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Chapter 6  Seismic Design

6-1  Seismic Design Responsibility and Policy

6-1.1  Responsibility of the Geotechnical Designer

The geotechnical designer is responsible for providing geotechnical/seismic input parameters to the structural engineers for their use in structural design of the transportation infrastructure (e.g., bridges, retaining walls, ferry terminals, etc.). Specific elements to be addressed by the geotechnical designer include the design ground motion parameters, site response, geotechnical design parameters, and geologic hazards. The geotechnical designer is also responsible for providing input for evaluation of soil-structure interaction (foundation response to seismic loading), earthquake-induced earth pressures on retaining walls, and an assessment of the impacts of geologic hazards on the structures.

6-1.2  Geotechnical Seismic Design Policies

6-1.2.1  Seismic Performance Objectives

In general, the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications is followed for structure classification of bridges, except that the designation "other" is replaced with "normal" in the WSDOT Bridge Design Manual LRFD (BDM) M 23-50.

In keeping with the current seismic design approaches employed both nationally and internationally, geotechnical seismic design shall be consistent with the philosophy identified in the WSDOT BDM for structure seismic design which defines the structure performance objectives for the Safety Evaluation Earthquake (SEE) and the Functional Evaluation Earthquake (FEE). For the SEE, the performance objective requires that the structure be designed for non-collapse due to earthquake shaking and geologic hazards associated with a design seismic event so that loss of life and serious injury due to structure collapse are minimized. This is the primary performance objective for bridges classified as "normal". This performance objective shall be achieved at a seismic hazard level that is consistent with the seismic hazard level required in the AASHTO specifications (e.g., 7 percent probability of exceedance in 75 years for other structures, which is an approximate return period of 1,000 years). Geotechnical design associated with structures shall be consistent with this performance objective and design hazard level.

For the FEE, the performance objective requires minimal to no earthquake damage and that the structure remain in full service after the earthquake. For bridges classified as "essential" or "critical", a two level seismic design is required: the SEE as defined above, except that the damage due to the earthquake is limited to minimal to moderate and limited service for the structure is expected after the earthquake, and the Functional Evaluation Earthquake (FEE). This FEE performance objective shall be achieved at a hazard level of 30 percent probability of exceedance in 75 years (or 210-year return period). Geotechnical design associated with structures shall also be consistent with this performance objective and design hazard level for essential and critical bridges. See the BDM Chapter 4, for additional details regarding the performance objectives and
associated design requirements. See GDM Section 6-3.1 for requirements to assess the hazard level.

Bridge approach embankments and fills through which cut-and-cover tunnels are constructed should be designed to remain stable during the design seismic event because of the potential to contribute to collapse or inadequate performance of the structure should they fail or deform excessively. The aerial extent of approach embankment (and embankment surrounding cut-and-cover tunnels) seismic design and mitigation (if necessary) should be such that the structure is protected against instability or loading conditions that could result in collapse or inadequate performance. The typical distance of evaluation and mitigation is within 100 feet of the abutment or tunnel wall, but the actual distance should be evaluated on a case-by-case basis. Instability or other seismic hazards such as liquefaction, lateral spread, downdrag, and settlement may require mitigation near the abutment or tunnel wall to ensure that the structure is not compromised during a design seismic event. The geotechnical designer should evaluate the potential for differential settlement between mitigated and non mitigated soils. Additional measures may be required to limit differential settlements to tolerable levels both for static and seismic conditions. For "normal" bridges, the seismic stability of the bridge approach embankment in the lateral direction may not be required if instability in the lateral direction will not significantly damage the bridge and will not cause a life safety issue. The bridge interior pier foundations should also be designed to be adequately stable with regard to liquefaction, lateral spreading, flow failure, and other seismic effects to prevent bridge collapse for "normal" bridges when considering the FEE and which otherwise could compromise the functioning of essential and critical bridges for both the SEE and FEE hazard levels.

All retaining walls and abutment walls, including reinforced slopes steeper than 0.5H:1V, which shall be considered to be a wall (see Section 15-5.6), shall be evaluated and designed for seismic stability internally and externally (i.e. sliding, eccentricity, and bearing capacity), with the exception of walls that meet the AASHTO LRFD Bridge Design Manual “No Seismic Analysis" provisions in AASHTO Article 11.5.4.2. Noise walls, as well as reinforced slopes steeper than 1.2H:1V, shall also be evaluated for seismic stability.

With regard to seismic overall slope stability (often referred to as global stability) involving a retaining wall/reinforced slope as defined above, or noise wall, the geotechnical designer shall evaluate the impacts of failure due to seismic loading, as well as for liquefied conditions after shaking. If the wall seismic global stability does not meet the requirements in Sections 6-4.2 and 6-4.3, collapse of the wall/reinforced slope or noise wall shall be considered likely and assumed to cause loss of life or severe injury to the public if the following are true:

- The maximum wall/reinforced slope height is greater than 10 feet in height and
- The wall/reinforced slope is close enough to the traveled way such that collapse of the wall/reinforced slope or the slope that it supports will cause an abrupt elevation change within part or all of the traveled way, or will result in debris from the collapsed wall and the material that it supports being deposited on part or all of the traveled way, or other adjacent facility/structure.

If the above two bullets are true, the stability of the wall/reinforced slope or noise wall shall be improved such that the life safety of the public is preserved. If the maximum wall/reinforced slope or noise wall height is less than 10 ft, but the second bullet
is still true, the potential for wall/reinforced slope collapse shall be evaluated to assess the severity of the impact to the traveled way and to the potential for life safety issues to occur. Similarly, if the wall height is greater than 10 ft, but it is not near the traveled way as defined above, the potential for wall/reinforced slope or noise wall collapse shall be evaluated to assess the severity of the impact to the public and the potential for life safety issues to occur. In either of these cases, if it is determined that failure of the wall will compromise the life safety of the public, the stability of the wall/reinforced slope or noise wall shall be improved such that the life safety of the public is preserved.

Note that the policy to stabilize retaining walls/reinforced slopes and noise walls for overall stability due to design seismic events may not be practical for walls/reinforced slopes or noise walls placed on marginally stable landslide areas or otherwise marginally stable slopes. In general, if the placement of a wall/reinforced slope within a marginally stable slope (i.e., marginally stable for static conditions) has only a minor effect on the seismic stability of the landslide or slope, or if the wall/reinforced slope has a relatively low risk of causing loss of life or severe injury to the traveling public if collapse occurs, the requirement of the wall/reinforced slope and slope above and/or below the structure to meet minimum seismic overall stability requirements may be waived, subject to the approval of the State Geotechnical Engineer. The State Geotechnical Engineer will assess the impact and potential risks caused by wall and slope seismic instability or poor performance, and the magnitude of the effect the presence of the wall/reinforced slope could have on the stability of the overall slope during the design seismic event. The effect on the corridor in addition to the portion of the corridor being addressed by the project will be considered. In general, if the presence of the wall/reinforced slope could decrease the overall slope stability factor of safety by more than 0.05, the requirement to meet minimum seismic overall slope stability requirements will not be waived. However, this requirement may be waived by the State Geotechnical Engineer if the seismic slope stability safety factor for the existing slope (for the design earthquake ground motion) is significantly less than 0.9, subject to the evaluation of the impacts described above.

Cut slopes in soil and rock, fill slopes, and embankments should be evaluated for instability due to design seismic events and associated geologic hazards. Instability associated with cuts and fills is usually not mitigated due to the high cost of applying such a design policy uniformly to all slopes statewide. However, slopes that could cause collapse of an adjacent structure (e.g., a bridge, building, or pipeline) if failure due to seismic loading occurs, shall be stabilized.

6-1.2.2 **Liquefaction Mitigation for Bridge Widений**

Bridge widenings require special considerations, as the existing bridge to be widened may not be adequately stabilized to resist the forces imparted to the bridge due to liquefaction effects such as downdrag and lateral spreading loads/deformations. See BDM Section 4.3 for bridge widening seismic design and existing bridge seismic retrofit policies.

To assess the effect of liquefaction induced foundation loading and deformation on the existing and widened bridge stability, the geotechnical engineer provides the structural engineer with the following:

- depth and extent of soil that is likely to liquefy for the applicable hazard level (i.e., for the SEE for normal bridges, and the SEE and FEE hazard levels for essential and critical bridges,
• liquefaction induced downdrag loads and settlement,
• p-y curve parameters for the soil in both a liquefied and not liquefied state,
• the lateral spreading soil deformation profile (i.e., free field displacements), and
• the lateral loads acting on the foundation elements if flow failure is likely.

With this information, the structural designer can then determine the seismic stability of the existing bridge and bridge widening, and the need for structural strengthening of the existing bridge. If that is not feasible, the geotechnical engineer assesses the need for ground improvement to prevent the liquefaction from occurring. If ground improvement is needed, the geotechnical engineer also provides a ground improvement design.

Note that the foundation loads caused by flow failure are affected by the foundation details and therefore may require some design iteration between the geotechnical and structural designer.

Details on the liquefaction analysis, mitigation needed if the bridge cannot be designed to resist the forces and soil deformation anticipated, and the input the geotechnical designer provides to the structural designer regarding liquefaction and its effect, are provided in Sections 6-4.2 and 6-5 of this GDM.

6-1.2.3 Maximum Considered Depth for Liquefaction

When evaluating liquefaction potential and its impacts to transportation facilities, the maximum considered liquefaction depth below the natural ground surface shall be limited to 80 feet. However, for sites that contain exceptionally loose soils that are apparently highly susceptible to liquefaction to greater depths, effective stress analysis techniques may be used to evaluate the potential for deeper liquefaction and the potential impacts of that liquefaction. The reasons for this depth limitation are as follows:

Limits of Simplified Procedures – The simplified procedures most commonly used to assess liquefaction potential are based on historical databases of liquefied sites with shallow liquefaction (i.e., in general, less than 50 feet). Thus, these empirical methodologies have not been calibrated to evaluate deep liquefaction. In addition, the simplified equation used to estimate the earthquake induced cyclic shear stress ratio (CSR) is based on a stress reduction coefficient, \( r_d \), which is highly variable at depth. For example, at shallow depth (15 feet), \( r_d \) ranges from about 0.94 to 0.98. As depth increases, \( r_d \) becomes more variable ranging, for example, from 0.40 to 0.80 at a depth of 65 feet. The uncertainty regarding the coefficient \( r_d \) and lack of verification of the simplified procedures used to predict liquefaction at depth, as well as some of the simplifying assumptions and empiricism within the simplified method with regard to the calculation of liquefaction resistance (i.e., the cyclic resistance ratio CRR), limit the depth at which these simplified procedures should be used. Therefore, simplified empirical methods to predict liquefaction at depths greater than 50 to 60 feet should be based on a site response analysis to obtain an appropriate, site-specific stress reduction profile, provided that sufficient subsurface data are available and that variability in the input ground motions is considered.
Lack of Verification and Complexity of More Rigorous Approaches – Several non-linear, effective stress analysis programs have been developed by researchers and can be used to estimate liquefaction potential at depth. However, there has been little field verification of the ability of these programs to predict liquefaction at depth because there are few well documented sites with deep liquefaction. Key is the ability of these approaches to predict pore pressure increase and redistribution in liquefiable soils during and after ground shaking. Calibration of such pore pressure models has so far been limited to comparison to laboratory performance data test results and centrifuge modeling. Furthermore, these more rigorous methods require considerable experience to obtain and apply the input data required, and to confidently interpret the results. Hence, use of such methods requires independent peer review (see Section 6-3 regarding peer review requirements) by expert(s) in the use of such methods for liquefaction analysis.

