR or an Environmental hazardous/contaminated materials/water baseline report should be coordinated with Environmental Services.

Regarding historical and subsurface investigations to assess the potential to encounter archeological artifacts, such investigations are conducted through environmental Services, the region, or their consultants. In general, the results of archeological investigations will not be included in the GDR, GBR, and Geotechnical Memoranda for WSDOT DB projects, but are contained in a separate report.
It should be recognized that at the time of the field exploration many of the project Conceptual Design features investigated may not be defined. The geotechnical engineer developing the GBR will have to utilize professional judgment in addition to assistance from the WSDOT project team to assess what project elements for the Conceptual Design are to be investigated and where they will likely be located in order to perform an adequate field investigation. When developing the exploration plan to investigate the project Conceptual Design, or other specific concept alternatives requested by the WSDOT project office, ensure that the plan is sufficient to develop an overall characterization of the project corridor, and also sufficient as a basis for pricing the final Conceptual Design portrayed in the RFP.

Risks to be considered that could require a more detailed investigation than what may be considered typical shall include, but not be limited to, the following:

- Liquefaction and other seismic hazards.
- Very soft soils.
- Areas of previous or potential instability (e.g., Landslides, rockfall, severe erosion).
- Site and soil conditions that may affect constructability.
- High groundwater, or complex groundwater regime.
- Shallow bedrock surface that is highly variable either in depth from the surface or in quality/strength.

The degree of investigation necessary to properly define and allocate these risks depends on the nature of the risk, the amount of detailed geotechnical information needed to mitigate that risk, and the impact such risks have on the potential project costs. To determine the amount of geotechnical investigation required, consider the impact of such conditions on the ability of Proposers to adequately estimate project costs and project staging/scheduling. It will remain up to the Design-Builder to assess the limitations in the exploration program provided in the RFP and perform the requisite explorations to be compliant with the GDM and AASHTO requirements during final design.

22-3 Purpose and Content of the Geotechnical Reports Included in the Contract Documents

In general, this section follows the guidelines provided in Essex, et al. (2007) as published by the American Society of Civil Engineers. As specifically applied to WSDOT DB projects, the geotechnical reports included in the contract documents shall be as described in this section.

Geotechnical Data Report (GDR) – The GDR contains all the factual geotechnical data gathered for the project, and shall be included as part of the project contract. The GDR should contain the following information:

- A description of the geotechnical site exploration program, including any explanatory information needed to understand the boring logs and in-situ field test logs.
- The logs of all borings, logs of other subsurface investigation techniques such as cone or geophysical, test pits, and other site investigations, including any existing subsurface geotechnical data.
- Ground water measurements.
• A description of the geologic and seismic setting for the project corridor (at a regional level).

• Results of all field tests conducted, including description and results.

• Installation details, logs, and measurements results of all geotechnical field instrumentation installed for the project or existing geotechnical instrumentation and measurement results usable for the project.

• A description of all laboratory tests conducted and the test results, as well as any previous geotechnical laboratory test results that are relevant for the project.

Existing boring and other subsurface data that are available within the project corridor should not be included in the GDR unless their level of accuracy is consistent with the new subsurface data obtained for the project. This older data should be included in a separate appendix to the RFP as an historical geotechnical reference document that is available to proposers as background information only, not part of the contract, and not be used to determine differing site conditions.

The GDR may also include subsurface profiles and cross-sections at key locations within the project limits, provided that subsurface data interpretations such as interpolation between borings to develop stratigraphy, as well as the geologic interpretation of the strata, are not done. In this case, boring logs are presented in a way that shows spatial relationships between the borings, but no stratigraphic interpretation of the factual data (i.e., the boring logs) is done. This also applies to the boring logs themselves – the boring logs should not contain geological interpretations of the soil and rock units encountered, but should only present the factual observations and test data.

Alternatively, these subsurface profiles and cross-sections that include the stratigraphic and geological interpretations could be included in a separate geotechnical interpretive report (a Geotechnical Reference document) included in an Appendix to the RFP for information only.

Regarding geotechnical field tests reports for exploration methodologies such as pressuremeter testing or geophysical testing, even though the test report will likely contain an interpretation of the raw test data, such test reports should still be included with the GDR. These test interpretations are fairly standardized and are customarily considered to be factual design data in geotechnical practice.

If there is historical information about past construction, the information should be summarized and included in the GDR, especially, for example, if there were geotechnical impacts such as boulders, high groundwater, soft soils, or documented changed conditions.

**Geotechnical Baseline Report (GBR)** – The GBR is an interpretive geotechnical document used to establish a common understanding between the contractor and the owner (WSDOT) of the subsurface conditions and their potential impact and effect of risk on the design and construction of the project Conceptual Design.
The primary focus of the GBR is to establish baselines regarding geotechnical subsurface conditions present within the project, but specifically focused on the project Conceptual Design as portrayed in the RFP. These baselines should clearly define the specific geotechnical conditions the DB contractor should consider as the basis for developing their price proposal. These baselines are also used to allocate risk between the owner (WSDOT) and the contractor. The GBR baselines are not intended to be used for final design. The GDR and geotechnical data generated by the Design-Builder are used as the basis for final design. The GBR should not contain design or construction requirements; instead, design and construction requirements belong in the RFP and associated mandatory standards.

When establishing baselines in the GBR, it must be recognized that subsurface conditions are inherently variable, and that variability can translate to design and construction risk. The baseline, however, must be as clear, concise, and measurable as possible, conveying to potential Proposers what to assume about the condition being baselined (i.e., essentially, a “line in the sand”) in a way that all Proposers will understand and interpret consistently. Baselines do not necessarily need to be supported by the available technical data. Baselines are engineering interpretations or assumptions about geotechnical conditions that can affect the design of a project feature or its constructability, expressed as contractual representations of anticipated geotechnical conditions (Essex, et al., 2007). The baseline is intended to resolve, at least contractually, the uncertainty in the geotechnical data or its interpretation. Baseline statements are not required to be factual but should address specific risk elements that WSDOT requires the Design-Builder to address or consider. However, baseline statements should not be overly broad or unrepresentative of the conditions such that the risk allocation is excessively shifted to the Design-Builder. It is important that baselines be as realistic as possible.

WSDOT DB contracts allow changes to occur. These changes could occur during the procurement process by the use and approval of an Alternative Technical Concept, or during contract administration by the use of the project changes to the specifications. Both of these options are administered based on the contract documents and each process may or may not include impacts or changes related to baseline assumptions.

Baseline statements should not be considered applicable to alternate locations of the project features that may be proposed by the Design-Builder, or Work that is not in conformance with the anticipated Work. To define the locations for which the baselines are applicable, contractual baseline boundaries should be established to define the area for which the baselines are applicable. This could either be done based on the project Conceptual Design plan feature locations and maximum offsets from those feature locations, or based on a maximum offset from each boring plus an anticipated variance of strata boundaries relative to each boring, or possibly some combination of the two.

Baselines do not need to be provided for every feature in a project that could require geotechnical considerations (e.g., fills or foundations placed on very dense moist or dry soils, small walls, cuts and fills for which the risk and impact of failure is low). Only the higher risk geotechnical features and issues in a project require baselines. What specifically is to be baselined should be determined collaboratively with the project office, and others as needed. The RFP should be clear that for items not baselined, the Design-Builder assumes the risk for bid and design assumptions as well as constructed means, methods, and sequences.
Where possible, baselines should be location and, as much as possible, stratigraphic unit specific, and applicable to the type(s) of construction anticipated with consideration to the Conceptual Design for the project. However, baselines should also avoid getting into specific means and methods. For example, where the need for deep bridge foundations exists in the Conceptual Design for the project, and loose wet sand is present, the baseline should alert proposers that caving conditions are present that may need to be considered. However, the baseline should not tell the proposers to assume that full depth casing will be required to get through the caving soil. An exception to this is possibly to baseline types of construction that are likely to not be successful given the soil/rock conditions. For example, use of sheet piles that must be driven into a soil unit that is very dense or hard, or bouldery, or use of sump pumps in excavations where very permeable water bearing strata will be intersected.