Decreasing Impact with Depth – Observation and analysis of damage in past earthquakes suggests that the damaging effects of liquefaction generally decrease as the depth of a liquefiable layer increases. This reduction in damage is largely attributed to decreased levels of relative displacement and the need for potential failure surfaces to extend down to the liquefying layer. For example, the effect of a 10 feet thick soil layer liquefying between depths of 80 and 90 feet will generally be much less severe than the effect of a layer between the depths of 10 and 20 feet. Note that these impacts are focused on the most damaging effects of liquefaction, such as lateral deformation and instability. Deeper liquefaction can, however, increase the magnitude and impact of vertical movement (settlement) and loading (downdrag) on foundations.

Difficulties Mitigating for Deep Liquefaction – The geotechnical engineering profession has limited experience with mitigation of liquefaction hazards at large depths, and virtually no field case histories on which to reliably verify the effectiveness of mitigation techniques for very deep liquefaction mitigation. In practicality, the costs to reliably mitigate liquefaction by either ground improvement or designing the structure to tolerate the impacts of very deep liquefaction are excessive and not cost effective for most structures.

6-1.3 Governing Design Specifications and Additional Resources

The specifications applicable to seismic design of a given project depend upon the type of facility.

For transportation facilities the following manuals, listed in hierarchical order, shall be the primary source of geotechnical seismic design policy for WSDOT:

1. This Geotechnical Design Manual (GDM)
2. AASHTO Guide Specifications for LRFD Seismic Bridge Design
3. AASHTO LRFD Bridge Design Specifications

If a publication date is shown, that version shall be used to supplement the geotechnical design policies provided in this WSDOT GDM. If no date is shown, the most current version, including interim publications of the referenced manuals, as of the WSDOT GDM publication date shall be used. This is not a comprehensive list; other publications are referenced in this WSDOT GDM and shall be used where so directed herein.
Until the AASHTO Guide Specifications for LRFD Bridge Seismic Design are fully adopted in the AASHTO LRFD Bridge Design Specifications, the seismic design provisions in the Guide Specifications regarding foundation design, liquefaction assessment, earthquake hazard assessment, and ground response analysis shall be considered to supersede the parallel seismic provisions in the AASHTO LRFD Bridge Design Specifications.

With regard to seismic hazard levels, the AASHTO Guide Specifications for LRFD Seismic Bridge Design and the AASHTO LRFD Bridge Design Specifications are based on the 2002 USGS website hazard model at a return period of 975 years (i.e., a probability of exceedance of approximately 7 percent in 75 years). The GDM and BDM seismic design requirements have been updated to use the 2014 USGS website hazard model at a probability of exceedance of 7 percent in 75 years and shall be considered to supersede the AASHTO specifications. Note that the USGS website refers to this hazard level as 5% in 50 years.

For seismic design of new buildings and non-roadway infrastructure, the International Building Code (IBC) (International Code Council), most current version should be used.

FHWA geotechnical design manuals, or other nationally recognized design manuals, are considered secondary relative to this WSDOT GDM and the AASHTO manuals (and for buildings, the IBC) listed above regarding WSDOT geotechnical seismic design policy, and may be used to supplement the WSDOT GDM, WSDOT BDM, and AASHTO design specifications.

A brief description of these additional references is as follows:

**FHWA Geotechnical Engineering Circular No. 3 (Kavazanjian, et al., 2011)** – This FHWA document provides design guidance for geotechnical earthquake engineering for highways. Specifically, this document provides guidance on earthquake fundamentals, seismic hazard analysis, ground motion characterization, site characterization, seismic site response analysis, seismic slope stability, liquefaction, and seismic design of foundations and retaining walls. The document also includes design examples for typical geotechnical earthquake engineering analyses.

**FHWA LRFD Seismic Analysis and Design of Bridges Reference Manual (Marsh et al., 2014)** – This manual adapts and updates FHWA Geotechnical Engineering Circular No. 3 to be applicable to LRFD for Bridges and their foundations. This manual includes both geotechnical and structural design.

**Geotechnical Earthquake Engineering Textbook** – The textbook titled Geotechnical Earthquake Engineering (Kramer, 1996) provides a wealth of information to geotechnical engineers for seismic design. The textbook includes a comprehensive summary of seismic hazards, seismology and earthquakes, strong ground motion, seismic hazard analysis, wave propagation, dynamic soil properties, ground response analysis, design ground motions, liquefaction, seismic slope stability, seismic design of retaining walls, and ground improvement.
In addition, the following website may be accessed to obtain detailed ground motion data that will be needed for design:

**United States Geological Survey (USGS) Website** - The USGS National Hazard Mapping Project website [https://earthquake.usgs.gov/hazards/hazmaps](https://earthquake.usgs.gov/hazards/hazmaps) is a valuable source for information regarding the mapping seismic hazard in the United States, and specifically on the details of the hazard model underlying the 2014 mapping. The website also includes a Unified Hazard Tool which allows the user to extract hazard curves and deaggregations for various return periods of interest for the 2008 and 2014 seismic hazard maps. This tool can be found at the following address: [https://earthquake.usgs.gov/hazards/interactive](https://earthquake.usgs.gov/hazards/interactive)

The results of the hazards analysis using the 2002 USGS website hazard model at a probability of exceedance of 5 percent in 50 years are the same as those from the AASHTO hazard analysis maps. However, the USGS has updated their hazards maps, and the new 2014 hazard maps and deaggregation data shall be used for seismic design (see USGS website for update and figures later in this GDM chapter).

Geotechnical seismic design is a rapidly developing sub-discipline within the broader context of the geotechnical engineering discipline, and new resources such as technical journal articles, as well as academic and government agency research reports, are becoming available to the geotechnical engineer. It is important when using these other resources, as well as those noted above, that a review be performed to confirm that the guidance represents the current state of knowledge and that the methods have received adequate independent review. Where new methods not given in the AASHTO Specifications or herein (i.e., Chapter 6) are proposed in the subject literature, use of the new method(s) shall be approved by the State Geotechnical Engineer for use in the project under consideration.

### 6-2 Geotechnical Seismic Design Considerations

#### 6-2.1 Overview

The geotechnical designer has four broad options available for seismic design. They are:

- Use specification/code based hazard (**Section 6-3.1**) with specification/code based ground motion response (**Section 6-3.2.1**), also referred to as the General Procedure
- Use specification/code based hazard (**Section 6-3.1**) with site specific ground motion response (**Section 6-3.2.2** and **Appendix 6-A**)
- Use site specific hazard (**Section 6-3.1** and **Appendix 6-A**) with specification/code based ground motion response (**Section 6-3.2.1**)
- Use site specific hazard (**Section 6-3.1** and **Appendix 6-A**) with site specific ground motion response (**Section 6-3.2.2** and **Appendix 6-A**)

Geotechnical parameters required for seismic design depend upon the type and importance of the structure, the geologic conditions at the site, and the type of analysis to be completed. For most structures, specification based design criteria appropriate for the site's soil conditions may be all that is required. Unusual, critical, or essential structures may require more detailed structural analysis, requiring additional geotechnical parameters. Finally, site conditions may require detailed geotechnical evaluation to quantify geologic hazards.
6-2.2 Site Characterization and Development of Seismic Design Parameters

As with any geotechnical investigation, the goal is to characterize the site soil conditions and determine how those conditions will affect the structures or features constructed when seismic events occur. In order to make this assessment, the geotechnical designer should review and discuss the project with the structural engineer, as seismic design is a cooperative effort between the geotechnical and structural engineering disciplines. The geotechnical designer should do the following as a minimum:

- Identify, in coordination with the structural designer, structural characteristics (e.g., fundamental frequency/period), anticipated method(s) of structural analysis, performance criteria (e.g., collapse prevention, allowable horizontal displacements, limiting settlements, target load and resistance factors, components requiring seismic design, etc.) and design hazard levels (e.g., 7 percent PE in 75 years or 30 percent in 75 years).
- Identify, in coordination with the structural engineer, what type of ground motion parameters are required for design (e.g., response spectra or time histories), and their point of application (e.g., mudline, bottom of pile cap, or depth of pile fixity).
- Identify, in coordination with the structural engineer, how foundation stiffness will be modeled and provide appropriate soil stiffness properties or soil/foundation springs.
- Identify potential geologic hazards, areas of concern (e.g. soft soils), and potential variability of local geology.
- Identify potential for large scale site effects (e.g., basin, topographic, and near fault effects).
- Identify, in coordination with the structural designer, the method by which risk-compatible ground motion parameters will be established (specification/code, deterministic, probabilistic, or a hybrid).
- Identify engineering analyses to be performed (e.g. site specific seismic response analysis, liquefaction susceptibility, lateral spreading/slope stability assessments).
- Identify engineering properties required for these analyses.
- Determine methods to obtain parameters and assess the validity of such methods for the material type.
- Determine the number of tests/samples needed and appropriate locations to obtain them.

It is assumed that the basic geotechnical investigations required for nonseismic (gravity load) design have been or will be conducted as described in Chapters 2, 5 and the individual project element chapters (e.g., Chapter 8 for foundations, Chapter 15 for retaining walls, etc.). Typically, the subsurface data required for seismic design is obtained concurrently with the data required for design of the project (i.e., additional exploration for seismic design over and above what is required for nonseismic foundation design is typically not necessary). However, the exploration program may need to be adjusted to obtain the necessary parameters for seismic design. For instance, a seismic cone might be used in conjunction with a CPT if shear wave velocity data is required. Likewise, if liquefaction potential is a significant issue, mud rotary drilling with SPT sampling should be used. In this case, preference shall be given to drill rigs furnished with automatic SPT hammers that have been recently (i.e., within the past 6 months) calibrated for hammer energy. Hollow-stem auger drilling and non-standard samplers (e.g., down-the-hole or
wire-line hammers) shall not be used to collect data used in liquefaction analysis and
mitigation design, other than to obtain samples for gradation.

The goal of the site characterization for seismic design is to develop the subsurface profile
and soil property information needed for seismic analyses. Soil parameters generally
required for seismic design include:

- Dynamic shear modulus at small strains or shear wave velocity;
- Shear modulus and material damping characteristics as a function of shear strain;
- Cyclic and post-cyclic shear strength parameters (peak and residual);
- Consolidation parameters such as the Compression Index or Percent Volumetric Strain
  resulting from pore pressure dissipation after cyclic loading, and
- Liquefaction resistance parameters.

Table 6-1 provides a summary of site characterization needs and testing considerations
for geotechnical/seismic design.

Chapter 5 covers the requirements for using the results from the field investigation,
the field testing, and the laboratory testing program separately or in combination to
establish properties for static design. Many of these requirements are also applicable for
seismic design.