In order to have baselines tied to specific subsurface conditions, a description and depth of soil and rock strata encountered in the borings should be provided. Typically, soil and rock strata locations in each boring can be summarized in a table of specific, interpreted, strata boundary locations in each of the borings. It must be clear that these strata locations are to be used only with respect to the baselines (i.e., these are Baseline Stratigraphic Units, or BSUs), and the proposers should expect the potential for those specific soil and rock strata and their depths will need adjustment for final design once the selected proposer conducts the final geotechnical explorations for the project. Stratigraphic units should not be identified in the boring logs themselves, as the additional subsurface explorations conducted by the Design-Builder for final design could require some adjustments to the stratigraphy.

Stratigraphic profiles or cross-sections in which the boring log specific BSUs discussed above are connected together to provide an overall two-dimensional stratigraphy should not be provided in the GBR. However, if the location, depth, or thickness of a high risk soil stratum in the vicinity of a specific Conceptual Design project feature such as a bridge is highly variable, the geotechnical engineer developing the GBR may need to consider including an assumed depth/location/thickness of the stratum in the baseline. This will be a risk allocation decision and as such, agreement between the Geotechnical Office, the region project design office, and possibly other offices such as Headquarters Construction and the Bridge Office should be sought before including this type of baseline in the GBR.

For project features such as walls and major cuts or fills that are not well defined and subject to significant changes relative to the project Conceptual Design, it may not be feasible to establish locations of BSUs that are specific enough to establish BSU specific baselines. In such cases, it may not even be possible to establish specific baselines, other than for known unstable areas such as landslides, or known locations of obstructions.

The baselines may draw upon data in the GDR as well as in geotechnical reference documents (see Section 22-5). However, the GBR should not specifically reference Geotechnical and other related Reference Documents that are not contractual.

Specific subject areas where baselines may be developed typically include the following, depending on the Conceptual Design and the nature of the project:

- Bridge foundation issues
- Bridge abutment and approach fill issues
- Retaining wall issues
• Seismic design issues, including liquefaction and its effects
• Embankment stability and settlement
• Cut stability
• Stormwater infiltration facilities
• Unstable slope issues and potential mitigation issues
• Ground improvement issues
• Utility impacts
• Noise wall foundation issues
• Groundwater issues, with the exception of groundwater elevation, which is specifically excluded in in RFP Section 1-04.4(5) of the RFP as being eligible for change orders
• Excavation and shoring issues, including potential dewatering issues
• Use of excavated materials
• Impact of poor ground, other than as specifically addressed above
• Known and potential obstructions
• Contaminated soils, though this is usually handled separately

In general, geotechnical design parameters (e.g., soil friction angles, earth pressures, permeability values) should not be baselined. If there is a significant risk issue associated with the selection of a geotechnical design parameter that WSDOT cannot afford to be determined by the Design-Builder as the Engineer of Record, the specification of such design parameters shall be approved by the State Geotechnical Engineer and the WSDOT project managers. These geotechnical design parameters should be described or defined in the RFP Section 2.6, and not in the GBR. Examples of this include the seismic ground response parameters for a given site, what soils are to be considered liquefiable, high risk troublesome soils such as glaciallacustrine soils as described in GDM Section 5-13.3, high risk landslide deposits, etc. This may be especially important for situations where the geotechnical designer has to use considerable judgment in establishing the design parameters, or where the design procedures and standards of practice are poorly defined.

For extremely large, complex projects, or for specific features that are long and/or uncertain as to their specific location, size, and extent of the geotechnical work needed, it may be too unwieldy to develop specific baselines for everything in the project that have significant geotechnical risks. In that case, the effort and costs expended to develop the GBR need to be strategic so that the most costly risks are addressed in enough detail to clearly apportion those risks. This strategy should be developed in collaboration with the project office and program managers. If it appears necessary to “scale down” the GBR baselins to accommodate these situations, this shall be done in consultation with the State Geotechnical Engineer and the Deputy State Construction Engineer as early as possible in the project, so that there is adequate time to make the course corrections needed for approval of the GBR baseline approach by the State Geotechnical Engineer and the Deputy State Construction Engineer to be obtained so that project development delays are avoided.