For routine designs, in-situ field measurements or laboratory testing for parameters
such as the dynamic shear modulus at small strains, shear modulus and damping ratio
characteristics versus shear strain, and residual shear strength are generally not obtained.
Instead, correlations based on index properties may be used in lieu of in-situ or laboratory
measurements for routine design to estimate these values. However, if a site specific
ground motion response analysis is conducted, field measurements of the shear wave
velocity $V_s$ should be obtained.
<table>
<thead>
<tr>
<th>Geotechnical Issues</th>
<th>Engineering Evaluations</th>
<th>Required Information for Analyses</th>
<th>Field Testing</th>
<th>Laboratory Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Site Response</strong></td>
<td>• source characterization and ground motion attenuation</td>
<td>• subsurface profile (soil, groundwater, depth to rock)</td>
<td>• SPT</td>
<td>• Atterberg limits</td>
</tr>
<tr>
<td></td>
<td>• site response spectra</td>
<td>• shear wave velocity</td>
<td>• CPT</td>
<td>• grain size distribution</td>
</tr>
<tr>
<td></td>
<td>• time history</td>
<td>• shear modulus for low strains</td>
<td></td>
<td>• specific gravity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• relationship of shear modulus with increasing shear strain, OCR, and PI</td>
<td>• seismic cone</td>
<td>• moisture content</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• equivalent viscous damping ratio with increasing shear strain, OCR, and PI</td>
<td>• geophysical testing (shear wave velocity)</td>
<td>• unit weight</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Poisson's ratio</td>
<td>• piezometer</td>
<td>• resonant column</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• unit weight</td>
<td></td>
<td>• cyclic direct simple shear test</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• relative density</td>
<td></td>
<td>• torsional simple shear test</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• seismicity (design earthquakes - source, distance, magnitude, recurrence)</td>
<td></td>
<td>• cyclic triaxial tests</td>
</tr>
<tr>
<td><strong>Geologic Hazards Evaluation</strong> (e.g., liquefaction, lateral spreading, slope stability, faulting)</td>
<td>• liquefaction susceptibility</td>
<td>• subsurface profile (soil, groundwater, rock)</td>
<td>• SPT</td>
<td>• grain size distribution</td>
</tr>
<tr>
<td></td>
<td>• liquefaction triggering</td>
<td>• shear strength (peak and residual)</td>
<td>• CPT</td>
<td>• Atterberg Limits</td>
</tr>
<tr>
<td></td>
<td>• liquefaction induced settlement</td>
<td>• unit weights</td>
<td></td>
<td>• specific gravity</td>
</tr>
<tr>
<td></td>
<td>• settlement of dry sands</td>
<td>• grain size distribution</td>
<td>• Becker penetration test</td>
<td>• organic content</td>
</tr>
<tr>
<td></td>
<td>• lateral spreading and flow failure</td>
<td>• plasticity characteristics</td>
<td>• vane shear test</td>
<td>• moisture content</td>
</tr>
<tr>
<td></td>
<td>• slope stability and deformations</td>
<td>• relative density</td>
<td>• piezometers</td>
<td>• unit weight</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• penetration resistance</td>
<td>• geophysical testing (shear wave velocity)</td>
<td>• soil shear strength tests (static and cyclic)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• shear wave velocity</td>
<td></td>
<td>• post-cyclic volumetric strain</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• seismicity (PGA, design earthquakes, deaggregation data, ground motion time histories)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• site topography</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Input for Structural Design</strong></td>
<td>• soil stiffness for shallow</td>
<td>• subsurface profile (soil, groundwater, rock)</td>
<td>• CPT</td>
<td>• grain size distribution</td>
</tr>
<tr>
<td></td>
<td>• foundations (e.g., springs)</td>
<td>• shear strength (peak and residual)</td>
<td>• SPT</td>
<td>• Atterberg limits</td>
</tr>
<tr>
<td></td>
<td>• P-Y data for deep foundations</td>
<td>• coefficient of horizontal subgrade reaction</td>
<td>• seismic cone</td>
<td>• specific gravity</td>
</tr>
<tr>
<td></td>
<td>• down-drag on deep foundations</td>
<td>• seismic horizontal earth pressure coefficients</td>
<td>• piezometers</td>
<td>• moisture content</td>
</tr>
<tr>
<td></td>
<td>• residual strength</td>
<td>• shear modulus for low strains or shear wave velocity</td>
<td>• geophysical testing (shear wave velocity, resistivity, natural gamma)</td>
<td>• unit weight</td>
</tr>
<tr>
<td></td>
<td>• lateral earth pressures</td>
<td>• relationship of shear modulus with increasing shear strain</td>
<td>• vane shear test</td>
<td>• resonant column</td>
</tr>
<tr>
<td></td>
<td>• lateral spreading/slope movement loading</td>
<td>• unit weight</td>
<td>• pressuremeter</td>
<td>• cyclic direct simple shear test</td>
</tr>
<tr>
<td></td>
<td>• post earthquake settlement</td>
<td>• Poisson's ratio</td>
<td></td>
<td>• triaxial tests (static and cyclic)</td>
</tr>
<tr>
<td></td>
<td>• Kinematic soil-structure interaction</td>
<td>• seismicity (PGA, design earthquake, response spectrum, ground motion time histories)</td>
<td></td>
<td>• torsional shear test</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• site topography</td>
<td></td>
<td>• direct shear interface tests</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Interface shear strength</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
If correlations are used to obtain seismic soil design properties, and site- or region-specific relationships are not available, then the following correlations should be used:

- **Table 6-2**, which presents correlations for estimating initial shear modulus based on relative density, penetration resistance or void ratio.

- Shear modulus reduction and equivalent viscous damping ratio equations by Darendeli (2001) as provided in equations 6-1 through 6-7, applicable to all soils except peats and gravels.

- For gravels, shear modulus reduction and viscous damping relationships provided in Rollins, et al. (1998).

- For peats, shear modulus reduction and viscous damping relationships provided in Kramer (1996, 2000).

- **Figures 6-1 through 6-3**, which present charts for estimating equivalent undrained residual shear strength for liquefied soils as a function of SPT blowcounts. These figures primarily apply to sands and silty sands. It is recommended that all these figures be checked to estimate residual strength and averaged using a weighting scheme. **Table 6-3** presents an example of a weighting scheme as recommended by Kramer (2007). Designers using these correlations should familiarize themselves with how the correlations were developed, assumptions used, and any limitations of the correlations as discussed in the source documents for the correlations before selecting a final weighting scheme to use for a given project. Alternate correlations based on CPT data may also be considered. For silts, laboratory testing using cyclic simple shear or cyclic triaxial testing should be conducted (see GDM Section 6-4.2.6).

Designers are encouraged to develop region or project specific correlations for these seismic design properties. Other well accepted correlations in peer reviewed publications may be used, subject to the approval of the State Geotechnical Engineer.

Regarding Figure 6-3, two curves are provided, one in which void redistribution is likely, and one in which void redistribution is not likely. Void redistribution becomes more likely if a relatively thick liquefiable layer is capped by relatively impermeable layer. Sufficient thickness of a saturated liquefiable layer is necessary to generate enough water for void redistribution to occur, and need capping by a relatively impermeable layer to prevent pore pressures from dissipating, allowing localized loosening near the top of the confined liquefiable layer. Engineering judgment will need to be applied to determine which curve in Figure 6-3 to use.

When using the above correlations, the potential effects of variations between the dynamic property from the correlation and the dynamic property for the particular soil should be considered in the analysis. The published correlations were developed by evaluating the response of a range of soil types; however, for any specific soil, the behavior of any specific soil can depart from the average, falling either above or below the average. These differences can affect the predicted response of the soil. For this reason sensitivity studies should be conducted to evaluate the potential effects of property variation on the design prediction.

For those cases where a single value of the property can be used with the knowledge that the design is not very sensitive to variations in the property being considered, a sensitivity analysis may not be required.
Table 6-2  Correlations for Estimating Initial Shear Modulus (Adapted from Kavazanjian, et al., 2011)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Correlation</th>
<th>Units (1)</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seed et al. (1984)</td>
<td>( G_{\text{max}} = 220 \left( K_2 \right)<em>{\text{max}} \left( \sigma'</em>{\text{m}} \right)^{1/3} ) ( (K_2)<em>{\text{max}} = 20(N_1)</em>{60}^{1/3} )</td>
<td>kPa</td>
<td>( (K_2)_{\text{max}} ) is about 30 for very loose sands and 75 for very dense sands; about 80 to 180 for dense well graded gravels; Limited to cohesionless soils</td>
</tr>
<tr>
<td>Imai and Tonouchi (1982)</td>
<td>( G_{\text{max}} = 15,560 N_{60}^{0.68} )</td>
<td>kPa</td>
<td>Limited to cohesionless soils</td>
</tr>
<tr>
<td>Hardin (1978)</td>
<td>( G_{\text{max}} = (6.25/0.3+e_o^{1.3})(P_a \sigma'_{\text{m}})^{0.5}OCR^k )</td>
<td>kPa ( (1)(3) )</td>
<td>Limited to cohesive soils ( P_a = ) atmospheric pressure</td>
</tr>
<tr>
<td>Jamiolkowski, et al. (1991)</td>
<td>( G_{\text{max}} = 6.25/(e_o^{1.3})(P_a \sigma'_{\text{m}})^{0.5}OCR^k )</td>
<td>kPa ( (1)(3) )</td>
<td>Limited to cohesive soils ( P_a = ) atmospheric pressure</td>
</tr>
<tr>
<td>Mayne and Rix (1993)</td>
<td>( G_{\text{max}} = 99.5(P_a)<em>{0.305}(qc)</em>{0.695}/(e_o)_{1.13} )</td>
<td>kPa ( (2) )</td>
<td>Limited to cohesive soils ( P_a = ) atmospheric pressure</td>
</tr>
</tbody>
</table>

Notes:
(1) 1 kPa = 20.885 psf
(2) P_a and qc in kPa
(3) The parameter k is related to the plasticity index, PI, as follows:

<table>
<thead>
<tr>
<th>PI</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>20</td>
<td>0.18</td>
</tr>
<tr>
<td>40</td>
<td>0.30</td>
</tr>
<tr>
<td>60</td>
<td>0.41</td>
</tr>
<tr>
<td>80</td>
<td>0.48</td>
</tr>
<tr>
<td>&gt;100</td>
<td>0.50</td>
</tr>
</tbody>
</table>

**Modulus Reduction Curve (Darendeli, 2001)** – The modulus reduction curve for soil, as a function of shear strain, should be calculated as shown in Equations 6-1 and 6-2.

\[
\frac{G}{G_{\text{max}}} = \frac{1}{1 + \left( \frac{\gamma}{\gamma_r} \right)^a}
\]  

(6-1)

where,
- \( G \) = shear modulus at shear strain \( \gamma \), in the same units as \( G_{\text{max}} \)
- \( \gamma \) = shear strain (%), and
- \( a \) = 0.92

\( \gamma_r \) is defined in Equation 6-2 as:

\[
\gamma_r = \left( \phi_1 + \phi_2 \times PI \times OCR^\phi \right) \times \sigma_0^{\phi_1}
\]  

(6-2)

where,
- \( \phi_1 = 0.0352; \phi_2 = 0.0010; \phi = 0.3246; \phi_1 = 0.3483 \) (from regression),
- OCR = overconsolidation ratio for soil
- \( \sigma_0' \) = effective vertical stress, in atmospheres, and
- PI = plastic index, in %
Seismic Design Chapter 6

**Damping Curve (Darendeli, 2001)** – The damping ratio for soil, as a function of shear strain, should be calculated as shown in Equations 6-3 through 6-7.