See Essex, et al. (2007) for additional guidance on developing GBRs, and their contents.
22-4  Geotechnical and Other Reference Documents

Geotechnical reference documents include interpretive or informational documents that should be made available to bidders, but that should not be considered part of the contract documents. Such documents include, but are not limited to, the following:

- Geotechnical interpretive reports containing results of preliminary geotechnical design used to establish the feasibility of the project design concept and to help quantify geotechnical risks.
- Interpretive geotechnical background information that was used to assess the feasibility of the project Conceptual Design or which could be used by Design-Builders as background information in support of their geotechnical design activities (e.g., geologic stratigraphy).
- As-built information for existing facilities within or adjacent to the project corridor that may or may not be directly affected by the project.
- Detailed construction records for existing facilities within the project corridor.
- Historical information about the project corridor.

The RFP could include as-built information and detailed construction records for existing facilities within the project corridor. In general it has been WSDOT policy to place the risk for the accuracy of as-built documents on the Design-Builder. Therefore, it is important from a contract interpretation standpoint where the as-built information is included in the RFP (e.g., in an appendix), and how it is identified in the RFP. In general, as-built information should not be included in the GBR or GDR, because doing so would place the risk of their accuracy and completeness on WSDOT.

Preliminary geotechnical engineering to develop the Conceptual Design and evaluate its feasibility during the contract development phase should be conducted. Since this is interpretive information developed for the purpose of developing the DB project documents, this information should not be included as part of the contract, but should be made available to Proposers as informational via a reference document.

The focus of any geotechnical analysis or design conducted to develop a design-build project should be to evaluate feasibility, and to assess the risk of bidders having wide swings in their bids due to geotechnical issues that have not been adequately defined. For example, if shafts or piles are proposed as foundations for a bridge, the specific foundation loads will not be known accurately enough during GBR and RFP development to determine foundation depths and sizes. Therefore, detailed analysis of foundation skin friction and end bearing resistance would be of little use. The Design-Builder would have to redo such calculations during final design anyway. What is of more use is whether or not shaft or pile foundations are feasible to install, considering impacts to adjacent facilities, ability for equipment of sufficient size to access potential pier locations, etc.
Typically, preliminary geotechnical design to assess feasibility and risk associated with the project Conceptual Design will consist of one or more of the following preliminary geotechnical design activities:

- Feasibility of proposed alignments with consideration to feasible slopes or need for walls, and the potential impact of those fill or cut slopes and walls on adjacent facilities.

- Structure foundation feasibility and risk, and potential impacts to adjacent facilities.

- Conceptual seismic hazard assessment, including site specific ground motion studies (if appropriate for the site and project scope) and the potential for liquefaction and associated seismic hazards caused by liquefaction.

- Preliminary assessment of other existing or potential geologic hazards such as landslides, rockfall, debris flows, etc., as well as the conceptual feasibility of mitigation strategies.

- Potential need for ground improvement to stabilize unstable ground, liquefaction, and excessive settlement, including the feasibility of various ground improvement techniques and their potential impact on adjacent facilities.

- Whether or not on-site materials will be usable as construction materials.

- Feasibility of site conditions present to infiltrate runoff water.

- Need for dewatering, its feasibility, and its potential impact to adjacent facilities.

- Any other preliminary geotechnical design activities needed to assess risks, to help establish baselines that will be included in the GBR, to ensure feasibility of the project Conceptual design, and to assist the WSDOT project office to develop an engineer's estimate for the project.

If there is potential for soil liquefaction at the site, a preliminary assessment of the depth and extent of the liquefiable soils should be considered. A preliminary assessment of the feasibility of potential mitigation schemes may also be considered, as well as an assessment of the impact of liquefaction on the proposed project features, depending on the impact to project feasibility. A more detailed liquefaction investigation and hazard assessment may need to be included in the contract documents to ensure bidding consistency if one or more of the following is true:

- The liquefaction hazard could affect the decision on whether to widen or replace an existing bridge or similar structure.