Initial step: Compute closed-form expression for Masing Damping for \( a = 1.0 \) (standard hyperbolic backbone curve):

\[
D_{\text{Masing, } a=1} (\gamma) [%] = \frac{100}{\pi} \left[ \frac{\gamma - \gamma_t \ln \left( \frac{\gamma + \gamma_t}{\gamma_t} \right)}{\gamma^2} - 2 \right]
\]  \hspace{1cm} (6-3)

For other values of \( a \) (e.g., \( a = 0.92 \), as used to calculate \( G \)):

\[
D_{\text{Masing, } a=1} (\gamma) [%] = c_1(D_{\text{masing, } a=1}) + c_2(D_{\text{masing, } a=1})^2 + c_3(D_{\text{masing, } a=1})^3
\]  \hspace{1cm} (6-4)

Where,

\[
\begin{align*}
c_1 &= 0.2523 + 1.8618a - 1.1143a^2 \\
c_2 &= -0.0095 - 0.0710a + 0.0805a^2 \\
c_3 &= 0.0003 + 0.0002a - 0.0005a^2
\end{align*}
\]

Final step: Compute damping ratio as function of shear strain:

\[
D(\gamma) = D_{\text{min}} + bD_{\text{Masing}} (\gamma) \left( \frac{G}{G_{\text{max}}} \right)^{0.1}
\]  \hspace{1cm} (6-5)

Where,

\[
D_{\text{min}} = \left( \phi_b + \phi_t \times PI \times OCR^h \right) \times \sigma_0^{\phi_b} \times \left( 1 + \phi_{i0} \ln(f\text{req}) \right)
\]

\[
b = \phi_{i11} + \phi_{i12} \times \ln(N)
\]  \hspace{1cm} (6-6)

Where:

\[
\begin{align*}
f\text{req} &= \text{frequency of loading, in Hz} \\
N &= \text{number of loading cycles} \\
\phi_b &= 0.8005; \\
\phi_t &= 0.0129; \\
\phi_0 &= -0.1069; \\
\phi_y &= -0.2889; \\
\phi_{i0} &= 0.2919; \\
\phi_{i11} &= 0.6329; \\
\phi_{i12} &= -0.0057
\end{align*}
\]

Table 6-3 Weighting Factors for Residual Strength Estimation (Kramer, 2007)

<table>
<thead>
<tr>
<th>Model</th>
<th>Weighting Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Idriss</td>
<td>0.2</td>
</tr>
<tr>
<td>Olson-Stark</td>
<td>0.2</td>
</tr>
<tr>
<td>Idriss-Boulanger</td>
<td>0.2</td>
</tr>
<tr>
<td>Hybrid</td>
<td>0.4</td>
</tr>
</tbody>
</table>
Figure 6-1  Estimation of Residual Strength Ratio from SPT Resistance (Olson and Stark, 2002)

Figure 6-2  Estimation of Residual Strength Ratio from SPT Resistance (Idriss and Boulanger, 2007)
Figure 6-3  Estimation of Residual Strength Ratio from SPT Resistance (Idriss and Boulanger, 2007)

Figure 6-4  Variation of Residual Strength Ratio with SPT Resistance and Initial Vertical Effective Stress Using Kramer-Wang Model (Kramer, 2007)
6-2.3 Information for Structural Design

The geotechnical designer shall recommend a design earthquake ground motion based on the SEE for normal bridges and both the SEE and FEE for essential and critical bridges, and shall evaluate geologic hazards for the project. For code based ground motion analysis, the geotechnical designer shall provide the Site Class B/C boundary spectral accelerations at periods of 0.2 and 1.0 seconds, the PGA, the site class, and site coefficients for the PGA and spectral accelerations to account for the effect of the site class on the design accelerations.

In addition, the geotechnical designer should evaluate the site and soil conditions to the extent necessary to provide the following input for structural design, with consideration to the structure classification (i.e., normal, essential, or critical bridges) and the hazard level required (i.e., SEE for normal bridges, and both SEE and FEE for essential and critical bridges):

- Foundation spring values for dynamic loading (lateral and vertical), as well as geotechnical parameters for evaluation of sliding resistance applicable to the foundation design. If liquefaction is possible, spring values for liquefied conditions should also be provided (primarily applies to deep foundations, as in general, shallow footings are not used over liquefied soils).
- Earthquake induced earth pressures (active and passive) for retaining structures and below grade walls, and other geotechnical parameters, such as sliding resistance, needed to complete the seismic design of the wall.
- If requested by the structural designer, passive soil springs to use to model the abutment fill resistance to seismic motion of the bridge.
- Impacts of seismic geologic hazards including fault rupture, liquefaction, lateral spreading, flow failure, and slope instability on the structure, including estimated loads and deformations acting on the structure due to the effects of the geologic hazard.
- If requested by the structural designer, for long bridges, potential for incoherent ground motion effects.
- Options to mitigate seismic geologic hazards, such as ground improvement. Note that seismic soil properties used for design should reflect the presence of the soil improvement.
6-3 Seismic Hazard and Site Ground Motion Response Requirements

For most projects, design code/specification based seismic hazard and ground motion response (referred to as the "General Procedure" in the AASHTO Guide Specifications for LRFD Seismic Bridge Design) are appropriate and shall be used, except that the 2014 seismic hazard data and maps described previously shall be used instead of the 2002 hazard information provided in the AASHTO Specifications. However, a site specific hazard or ground motion response analysis is required in situations for which the General Procedure is not applicable, and may also be considered for situations in which the General Procedure is applicable.

6-3.1 Determination of Seismic Hazard Level

All transportation structures (e.g., bridges, pedestrian bridges, walls, etc.) classified as "other" or "normal" (i.e., not critical or essential) are designed for the SEE (see Section 6-1.2.1) based on a hazard level of 7 percent PE in 75 years (i.e., an approximately 1,000 year return period). For essential or critical bridges, a two level seismic hazard design is required: the Safety Evaluation Earthquake (SEE) and the Functional Evaluation Earthquake (FEE). In this case, the SEE hazard level is as defined above. The FEE is based on a hazard level of 30 percent probability of exceedance in 75 years (or 210-year return period).

For buildings on terminal structures, the design hazard level shall be consistent with IBC requirements, which uses a risk adjusted 2,475 year event as its basis (MCER).

The AASHTO Guide Specifications for LRFD Seismic Bridge Design shall be used for WSDOT transportation facilities for code/specification based seismic hazard evaluation, except that Figures 6-5, 6-6, and 6-7 shall be used to estimate the PGA, 0.2 sec. spectral acceleration ($S_2$), and 1.0 sec. spectral acceleration values ($S_1$), respectively, for the SEE. By definition for Figures 6-5, 6-6, and 6-7, PGA, $S_2$ and $S_1$ are for the Site Class B/C boundary (very hard or very dense soil or soft rock) conditions. The PGA contours in Figure 6-5, in addition $S_2$ and $S_1$ in Figures 6-6 and 6-7, are based on information published by the USGS National Seismic Hazards Mapping Project (USGS, 2014) and supersede the AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for LRFD Seismic Bridge Design. Interpolation between contours in Figures 6-5, 6-6, and 6-7 should be used when establishing the PGA for the Site Class B/C boundary for a project. High resolution images of these three acceleration maps are provided in Appendix 6-B.
Figure 6-5  Peak Horizontal Acceleration (%G) for 7% Probability of Exceedance in 75 Years for Site Class B/C Boundary (Adapted From USGS 2014)
Figure 6-6  Horizontal Spectral Acceleration at 0.2 Second Period (%g) for 7% Probability of Exceedance in 75 Years with 5% of Critical Damping for Site Class B/C Boundary (Adapted from USGS 2014)
Figure 6-7  Horizontal Spectral Acceleration at 1.0 Second Period (%g) for 7% Probability of Exceedance in 75 Years With 5% of Critical Damping for Site Class B/C Boundary (Adapted from USGS 2014)

To obtain the PGA, 0.2 sec. spectral acceleration ($S_2$), and 1.0 sec. spectral acceleration values ($S_1$) for the FEE i.e., 30 percent probability of exceedance in 75 years (or 210-year return period), go to the USGS website at: [https://earthquake.usgs.gov/hazards/interactive](https://earthquake.usgs.gov/hazards/interactive)

When a transportation structure (e.g., bridges, walls, and WSF terminal structures such as docks, etc.) is designated as critical or essential by WSDOT, a more stringent seismic hazard level may be required by the State Bridge Engineer. If a different hazard level than that specified herein and in the AASHTO LRFD Seismic design specifications is selected, the most current seismic hazard maps from the USGS National Seismic Hazards Mapping Project should be used, unless a site specific seismic hazard analysis is conducted, subject to the approval of the State Bridge Engineer and State Geotechnical Engineer.
A site specific hazard analysis should be considered in the following situations:

- A more accurate assessment of hazard level is desired, or
- Information about one or more active seismic sources for the site has become available since the USGS Seismic Hazard Maps specified herein (USGS 2014) were developed, and the new seismic source information may result in a significant change of the seismic hazard at the site.

If the site is located within 6 miles of a known active fault capable of producing a magnitude 5 or greater earthquake and near fault effects are not adequately modeled in the development of ground motion maps used, directivity and directionality effects shall be addressed as described in Article 3.4.3.1 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design and its commentary.

If a site specific hazard analysis is conducted, it shall be conducted in accordance with AASHTO Guide Specifications for LRFD Seismic Bridge Design and GDM Appendix 6-A.

If a site specific probabilistic seismic hazard analysis (PSHA) is conducted, it shall be conducted in a manner to generate a uniform-hazard acceleration response spectrum considering a 7 percent probability of exceedance in 75 years for spectral values over the entire period range of interest. This analysis shall follow the same basic approach as used by the USGS in developing seismic hazards maps for AASHTO and for the 2014 maps included in this GDM chapter. In this approach it is necessary to establish the following:

- The contributing seismic sources,
- A magnitude fault-rupture-length or source area relation for each contributing fault or source area to estimate an upper-bound earthquake magnitude for each source zone,
- Median ground motion attenuation equations for acceleration response spectral values and their associated standard deviations,
- A magnitude-recurrence relation for each source zone, and
- Weighting factors, with justification, for all branches of logic trees used to establish ground shaking hazards.

AASHTO allows site-specific ground motion hazard levels to be based on a deterministic seismic hazard analysis (DSHA) in regions of known active faults, provided that deterministic spectrum is no less than two-thirds of the probabilistic spectrum (see AASHTO Article 3.10.2.2). This requires that:

- The ground motion hazard at a particular site is largely from known faults (e.g., "random" seismicity is not a significant contributor to the hazard), and
- The recurrence interval for large earthquakes on the known faults are generally less than the return period corresponding to the specified seismic hazard level (e.g., the earthquake recurrence interval is less than a return period of 1,000 years that corresponds to a seismic hazard level of 7 percent probability of exceedance in 75 years).

Currently, these conditions are generally not met for sites in Washington State. Approval by the State Geotechnical Engineer and State Bridge Engineer is required before DSHA-based ground motion hazard level is used on a WSDOT project.
Where use of a deterministic spectrum is appropriate, the spectrum shall be either:

- The envelope of a median spectra calculated for characteristic maximum magnitude earthquakes on known active faults; or
- The deterministic spectra for each fault, and in the absence of a clearly controlling spectrum, each spectrum should be used.

Uncertainties in source modeling and parameter values shall be taken into consideration in the PSHA and DSHA. Detailed documentation of seismic hazard analysis shall be provided.

For buildings, restrooms, and shelters, specification based seismic design parameters required by the most current version of the International Building Code (IBC) shall be used. For covered pedestrian walkways, the AASHTO LRFD Bridge Design Specifications or AASHTO Guide Specifications for LRFD Seismic Bridge Design shall be used.

The seismic design requirements of the IBC are based on a hazard level of 2 percent PE in 50 years which has been risk adjusted. The 2 percent PE in 50 years hazard level corresponds to the maximum considered earthquake (MCE), and the risk adjusted earthquake (MCER) corresponds to 1 percent probability of collapse in 50 years. The IBC identifies procedures to develop a maximum considered earthquake acceleration response spectrum, at the ground surface by adjusting Site Class B/C boundary spectra for local site conditions, similar to the methods used by AASHTO except that the probability of exceedance is lower (i.e., 2 percent in 50 years versus 7 percent in 75 years). However, the IBC defines the design response spectrum as two-thirds of the value of the maximum considered earthquake acceleration response spectrum. As is true for transportation structures, for critical or unique structures, for sites characterized as soil profile Type F (thick sequence of soft soils in the IBC) or liquefiable soils, or for soil conditions that do not adequately match the specification based soil profile types, site specific response analysis may be required as discussed in Appendix 6-A.