- The design assumptions and parameters needed to make that liquefaction assessment could vary significantly between proposers such that the project scope could vary significantly (e.g., some proposers feel no stabilization is needed, while others feel that stabilization is necessary or the bridge must be replaced rather than widened).

Similarly, for complex site conditions and large, important structures, it may be necessary to include the results of site specific seismic ground motion or seismic hazard studies in the contract documents rather than just as informational geotechnical reference documents (see Section 22-6).
22-5  Geotechnical RFP Development

The geotechnical portions of the RFP should rely heavily upon the GDM and the AASHTO Bridge Design Specifications. Since the GDM must function as both a practice manual for in-house staff and WSDOT’s geotechnical consultants and as a contract document for DB projects, the RFP should clarify how to interpret the GDM for the purposes of the DB contract, to fit the GDM within the context of the project specific contract. Furthermore, the GDM may not cover every geotechnical design situation needed in the DB project, and the RFP may need to include additional design provisions not covered by the GDM, AASHTO, or other available design specifications or manuals. The RFP essentially is contractually establishing the geotechnical engineering design requirements for the DB project.

Table 1-2, defines words used in the GDM to convey design policy (e.g., “should,” “shall,” “may”). These words also have important contractual implications in the RFP for conveying whether or not the Design-Builder has any options with regard to the specific design requirement. The GDM also identifies design policy issues and options that require specific approval from the State Geotechnical Engineer and/or Bridge Design Engineer. In such cases, as it applies to DB contracts, the Design-Builder should assume that design provisions requiring approval from the State Geotechnical Engineer and/or the Bridge Design Engineer are not approved, but can only be considered through the Alternative Technical Concepts (ATC) process. Since these address design policy issues, the State Geotechnical Engineer and/or Bridge Design Engineer in this context are not to be considered equivalent to the designer of record for the DB contractor, as decisions on these policy issues are not within the authority of the Engineer of Record.

The GDM is written to augment or supersede the AASHTO Bridge Design Specifications; therefore, if there is an apparent conflict between the GDM and the AASHTO specifications or other referenced documents, the GDM should be considered to be higher in the order of precedence than the AASHTO specifications or other referenced design documents.

With regard to the geotechnical conditions (not design and construction requirements), the GBR should be considered to be highest in the order of precedence in the RFP.

22-6  Geotechnical Investigation During RFP Advertisement

Often with DB, specific project elements cannot be reasonably defined at the time the contract documents are produced. To help minimize contingency costs in the bids and limit risk, it may be desirable to perform supplemental geotechnical investigations after the RFP has been advertised (while the bidders are preparing proposals) to augment the GDR and GBR. Whether or not supplemental geotechnical investigations should be completed during the RFP process is determined by mutual agreement between the State Geotechnical Engineer and Region/Headquarters management prior to advertisement of the RFP. The defined term for this in the RFP is as follows: Supplemental Geotechnical Data Report (SGDR). The Contract Document developed pursuant to ITP Section X.X.X, that contains factual subsurface data collected prior to the Proposal Date, and which is included in Appendix XX. Should supplemental investigation occur, the short-listed Proposers should submit requests for additional information including locations and depths of borings. The State will evaluate the requests and develop an exploration program that eliminates duplication of borings in specific locations. Doing
this will eliminate potential conflicts between Proposers, unwanted congestion due to the presence of multiple sets of drilling rigs and multiple crews, and to excessive costs through elimination of duplicated efforts. An example of Instructions to Proposers (ITP) language for a supplementary boring program is provided in Appendix 22-A.

Once the supplemental boring program is completed, the new subsurface data should be included in the GDR through a contract addendum. If the supplemental borings conflict with the GBR, an amendment to the GBR should be developed by the Headquarters Geotechnical Office or the WSDOT Geotechnical Consultant who developed the GBR and included as an addendum to the contract.