### 6.3.2 Site Ground Motion Response Analysis

#### 6.3.2.1 General Procedure

The AASHTO Guide Specifications for LRFD Bridge Seismic Design require that site effects be included in determining seismic loads for design of bridges. Article 3.4.1 of the AASHTO Guide Specifications for LRFD Bridge Seismic Design (also Article 3.10.4.1 of the AASHTO LRFD Bridge Design Specifications) provide requirements for developing a design response spectrum when using the General Procedure. When conducting a seismic design based on the General Procedure, the site response spectrum shall be developed in accordance with the AASHTO Guide Specifications for LRFD Bridge Seismic Design, except that the USGS 2014 deaggregation/ground motions as depicted in Figures 6-5, 6-6, and 6-7 shall be used to establish the PGA, Ss, and S1 accelerations used as input. With regard to characterization of the site subsurface conditions, Tables 6-4, 6-5, and 6-6 shall be used as input to establish the site seismic response spectrum instead of the site coefficients provided in the AASHTO specifications.
The guide specifications characterize all subsurface conditions with six Site Classes (A through F). The site soil coefficients for PGA ($F_{p_{\text{pga}}}$), SS ($F_a$), and $S_1$ ($F_{v}$) provided in the Guide Specifications are updated herein for use with the 2014 seismic acceleration maps. Site soil coefficients for five of the Site Classes (A through E) are provided in Tables 6-4, 6-5, and 6-6. Code/specification based response spectra that include the effect of ground motion amplification or de-amplification from the soil/rock stratigraphy at the site can be developed from the PGA, $S_S$, $S_1$ and the Site-Class based site coefficients $F_{p_{\text{pga}}}$, $F_a$, and $F_v$. Note that the site class should be determined considering the soils up to the ground surface, not just soil below the foundations.

The geotechnical designer shall determine the appropriate site coefficient ($F_{p_{\text{pga}}}$ for PGA, $F_a$ for SS, and $F_v$ for $S_1$) to construct the code/specification based response spectrum for the specific site subsurface conditions.

### Table 6-4  Values of Site Coefficient, $F_{p_{\text{pga}}}$, for Peak Ground Acceleration

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped Peak Ground Acceleration Coefficient (PGA)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PGA $\leq$ 0.10</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>0.9</td>
</tr>
<tr>
<td>C</td>
<td>1.3</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.4</td>
</tr>
<tr>
<td>F</td>
<td>*</td>
</tr>
</tbody>
</table>

* Site-specific response geotechnical investigation and dynamic site response analysis should be considered.

Note: Use straight line interpolation for intermediate values of PGA.

### Table 6-5  Values of Site Coefficient, $F_a$, for 0.2-sec Period Spectral Acceleration

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped Spectral Acceleration Coefficient at Period 0.2 sec ($S_S$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_S \leq$ 0.25</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>0.9</td>
</tr>
<tr>
<td>C</td>
<td>1.3</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.4</td>
</tr>
<tr>
<td>F</td>
<td>*</td>
</tr>
</tbody>
</table>

* Site-specific response geotechnical investigation and dynamic site response analysis should be considered.

Note: Use straight line interpolation for intermediate values of $S_S$. 
Table 6-6  Values of Site Coefficient, $F_v$, for 1.0-sec Period Spectral Acceleration

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$S_1 \leq 0.1$</th>
<th>$S_1 = 0.2$</th>
<th>$S_1 = 0.3$</th>
<th>$S_1 = 0.4$</th>
<th>$S_1 = 0.5$</th>
<th>$S_1 \geq 0.6$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>C</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.4</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
<td>2.2</td>
<td>2.0</td>
<td>1.9</td>
<td>1.8</td>
<td>1.7</td>
</tr>
<tr>
<td>E</td>
<td>4.2</td>
<td>3.3</td>
<td>2.8</td>
<td>2.4</td>
<td>2.2</td>
<td>2.0</td>
</tr>
<tr>
<td>F</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
</tbody>
</table>

* Site-specific response geotechnical investigation and dynamic site response analysis should be considered.

Note: Use straight line interpolation for intermediate values of $S_1$.

6-3.2.2  Site Specific Ground Motion Response Analysis

When to Conduct: A site specific ground motion response analysis shall be performed in the following situations:

- The facility is identified as critical or essential,
- Sites where geologic conditions are likely to result in un-conservative spectral acceleration values if the generalized code response spectra is used (e.g., within the upper 100 ft a sharp change in impedance between subsurface strata is present, etc.), or
- Site subsurface conditions are classified as Site Class F, and in some cases Site Class E as identified in Table 6-5.

There may be other reasons why the general procedure cannot be used, such as the situation where the spectral acceleration coefficient at 1.0 second is greater than the spectral acceleration coefficient at 0.2 second. In such cases, a site specific ground motion analysis should be conducted. A site specific ground motion response analysis should also be considered for sites where:

- the effects of liquefaction on the ground motion response could be overly conservative.
- basin effects could have a strong impact on the ground motion. However, the current (2014) acceleration maps partially consider basin effects. Whether or not basin effects should be considered for a particular site will be determined on a case by case basis as directed by the State Geotechnical Engineer and State Bridge Engineer.

Note that where the response spectrum is developed using a site-specific hazard analysis, a site specific ground motion response analysis, or both, the AASHTO specifications require that the spectrum not be lower than two-thirds of the response spectrum at the ground surface determined using the general procedure as specified in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, Article 3.4.1. For this comparison, the general procedure response spectrum is adjusted by the site coefficients ($F_{pga}$) in Tables 6-4, 6-5, and 6-6 in the region of $0.5T_F$ to $2T_F$ of the spectrum, where $T_F$ is the bridge fundamental period. For other analyses such as liquefaction assessment and retaining wall design, the free field acceleration at the ground surface determined from a site specific analysis should not be less than two-thirds of the PGA multiplied by the specification based site coefficient $F_{pga}$.
No site coefficients are available for Site Class F and in some cases Site Class E. In these cases, a site specific ground response analysis shall be conducted (see the AASHTO Guide Specifications for LRFD Bridge Seismic Design for additional details on site conditions that are considered to be included in Site Class F). Furthermore, there are no site coefficients for liquefiable soils. No consensus currently exists regarding the appropriate site coefficients for these cases. When estimating the minimum ground surface response spectrum using two-thirds of the response spectrum from the specification based procedures provided in the AASHTO Guide Specifications for LRFD Seismic Bridge Design and as provided herein, unless directed otherwise by the State Geotechnical Engineer and the State Bridge Engineer, the following approach shall be used:

- For liquefiable sites, use the specification based site coefficient for soil conditions without any modifications for liquefaction. This approach is believed to be conservative for higher frequency motions (i.e., TF < 1.0 sec).

- If a site specific ground response analysis is conducted, the response spectrum shall not be lower than two-thirds of the non-liquefied specification based spectrum, unless specifically approved by the State Bridge and Geotechnical Engineers to go lower. When accepting a spectrum lower than the specification based spectrum, the uncertainties in the analysis method should be carefully reviewed, particularly for longer periods (i.e., T > 1.0 sec.) where increases in the spectral ordinate may occur. Because of this, for structures that are characterized as having a fundamental period, TF, greater than 1.0 sec., a site specific ground response analysis shall be conducted if liquefiable soils are determined to be present.

Sites that contain a strong impedance contrast, i.e., a boundary between adjacent layers with shear wave velocities that differ by a factor of 2 or more are not specifically considered in the site soil coefficients and a site- specific seismic ground response analysis should be conducted. The strong impedance contrast can occur where a thin soil profile (e.g., < 20 to 30 feet) overlies rock or where layers of soft and stiff soils occur.

**How to Conduct:** Input ground motion (i.e., acceleration time histories) selection and processing (e.g., matching through scaling with consideration to a target spectrum) for site specific ground motion response analyses should be conducted using procedures provided in Kramer et al. (2012). A WSDOT website link to the ground motion selection and processing tool cited in that reference (i.e., a modified version of SigmaSpectra with a ground motion database developed for Washington) is as follows: http://www.wsdot.wa.gov/Business/MaterialsLab/GeotechnicalServices.htm

Additional background and guidance on the subject of input ground motion selection and processing to produce a site specific base rock spectrum for conducting a site specific ground motion response analysis is provided in Kramer (1996), Bommer and Acevedo (2004), NEHRP (2011), and Kavazanjian, et al. (2011).

Once the input (i.e., base rock) ground motions are established, the frequency domain site specific response spectra needed for structure design (also commonly referred to as a site response analysis) is developed based on the requirements in Appendix 6-A.5. For the more complex sites or structures, a nonlinear time history analysis may be necessary. Appendix 6-A.6 provides requirements for conducting time history analysis to obtain the needed ground motions for structure design.
See Appendix 6-A for additional requirements and guidance regarding site specific
ground response analyses, including requirements for time history analyses. Matasovic
and Hashash (2012) also provide a good overview of the process used to conduct site
specific ground motion response analysis from development of input ground motions to
development of the structure design response spectra.

6-3.3 Need for Peer Review of Site Specific Hazard and Ground Motion
Response Analyses

If a site specific hazard analysis is conducted, it shall be independently peer reviewed
in all cases by someone with expertise in site specific seismic hazard analyses. When
the site specific hazard analysis is conducted by a consultant working for the State or a
design-builder, the peer reviewer shall not be a staff member of the consultant(s) doing
the engineering design for the project, even if not part of the specific team within those
consultants doing the project design. The expert peer reviewer must be completely
independent of the design team consultant(s).

A site specific ground motion response analysis to establish a response spectrum that is
lower than two-thirds of the specification based spectrum shall be approved by the State
Geotechnical and Bridge Engineers. If the site specific response analysis is conducted
for this purpose, the site specific analysis shall be independently peer reviewed. The
peer reviewer shall meet the same requirements as described in the previous paragraph,
except that their expertise must be in the site specific ground motion response analysis
technique used to conduct the analysis.

6-3.4 IBC for Site Response

The IBC, Sections 1613 through 1615, provides procedures to estimate the earthquake
loads for the design of buildings and similar structures. Earthquake loads per the IBC are
defined by acceleration response spectra, which can be determined through the use of
the IBC general response spectrum procedures or through site-specific procedures. The
intent of the IBC MCE is to reasonably account for the maximum possible earthquake at a
site, to preserve life safety and prevent collapse of the building.

The general response spectrum per the IBC utilizes mapped Maximum Considered
Earthquake (MCE) spectral response accelerations at short periods ($S_s$) and at 1-second
($S_1$) to define the seismic hazard at a specific location in the United States.

The IBC uses the six site classes, Site Class A through Site Class F, to account for the
effects of soil conditions on site response. The geotechnical designer shall identify
the appropriate Site Class for the site. Note that the site class should be determined
considering the soils up to the ground surface, not just soil below the foundations.

Once the Site Class and mapped values of $S_s$ and $S_1$ are determined, values of the Site
Coefficients $F_a$ and $F_v$ (site response modification factors) can be determined. The Site
Coefficients and the mapped spectral accelerations $S_s$ and $S_1$ can then be used to define
the MCE and design response spectra. The PGA at the ground surface may be estimated
as 0.4 of the 0.2 sec design spectral acceleration.
For sites where Site Class F soils are present, the IBC requires that a site-specific geotechnical investigation and dynamic site response analysis be completed (see Appendix 6-A). Dynamic site response analysis may not be required for liquefiable soil sites for structures with predominant periods of vibration less than 0.5 seconds.

6-3.5 Determination of $A_s$ for Geotechnical Seismic Design

The ground acceleration $A_s$ is determined by multiplying the PGA from Figure 6-8, which provides the ground acceleration for Class B/C rock/soil conditions, by its site coefficient $F_{p_{ga}}$ (Table 6-4) to determine $A_s$ for other site classes. $A_s$ determined in this manner is used for assessing the potential for liquefaction and for the estimation of seismic earth pressures and inertial forces for retaining wall and slope design. For liquefaction assessment and retaining wall and slope design, the site coefficient presented in Table 6-4 shall be used, unless a site specific evaluation of ground response conducted in accordance with these AASHTO Guide specifications and Section 6-3 and Appendix 6-A is performed. Note that the site class should be determined considering the soils up to the ground surface, not just soil below the foundations.