22-7 Geotechnical Support for Design-Build Projects During RFP Advertisement and Post-Award

Regarding the geotechnical review of proposals, the focus of this geotechnical support is to evaluate geotechnical aspects of the Proposal in terms of the scoring criteria spelled out in the Instructions to Proposers. Whether or not geotechnical review of bidder proposals is required will depend on the importance and complexity of the geotechnical issues in the project, and if there are any scoring criteria focused on geotechnical issues. Alternative Technical Concepts (ATCs) may also be proposed during the bidding phase. Similarly, the geotechnical support needed includes the assessment of the technical adequacy of the ATC relative to the contract design documents, or that at least the ATC will provide a level of quality that is equal to or better than the contract Conceptual Design and that is consistent with accepted design practice which in general is defined by the RFP.

Once the contract is awarded, geotechnical oversight by the owner (WSDOT) is required to ensure that the final design and its construction meet the contract requirements. This geotechnical oversight is also needed to address unanticipated site conditions (see Differing Site Conditions clause in 1-04.7 of the RFP, i.e., Request for Proposals, in WSDOT projects) and potential ambiguities in the contract specifications, if such problems occur.

From this point forward, owner (WSDOT) geotechnical support is focused on review of contractor design and construction submittals and assisting the project office with oversight to verify that the Design-Builder is appropriately addressing geotechnical design or construction problems as they come up, in accordance with the contract. The geotechnical support person must become intimately familiar with the RFP and referenced contractual documents, as those documents dictate the focus of the geotechnical submittal reviews. The geotechnical support person must consider themselves to be a member of the WSDOT project team, and the findings of their review activities are therefore provided to the WSDOT project managers for implementation. The goal is to provide the WSDOT project management with a technical assessment as to whether or not the Design-Builder met the contract technical requirements, verifying that their Quality Control/Quality Assurance (QC/QA) program with regard to geotechnical issues is being properly implemented and is effective in producing a geotechnical design that meets the contract requirements. The purpose of the geotechnical review is not to provide the DB contractor with QC/QA of their design, as the contractor is responsible for their design QC/QA.
Ordinarily, the DB Contract Technical Requirements will require the Design-Build to define a process in their Quality Management Plan for recording, logging, tracking, responding to, and resolving WSDOT design review comments. This process is managed by the Design-Build. Geotechnical comments should be incorporated into this process.

Designer preferences, or differences in opinion between the reviewer's and the Design-Build's judgments/assumptions, etc., are generally not relevant to these reviews. The focus must be on compliance of the geotechnical design/construction with the contract requirements.

This does not mean that the geotechnical support person is conducting these reviews only at the “30,000 foot level.” There may be times when the geotechnical support person must do a comparative design to figure out if the contractor’s submittal does meet the contract intent. But in other cases, an evaluation based on the reviewer's geotechnical engineering experience may be sufficient. If problems in the design start to repeat themselves, this may be an indication that either the contractor is not interpreting the contract in a way that is consistent with how WSDOT is interpreting it, or the contractor's design QC/QA is not fully functional. In such cases an oversight review (i.e., a Quality Verification, or QV, review) of the Design-Build’s QA/QC process should be conducted, documenting the review in the Construction Audit Tracking System (CATS), and issuing Non-conforming Issue Reports (NCIs) as appropriate so that the problem can be properly addressed within the provisions of the contract.

The geotechnical support person may also be involved in over-the-shoulder reviews and design task forces of the Design-Build's work as it progresses. The purpose of such reviews and involvement in the task forces is to not provide design QC/QA or technical direction to the Design-Build, but simply to work in a cooperative manner with the Design-Build to head off problems in the design before they get too far along, keeping in mind that the focus is on meeting the contract requirements.

There may be cases where the site conditions encountered by the contractor through additional subsurface explorations or during construction appear to differ from those in the contract documents. Just like any other potential differing site conditions situation, the geotechnical support person should be working with the project management team and Headquarters Construction Office to provide a technical assessment of the claim.