6-3.6 Earthquake Magnitude

Assessment of liquefaction and lateral spreading require an estimate of the earthquake magnitude. The magnitude should be assessed using the seismic deaggregation data for the site, available through the USGS national seismic hazard website (earthquake.usgs.gov/hazards/) as discussed in Appendix 6-A. The deaggregation used shall be for a seismic hazard level consistent with the hazard level used for the structure for which the liquefaction analysis is being conducted (typically, a probability of exceedance of 5 percent in 50 years in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design). Additional discussion and guidance regarding the selection of earthquake magnitude values are provided in the AASHTO Guide Specifications for LRFD Bridge Seismic Design.

6-4 Seismic Geologic Hazards

The geotechnical designer shall evaluate seismic geologic hazards including fault rupture, liquefaction, lateral spreading, ground settlement, and slope instability. The potential effects associated with seismic geologic hazards shall be evaluated by the geotechnical designer.

6-4.1 Fault Rupture

Washington State is recognized as a seismically active region; however, only a relatively small number of active faults have been identified within the state. Thick sequences of recent geologic deposits, heavy vegetation, and the limited amount of instrumentally recorded events on identified faults are some of the factors that contribute to the difficulty in identifying active faults in Washington State. Considerable research is ongoing throughout Washington State to identify and characterize the seismicity of active faults, and new technology makes it likely that additional surface faults will be identified in the near future. The best source of fault information that can be considered for design is the USGS at the following website: https://earthquake.usgs.gov/hazards/qfaults
The potential impacts of fault rupture include abrupt, large, differential ground movements and associated damage to structures that might straddle a fault, such as a bridge. Until the recent application of advanced mapping techniques (e.g., LIDAR and aeromagnetics) in combination with trenching and age dating of apparent ground offsets, little information was available regarding the potential for ground surface fault rupture hazard in Washington State.

In view of the advances that will likely be made in the area of fault identification, the potential for fault rupture should be evaluated and taken into consideration in the planning and design of new facilities. These evaluations should incorporate the latest information identifying potential Holocene ground deformation.

6-4.2 Liquefaction

Liquefaction has been one of the most significant causes of damage to bridge structures during past earthquakes (ATC-MCEER Joint Venture, 2002). Liquefaction can damage bridges and structures in many ways including:
- Modifying the nature of ground motion;
- Bearing failure of shallow foundations founded above liquefied soil;
- Changes in the lateral soil reaction for deep foundations;
- Liquefaction induced ground settlement;
- Lateral spreading of liquefied ground;
- Large displacements associated with low frequency ground motion;
- Increased earth pressures on subsurface structures;
- Floating of buoyant, buried structures; and
- Retaining wall failure.

Liquefaction refers to the significant loss of strength and stiffness resulting from the generation of excess pore water pressure in saturated, predominantly cohesionless soils. Kramer (1996) provides a detailed description of liquefaction including the types of liquefaction phenomena, evaluation of liquefaction susceptibility, and the effects of liquefaction.

All of the following general conditions are necessary for liquefaction to occur:
- The presence of groundwater, resulting in a saturated or nearly saturated soil.
- Predominantly cohesionless soil that has the right gradation and composition. Liquefaction has occurred in soils ranging from low plasticity silts to gravels. Clean or silty sands and non-plastic silts are most susceptible to liquefaction.
- A sustained ground motion that is large enough and acting over a long enough period of time to develop excess pore-water pressure, equal to the effective overburden stress, thereby significantly reducing effective stress and soil strength,
- The state of the soil is characterized by a density that is low enough for the soil to exhibit contractive behavior when sheared undrained under the initial effective overburden stress.
Methods used to assess the potential for liquefaction range from empirically based design methods to complex numerical, effective stress methods that can model the time-dependent generation of pore-water pressure and its effect on soil strength and deformation. Furthermore, dynamic soil tests such as cyclic simple shear or cyclic triaxial tests can be used to assess liquefaction susceptibility and behavior to guide input for liquefaction analysis and design.

Liquefaction hazard assessment includes identifying soils susceptible to liquefaction, evaluating whether the design earthquake loading will initiate liquefaction, and estimating the potential effects of liquefaction on the planned facility. Liquefaction hazard assessment is required in the AASHTO Guide Specifications for LRFD Seismic Bridge Design if the site Seismic Design Category (SDC) is classified as SDC C or D, and the soil is identified as being potentially susceptible to liquefaction (see Section 6-4.2.1). The SDC is defined on the basis of the site-adjusted spectral acceleration at 1 second (i.e., $S_{D1} = F_v S_1$) where SDC C is defined as $0.30 \leq S_{D1} < 0.5$ and SDC D is defined as $S_{D1} \geq 0.50$. Where loose to very loose, saturated sands are within the subsurface profile such that liquefaction could impact the stability of the structure, the potential for liquefaction in SDC B ($0.15 \leq S_{D1} < 0.3$) should also be considered as discussed in the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

To determine the location of soils that are adequately saturated for liquefaction to occur, the seasonally averaged groundwater elevation should be used. Groundwater fluctuations caused by tidal action or seasonal variations will cause the soil to be saturated only during a limited period of time, significantly reducing the risk that liquefaction could occur within the zone of fluctuation.

For sites that require an assessment of liquefaction, the potential effects of liquefaction on soils and foundations shall be evaluated. The assessment shall consider the following effects of liquefaction:

- Loss in strength in the liquefied layer(s) with consideration of potential for void redistribution due to the presence of impervious layers within or bounding a liquefiable layer
- Liquefaction-induced ground settlement, including downdrag on deep foundation elements
- Slope instability induced by flow failures or lateral spreading

During liquefaction, pore-water pressure build-up occurs, resulting in loss of strength and then settlement as the excess pore-water pressures dissipate after the earthquake. The potential effects of strength loss and settlement include:

- **Slope Instability Due to Flow Failure or Lateral Spreading** – The strength loss associated with pore-water pressure build-up can lead to slope instability. Generally, if the factor of safety against liquefaction is less than approximately 1.2 to 1.3, a potential for pore-water pressure build-up will occur, and the effects of this build-up shall be assessed. If the soil liquefies, slope stability is determined using the residual strength of the soil to assess the potential for flow failure. The residual strength of liquefied soils can be estimated using empirical methods. Loss of soil resistance can allow abutment soils to move laterally, resulting in bridge substructure distortion and unacceptable deformations and moments in the superstructure. See Section 6-4.3.1 for additional requirements to assess flow failure and lateral spreading.
• **Reduced foundation bearing resistance** – The residual strength of liquefied soil is often a fraction of nonliquefied strength. This loss in strength can result in large displacements or bearing failure. For this reason spread footing foundations are not recommended where liquefiable soils exist unless the spread footing is located below the maximum depth of liquefaction or soil improvement techniques are used to mitigate the effects of liquefaction.

• **Reduced soil stiffness and loss of lateral support for deep foundations** – This loss in strength can change the lateral response characteristics of piles and shafts under lateral load.

Vertical ground settlement will occur as excess pore-water pressures induced by liquefaction dissipate, resulting in downdrag loads on and loss of vertical support for deep foundations. If liquefaction-induced downdrag loads can occur, the downdrag loads shall be assessed as specified in Sections 6-5.3 and 8-12.2.7, and in Article 3.11.8 in the AASHTO LRFD Bridge Design Specifications.

The effects of liquefaction will depend in large part on the amount of soil that liquefies and the location of the liquefied soil with respect to the foundation. On sloping ground, lateral flow, spreading, and slope instability can occur even on gentle slopes on relatively thin layers of liquefiable soils, whereas the effects of thin liquefied layer on the lateral response of piles or shafts (without lateral ground movement) may be negligible. Likewise, a thin liquefied layer at the ground surface results in essentially no downdrag loads, whereas the same liquefied layer deeper in the soil profile could result in large downdrag loads. Given these potential variations, the site investigation techniques that can identify relatively thin layers should be used part of the liquefaction assessment.

The following sections provide requirements for liquefaction hazard assessment and its mitigation.

6-4.2.1 **Methods to Evaluate Potential Susceptibility of Soil to Liquefaction**

Evaluation of liquefaction potential shall be completed based on soil characterization using in-situ testing such as Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT). Liquefaction potential may also be evaluated using shear wave velocity (V_s) testing and Becker Penetration Tests (BPT) for soils that are difficult to test using SPT and CPT methods, such as gravelly soils (see Andrus and Stokoe 2000); however, these methods are not preferred and are used less frequently than SPT or CPT methods. If the CPT method is used, SPT sampling and soil gradation testing shall still be conducted to obtain information on soil gradation parameters for liquefaction susceptibility assessment and to provide a comparison to CPT based analysis.

Simplified screening criteria to assess the potential liquefaction susceptibility of sands and silts based on soil gradation and plasticity indices should be used. In general, gravelly sands through low plasticity silts should be considered potentially liquefiable, provided they are saturated and very loose to medium dense.

If a more refined analysis of liquefaction potential is needed, laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate liquefaction susceptibility and initiation in lieu of empirical soil gradation/PI/density criteria, in accordance with Section 6-4.2.6.
**Preliminary Screening** – A detailed evaluation of liquefaction potential is required if all of the following conditions occur at a site, and the site Seismic Design Category is classified as SDC C or D:

- The estimated maximum groundwater elevation at the site is determined to be within 50 feet of the existing ground surface or proposed finished grade, whichever is lower.
- The subsurface profile is characterized in the upper 75 feet as having low plasticity silts, sand, or gravelly sand with a measured SPT resistance, corrected for overburden depth and hammer energy ($N_{160}$), of 25 blows/ft or less, or a cone tip resistance ($q_{cN}$) of 150 or less, or a geologic unit is present at the site that has been observed to liquefy in past earthquakes. For low plasticity silts and clays, the soil is considered liquefiable as defined by the Bray and Sancio (2006) or Boulanger and Idriss (2006) criteria.

For loose to very loose sand sites [e.g., $(N_{160} \leq 10 \text{ bpf or } q_{c1N} \leq 75)$], a potential exists for liquefaction in SDC B, if the acceleration coefficient, $A_s$ (i.e., $PGA \times F_{pga}$), is 0.15 or higher. The potential for and consequences of liquefaction for these sites will depend on the dominant magnitude for the seismic hazard and just how loose the soil is. As the magnitude decreases, the liquefaction resistance of the soil increases due to the limited number of earthquake loading cycles. Generally, if the magnitude is 6 or less, the potential for liquefaction, even in these very loose soils, is either very low or the extent of liquefaction is very limited. Nevertheless, a liquefaction assessment should be made if loose to very loose sands are present to a sufficient extent to impact bridge stability and $A_s$ is greater than or equal to 0.15. These loose to very loose sands are likely to be present in hydraulically placed fills and alluvial or estuarine deposits near rivers and waterfronts. See Idriss and Boulanger (2008) for additional information that relates liquefaction susceptibility to the depositional environment and geologic age of the deposit.

If the site meets the conditions described above, a detailed assessment of liquefaction potential shall be conducted. If all conditions are met except that the water table depth is greater than 50 feet but less than 75 feet, a liquefaction evaluation should still be considered, and if deep foundations are used, the foundation tips shall be located below the bottom of the liquefiable soil, or adequately above the liquefiable zone such that the impact of the liquefaction does not cause bridge or wall collapse.

**Liquefaction Susceptibility of Silts** – Liquefaction susceptibility of silts should be evaluated using the criteria developed by Bray and Sancio (2006) or Boulanger and Idriss (2006) if laboratory cyclic triaxial or cyclic simple shear tests are not conducted. The Modified Chinese Criteria (Finn, et al., 1994) that has been in use in the past has been found to be unconservative based on laboratory and field observations (Boulanger and Idriss, 2006). Therefore, the new criteria proposed by Bray and Sancio or Boulanger and Idriss are recommended. According to the Bray and Sancio criteria, fine-grained soils are considered susceptible to liquefaction if:

- The soil has a water content ($w_c$) to liquid limit ($LL$) ratio of 0.85 or more; and
- The soil has a plasticity index (PI) of less than 12.