22-8 References


22-9 Appendices

Appendix 22-A   Example Supplemental Geotechnical Boring Program ITP Language
Appendix 22-A  Example Supplemental Geotechnical Boring Program ITP Language

Language that may be used in the ITP regarding the availability of a supplemental boring program is provided below. Note that in the first paragraph, this example language allows up to 5 borings to be selected by each of the proposers (typically, three proposers), though for proposed borings that are in close proximity of one another, borings may be combined. This number of supplemental borings (up to $3 \times 5 = 15$ borings) would typically apply to larger, more complex projects. A smaller number of borings could be used for smaller less complex projects. Ultimately, the number of supplemental borings is a project-specific decision that is made jointly between the Geotechnical Division and the project team.

22-A-1  Supplemental Geotechnical Data Report

Each Proposer is entitled to obtain certain additional geotechnical information by means of a Supplemental Geotechnical Data Report that WSDOT will conduct at WSDOT’s own expense. Under the Supplemental Geotechnical Data Report, Proposers may request WSDOT to perform up to five additional test borings and to provide an analysis of the resultant samples.

A request under the Supplemental Geotechnical Data Report must be submitted no later than the Request for Supplemental Boring Deadline set forth in this ITP. Each request shall set forth the location (by station and offset) and highest bottom elevation of the requested borings. Each request shall also include specific requests regarding the frequency and depth of field vane tests; the locations of split-spoon samples and Standard Penetration Tests; the length and diameter of rock cores; the depth of disturbed samples, undisturbed samples, and rock cores sought by the Proposer; and the tests the Proposer desires WSDOT to conduct in relation to the sample gathered.

WSDOT will make reasonable efforts to comply with Proposers’ requests under the Supplemental Geotechnical Data Report, but is not obligated to conduct borings at the precise locations requested. To the extent boring locations requested by one or more Proposers are within 20 feet of each other, the locations will be averaged and only one test boring will be conducted. If a Proposer’s boring is averaged with another Proposer’s boring, neither Proposer will be allowed an additional boring for this supplemental boring program. Survey personnel provided by WSDOT will establish the boring locations and elevations. A qualified inspector working for WSDOT will inspect the borings. WSDOT staff or an independent, qualified drilling contractor will perform the borings. At the option of the Proposers, each Proposer may dispatch a maximum of one person to observe the drilling, sampling, testing, and coring, and shall coordinate transportation of the chosen observer to the drilling site with WSDOT. The Proposers’ on-site observers shall not interfere with the operation of the surveyor, driller, or inspector.
The WSDOT drill crew or drilling contractor will conduct the following sampling and testing:

- Split-spoon samples and Standard Penetration Tests at 5-foot intervals and every change in stratum.
- Minimum NQ-size rock cores.
- Minimum 10-foot rock cores with RQD.
- Field vane shear tests in soft clays.
- Electronic cone penetrometer testing.
- Conventional laboratory classification testing on disturbed soil samples.
- Conventional laboratory tests on rock samples.
- Such other tests requested by a Proposer and agreed to by WSDOT at WSDOT’s sole discretion.

WSDOT will perform the test borings in whatever manner or sequence it deems appropriate at WSDOT’s sole discretion. The Supplemental Geotechnical Data Report, including the final boring logs and laboratory test results, will be provided to all Proposers according to Section 1 of this ITP and is included as Appendix G9 of the RFP. To the extent not consumed by testing, the samples resulting from the Supplemental Geotechnical Data Report will be turned over to the Design-Builder immediately after the Contract is awarded.

WSDOT makes no representation as to whether the Supplemental Geotechnical Data Report will be sufficient for the Proposer to prepare its Proposal. Each Proposer must make this determination independently based upon its own independent judgment and experience. Failure by a Proposer to submit a request for test borings under the Supplemental Geotechnical Data Report constitutes a conclusive presumption that the Proposer has determined that it does not require any additional geotechnical data to properly design, construct, and price the Work, or that it will obtain any necessary geotechnical data at its own expense using its own forces. If permits are required for supplemental borings (in addition to those permits already required for the Project), WSDOT may not be able to permit the borings within the deadline.