For fine grained soils that are outside of these ranges of plasticity, cyclic softening resulting from seismic shaking may need to be considered. According to the Boulanger and Idriss (2006) criterion, fine grained soils are considered susceptible to liquefaction if the soil has a PI of less than 7. Since there is a significant difference in the screening criteria for liquefaction of silts in the current literature, for soils that are marginally
susceptible or not susceptible to liquefaction, cyclic triaxial or simple shear laboratory testing of undisturbed samples is recommended to assess whether or not the silt is susceptible to liquefaction, rather than relying solely on the screening criteria.

**Liquefaction Susceptibility of Gravels** – Other than through correlation to shear wave velocity as described in Andrus and Stokoe (2000), no specific guidance regarding susceptibility of gravels to liquefaction is currently available. The primary reason why gravels may not liquefy is that their high permeability frequently precludes the development of undrained conditions during and after earthquake loading. When bounded by lower permeability layers, however, gravels should be considered susceptible to liquefaction and their liquefaction potential evaluated. A gravel that contains sufficient sand to reduce its permeability to a level near that of the sand, even if not bounded by lower permeability layers, should also be considered susceptible to liquefaction and its liquefaction potential evaluated as such. Becker hammer testing and sampling, or sonic coring, could be useful for obtaining a representative sample of the sandy gravel that can be used to get an accurate soil gradation for assessing liquefaction potential. Downhole suspension logging (suspension logging in a mud rotary hole, not cased boring) should also be considered in such soils, as high quality $V_s$ testing can overcome the variation in SPT test results caused by the presence of gravels.

### 6-4.2.2 Determination of Whether or Not a Soil will Liquefy

The most common method of assessing liquefaction involves the use of empirical methods (i.e., Simplified Procedures). These methods provide an estimate of liquefaction potential based on SPT blowcounts, CPT cone tip resistance, BPT blowcounts, or shear wave velocity. This type of analysis shall be conducted as a baseline evaluation, even when more rigorous methods are used. More rigorous, nonlinear, dynamic, effective stress computer models may be used for site conditions or situations that are not modeled well by the simplified methods, subject to the approval of the State Geotechnical Engineer. For situations where simplified (empirical) procedures are not allowed (e.g., to assess liquefaction at depths greater than 50 to 80 ft as described in Section 6-1.2.3), these more rigorous computer models should be used, and independent peer review, as described in Section 6-3, of the results from these more rigorous computer models shall be conducted.

**Simplified Procedures** – Procedures that should be used for evaluating liquefaction susceptibility using SPT, CPT, $V_s$, and BPT criteria are provided in Youd et al. (2001). Youd et al. summarize the consensus of the profession up to year 2000 regarding the use of the simplified (i.e., empirical) methods. Since the publication of this consensus paper, various other modifications to the consensus approach have been introduced, including those by Cetin et al. (2004), Moss et al. (2006), Boulanger and Idriss (2006, 2014), and Idriss and Boulanger (2008). These more recent modifications to these methods account for additions to the database on liquefaction, as well as refinements in the interpretation of case history data. The updated methods potentially offer improved estimates of liquefaction potential, and should be considered for use. National Academies of Sciences, Engineering, and Medicine (2016) provides the most recent consensus report on liquefaction and should be consulted to obtain the most up to date consensus guidance on this subject.
The simplified procedures are based on comparing the cyclic resistance ratio (CRR) of a soil layer (i.e., the cyclic shear stress required to cause liquefaction) to the earthquake induced cyclic shear stress ratio (CSR). The CRR is a function of the soil relative density as represented by an index property measure (e.g., SPT blowcount), the fines content of the soil taken into account through the soil index property used, the in-situ vertical effective stress as represented by a factor $K_o$, an earthquake magnitude scaling factor, and possibly other factors related to the geologic history of the soil. The soil index properties are used to estimate liquefaction resistance based on empirical charts relating the resistance available to specific index properties (i.e., SPT, CPT, BPT or shear wave velocity values) and corrected to an equivalent magnitude of 7.5 using a magnitude scaling factor. The earthquake magnitude is used to empirically account for the duration of shaking or number of cycles.

The basic form of the simplified procedures used to calculate the earthquake induced CSR for the Simplified Method is as shown in Equation 6-8:

$$CSR = 0.65 \frac{A_{\text{max}}}{g} \frac{\sigma_o}{\sigma_{o}'} \frac{r_d}{MSF} \quad (6-8)$$

where,
- $A_{\text{max}}$ = peak ground acceleration accounting for site amplification effects
- $g$ = acceleration due to gravity
- $\sigma_o$ = initial total vertical stress at depth being evaluated
- $\sigma_{o}'$ = initial effective vertical stress at depth being evaluated
- $r_d$ = stress reduction coefficient
- $MSF$ = magnitude scaling factor

Note that $A_{\text{max}}$ is the PGA times the acceleration due to gravity, since the PGA is actually an acceleration coefficient, and $A_{\text{max}}/g$ is equal to $A_g$.

The factor of safety against liquefaction is defined by Equation 6-9:

$$FS_{\text{liq}} = \frac{\text{CRR}}{\text{CSR}} \quad (6-9)$$

The SPT procedure has been most widely used and has the advantage of providing soil samples for gradation and Atterberg limits testing. The CPT provides the most detailed soil stratigraphy, is less expensive, can provide shear wave velocity measurements, and is more reproducible. If the CPT is used, soil samples shall be obtained using the SPT or other methods so that detailed gradational and plasticity analyses can be conducted. The use of both SPT and CPT procedures can provide a detailed liquefaction assessment for a site.

Where SPT data is used, sampling and testing shall be conducted in accordance with Chapter 3. In addition:

- Correction factors for borehole diameter, rod length, hammer type, and sampler liners shall be used, where appropriate.
- Where gravels or cobbles are present, the use of short interval adjusted SPT N values may be effective for estimating the N values for the portions of the sample not affected by gravels or cobbles.
• Blowcounts obtained when sampling using Dames and Moore or modified California samplers or non-standard hammer weights and drop heights, including wireline and downhole hammers, shall not be used for liquefaction evaluations.

As discussed in Section 6-1.2.2, the limitations of the simplified procedures should be recognized. The simplified procedures were developed from empirical evaluations of field observations. Most of the case history data was collected from level to gently sloping terrain underlain by Holocene-age alluvial or fluvial sediment at depths less than 50 feet. Therefore, the simplified procedures are most directly applicable to these site conditions. Caution should be used for evaluating liquefaction potential at depths greater than 50 feet using the simplified procedures. In addition, the simplified procedures estimate the earthquake induced cyclic shear stress ratio based on a coefficient, $r_d$, that is highly variable at depth as discussed in Section 6-1.2.2.

As an alternative to the use of the $r_d$ factor, to improve the assessment of liquefaction potential, especially at greater depths, if soft or loose soils are present, equivalent linear or nonlinear site specific, one dimensional ground response analyses may be conducted to determine the maximum earthquake induced shear stresses at depth in the Simplified Method. For example, the linear total stress computer programs ProShake (EduPro Civil Systems, 1999), Shake2000 (Ordoñez, 2000), or DEEPSOIL (Hashash, et al., 2016) may be used for this purpose. Consideration should be given to the consistency of site specific analyses with the procedures used to develop the liquefaction resistance curves. A minimum of seven time histories (see Section 6-3.2.2 and Appendix 6-A) should be used to conduct these analyses to obtain a reasonably stable mean $r_d$ value as a function of depth.

Nonlinear Effective Stress Methods – An alternative to the simplified procedures for evaluating liquefaction susceptibility is to complete a nonlinear, effective stress site response analysis utilizing a computer code capable of modeling pore water pressure generation and dissipation. This is a more rigorous analysis that requires additional parameters to describe the stress-strain behavior and pore pressure generation characteristics of the soil.

The advantages with this method of analysis include the ability to assess liquefaction potential at all depths, including those greater than 50 feet, and the effects of liquefaction and large shear strains on the ground motion. In addition, pore-water redistribution during and following shaking can be modeled, seismically induced deformation can be estimated, and the timing of liquefaction and its effects on ground motion at and below the ground surface can be assessed.

Several one-dimensional non-linear, effective stress analysis programs are available for estimating liquefaction susceptibility at depth, and these methods are being used more frequently by geotechnical designers. However, a great deal of caution needs to be exercised with these programs, as there has been little verification of the ability of these programs to predict liquefaction at depths greater than 50 feet. This limitation is partly the result of the very few well documented sites with pore-water pressure measurements during liquefaction, either at shallow or deep depths, and partly the result of the one-dimensional approximation. For this reason greater reliance must be placed on observed response from laboratory testing or centrifuge modeling when developing the soil and pore pressure models used in the effective stress analysis method. The success of the effective stress model is, therefore, tied in part to the ability of the laboratory or centrifuge modeling to replicate field conditions.
A key issue that can affect the results obtained from nonlinear effective stress analyses is whether or not, or how well, the pore pressure model used addresses soil dilation during shearing. Even if good pore pressure data from laboratory liquefaction testing is available, the models used in some effective stress analysis methods may not be sufficient to adequately model dilation during shearing of liquefied soils. This limitation may result in unconservative predictions of ground response when a deep layer liquefies early during ground shaking. The inability to transfer energy through the liquefied layer could result in "shielding" of upper layers from strong ground shaking, potentially leading to an unconservative site response (see Anderson, et al. 2011 for additional explanation and guidance regarding effective stress modeling). See Appendix 6-A for additional considerations regarding modeling accuracies.

Two-dimensional effective stress analysis models can overcome some of these deficiencies, provided that a good soil and pore pressure model is used (e.g., the UBC sand model) – see Appendix 6-A. However, they are even more complex to use and certainly not for novice designers.

It should also be recognized that the results of nonlinear effective stress analyses can be quite sensitive to soil parameters that are often not as well established as those used in equivalent linear analyses. Therefore, it is incumbent upon the user to calibrate the model, evaluate the sensitivity of its results to any uncertain parameters or modeling assumptions, and consider that sensitivity in the interpretation of the results. Therefore, the geotechnical designer shall provide documentation that their model has been validated and calibrated with field data, centrifuge data, and/or extensive sensitivity analyses.

Analysis results from nonlinear effective stress analyses shall not be considered sufficient justification to conclude that the upper 40 to 50 feet of soil will not liquefy as a result of the ground motion dampening effect (i.e., shielding, or loss of energy) caused by deeper liquefiable layers. However, the empirical liquefaction analyses identified in this section may be used to justify that soil layers and lenses within the upper 65 feet of soil will not liquefy. This soil/pore pressure model deficiency for nonlinear effective stress methodologies could be crudely and conservatively addressed by selectively modifying soil parameters and/or turning off the pore pressure generation in given layers to bracket the response.

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 shall be conducted.

6-4.2.3 Minimum Factor of Safety Against Liquefaction

Liquefaction hazards assessment and the development of hazard mitigation measures shall be conducted if the factor of safety against liquefaction (Equation 6-9) is less than 1.2 or if the soil is determined to be liquefiable for the return period of interest (e.g., 975 years) using the performance based approach as described by Kramer and Mayfield (2007) and Kramer (2007). Note that for silts and low plasticity clays, a factor of safety is not calculated – the basis for determining whether or not liquefaction will occur is through cyclic simple shear or cyclic triaxial testing, or just whether or not the liquefaction susceptibility criteria are met. The hazard level used for this analysis shall be consistent
with the hazard level selected for the structure for which the liquefaction analysis is being conducted (typically, a probability of exceedance of 7 percent in 75 years in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design). While performance based techniques can be accomplished using the WSLIQ software (Kramer, 2007), the performance based option (as well as the multi-hazard option) in that software uses the 2002 USGS ground motions and has not been updated to include more recent ground motion data that would be consistent with the ground motions used to produce the 2014 USGS seismic maps. Until that software is updated to use the new ground motion database, the multi-hazard and performance based options in WSLIQ shall not be used. Liquefaction hazards to be assessed include settlement and related effects, and liquefaction induced instability (e.g., flow failure or lateral spreading), and the effects of liquefaction on foundations.

### 6-4.2.4 Liquefaction Induced Settlement

Both dry and saturated deposits of loose granular soils tend to densify and settle during and/or following earthquake shaking. Settlement of unsaturated granular deposits is discussed in Section 6-4.4. Settlement of saturated granular deposits due to liquefaction shall be estimated using techniques based on the Simplified Procedure, or if nonlinear effective stress models are used to assess liquefaction in accordance with Section 6-4.4.2, such methods may also be used to estimate liquefaction settlement.

If the Simplified Procedure is used to evaluate liquefaction potential, liquefaction induced ground settlement of saturated granular deposits should be estimated using the procedures by Tokimatsu and Seed (1987) or Ishihara and Yoshimine (1992). The Tokimatsu and Seed (1987) procedure estimates the volumetric strain as a function of earthquake induced CSR and corrected SPT blowcounts. The Ishihara and Yoshimine (1992) procedure estimates the volumetric strain as a function of factor of safety against liquefaction, relative density, and corrected SPT blowcounts or normalized CPT tip resistance. Updated procedures for estimating liquefaction settlement using CPT data are also provided in Zhang, et al. (2002). Example charts used to estimate liquefaction induced settlement using the Tokimatsu and Seed procedure and the Ishihara and Yoshimine procedure are presented as Figures 6-8 and 6-9, respectively.

If a more refined analysis of liquefaction induced settlement is needed, laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate the liquefaction induced vertical settlement in lieu of empirical SPT or CPT based criteria, in accordance with Section 6-4.2.6.

The empirically based analyses should be conducted as a baseline evaluation, even when laboratory volumetric strain test results are obtained and used for design, to qualitatively check the reasonableness of the laboratory test results.
Figure 6-8  Liquefaction Induced Settlement Estimated Using the Tokimatsu and Seed procedure (Tokimatsu and Seed, 1987)

Figure 6-9  Liquefaction Induced Settlement Estimated Using the Ishihara and Yoshimine procedure (Ishihara and Yoshimine, 1992)
6-4.2.5  **Residual Strength Parameters**

Liquefaction induced instability is strongly influenced by the residual strength of the liquefied soil. Instability occurs when the shear stresses required to maintain equilibrium exceed the residual strength of the soil deposit. Evaluation of residual strength of a liquefied soil deposit is one of the most difficult problems in geotechnical practice (Kramer, 1996). A variety of empirical methods are available to estimate the residual strength of liquefied soils. The empirical relationships provided in Figures 6-1 through 6-3 and Table 6-3 shall be used to estimate residual strength of liquefied soil unless soil specific laboratory performance tests are conducted as described below. These procedures for estimating the residual strength of a liquefied soil deposit are based on an empirical relationship between residual undrained shear strength and equivalent clean sand SPT blowcounts or CPT $q_{c,1n}$ values, using the results of back-calculation of the apparent shear strengths from case histories of large displacement flow slides. The significant level of uncertainty in these estimates of residual strength should be taken into account in design and evaluation of calculation results. See Section 6-2.2 for additional requirements regarding this issue.

If a more refined analysis of residual strength is needed, laboratory cyclic triaxial shear or cyclic simple shear testing may be used to evaluate the residual strength in lieu of empirical SPT or CPT based criteria, in accordance with Section 6-4.2.6.

The empirically based analyses should be conducted as a baseline evaluation, even when laboratory residual shear strength test results are obtained and used for design, to qualitatively check the reasonableness of the laboratory test results. The final residual shear strength value selected should also consider the shear strain level in the soil that can be tolerated by the structure or slope impacted by the reduced shear strength in the soil (i.e., how much lateral deformation can the structure tolerate?). Numerical modeling techniques may be used to determine the soil shear strain level that results in the maximum tolerable lateral deformation of the structure being designed.

6-4.2.6  **Assessment of Liquefaction Potential and Effects Using Laboratory Test Data**

If a more refined analysis of liquefaction potential, liquefaction induced settlement, or residual strength of liquefied soil is needed, laboratory cyclic simple shear or cyclic triaxial shear testing may be used in lieu of empirical soil gradation/PI/density (i.e., SPT or CPT based) criteria, if high quality undisturbed samples can be obtained. Laboratory cyclic simple shear or cyclic triaxial shear testing may also be used to evaluate liquefaction susceptibility of and effects on sandy soils from reconstituted soil samples. However, due to the difficulties in creating soil test specimens that are representative of the actual in-situ soil, liquefaction testing of reconstituted soil may be conducted only if approved by the State Geotechnical Engineer. Requests to test reconstituted soil specimens will be evaluated based on how well the proposed specimen preparation procedure mimics the in-situ soil conditions and geologic history.

The number of cycles, and either the cyclic stress ratios (stress-controlled testing) or cyclic shear strain (strain-controlled testing) used during the cyclic testing to liquefy or to attempt to liquefy the soil, should cover the range of the number of cycles and cyclic loading anticipated for the earthquake/ground motion being modeled. Testing to more than one stress or strain ratio should be done to fully capture the range of stress or
strain ratios that could occur. Preliminary calculations or computer analyses to estimate the likely cyclic stresses and/or strains anticipated should be conducted to help provide a basis for selection of the cyclic loading levels to be used for the testing. The vertical confining stress should be consistent with the in-situ vertical effective stress estimated at the location where the soil sample was obtained. Therefore $K_0$ consolidation is required in triaxial tests.

Defining liquefaction in these laboratory tests can be somewhat problematic. Theoretically, initial liquefaction is defined as being achieved once the excess pore pressure ratio in the specimen, $r_u$, is at 100 percent. The assessment of whether or not this has been achieved in the laboratory tested specimen depends on how the pore pressure is measured in the specimen, and the type of soil contained in the specimen. As the soil gets siltier, the possibility that the soil will exhibit fully liquefied behavior (i.e., initial liquefaction) at a measured pore pressure in the specimen of significantly less than 100 percent increases. A more practical approach that should be used in this case is to use a strain based definition to identify the occurrence of enough cyclic softening to consider the soil to have reached a failure state caused by liquefaction. Typically, if the soil reaches shear strains during cyclic loading of 3 percent or more, the soil, for practical purposes, may be considered to have achieved a state equivalent to initial liquefaction.

Note that if the testing is carried out well beyond initial liquefaction, cyclic triaxial testing is not recommended. In that case, necking of the specimen can occur, making the cyclic triaxial test results not representative of field conditions.

For the purpose of estimating liquefaction induced settlement, after the cyclic shearing is completed, with the vertical stress left on the specimen, the vertical strain is measured as the excess pore pressure is allowed to dissipate.

Note that once initial liquefaction has been achieved, volumetric strains are not just affected by the excess pore pressure generated through cyclic loading, but are also affected by damage to the soil skeleton as cyclic loading continues. Therefore, to obtain a more accurate estimate of post liquefaction settlement, the specimen should be cyclically loaded to the degree anticipated in the field, which may mean continuing cyclic loading after initial liquefaction is achieved.

If the test results are to be used with simplified ground motion modeling techniques (e.g., specification based ground response analysis or total stress site specific ground motion analysis), volumetric strain should be measured only for fully liquefied conditions. If effective stress ground motion analysis (e.g., DEEPSOIL) is conducted, volumetric strain measurements should be conducted at the cyclic stress ratio and number of loading cycles predicted by the effective stress analysis for the earthquake being modeled at the location in the soil profile being modeled, whether or not that combination results in a fully liquefied state. Vertical settlement prediction should be made by using the laboratory test data to develop a relationship between the measured volumetric strain and either the shear strain in the lab test specimens or the excess pore pressure measured in the specimens, and correlating the predicted shear strain or excess pore pressure profile predicted from the effective stress analysis to the laboratory test results to estimate settlement from volumetric strain; however, the shear strain approach is preferred.

To obtain the liquefied residual strength, after the cyclic shearing is completed, the drain lines in the test should be left closed, and the sample sheared statically. If the test results are to be used with simplified ground motion modeling techniques (e.g., specification
based ground response analysis or total stress site specific ground motion analysis), residual strength should be measured only for fully liquefied conditions. If effective stress ground motion analysis (e.g., DEEPSOIL) is conducted, residual shear strength testing should be conducted at the cyclic stress ratio and number of loading cycles predicted by the effective stress analysis for the earthquake being modeled at the location in the soil profile being modeled, whether or not that combination results in a fully liquefied state.

See Kramer (1996), Seed. et al. (2003), and Idriss and Boulanger (2008) for additional details and cautions regarding laboratory evaluation of liquefaction potential and its effects.

6-4.2.7 Combining Seismic Inertial Loading with Analyses Using Liquefied Soil Strength

The number of loading cycles required to initiate liquefaction, and hence the time at which liquefaction is triggered, tends to vary with the relative density and composition of the soil (i.e., denser soils require more cycles of loading to cause initial liquefaction). Whether or not the geologic hazards that result from liquefaction (e.g., lateral soil displacement such as flow failure and lateral spreading, reduced soil stiffness and strength, and settlement/downdrag) are concurrent with the strongest portion of the design earthquake ground motion depends on the duration of the motion and the resistance of the soil to liquefaction. For short duration ground motions and/or relatively dense soils, liquefaction may be triggered near the end of shaking. In this case, the structure of interest is unlikely to be subjected to high inertial forces after the soil has reached a liquefied state, and the evaluation of the peak inertial demands on the structure can be essentially decoupled from evaluation of the deformation demands associated with soil liquefaction. However, for long-duration motions (which are usually associated with large magnitude earthquakes such as a subduction zone earthquake as described in GDM Appendix 6-A) and/or very loose soils, liquefaction may be triggered earlier in the motion, and the structure may be subjected to strong shaking while the soil is in a liquefied state.

There is currently no consensus on how to specifically address this issue of timing of seismic acceleration and the development of initial liquefaction and its combined impact on the structure. More rigorous analyses, such as by using nonlinear, effective stress methods, are typically needed to analytically assess this timing issue. Nonlinear, effective stress methods can account for the build-up in pore-water pressure and the degradation of soil stiffness and strength in liquefiable layers. Use of these more rigorous approaches requires considerable skill in terms of selecting model parameters, particularly the pore pressure model. The complexity of the more rigorous approaches is such that approval by the State Geotechnical Engineer to use these approaches is mandatory, and an independent peer reviewer with expertise in nonlinear, effective stress modeling shall be used to review the specific methods used, the development of the input data, how the methods are applied, and the resulting impacts.

While flow failure due to liquefaction is not really affected by inertial forces acting on the soil mass (see Section 6-4.3.1), it is possible that lateral forces on a structure and its foundations due to flow failure may be concurrent with the structure inertial forces if the earthquake duration is long enough (e.g., a subduction zone earthquake). Likewise, for lateral spreading, since seismic inertial forces are acting on the soil during the development of lateral spreading (see Section 6-4.3.1), logically, inertial forces may also
be acting on the structure itself concurrently with the development of lateral forces on the structure foundation.

However, there are several factors that may affect the magnitude of the structural inertial loads, if any, acting on the foundation. Brandenberg, et al. (2007a and b) provide examples from centrifuge modeling regarding the combined effect of lateral spreading and seismic structural inertial forces on foundation loads and some considerations for assessing these inertial forces. They found that the total load on the foundation was approximately 40 percent higher on average than the loads caused by the lateral spreading alone. However, the structural column used in this testing did not develop any plastic hinging, which, had it occurred could have resulted in structural inertial loads transmitted to the foundation that could have been as low as one-fourth of what was measured in this testing. Another factor that could affect the potential combination of lateral spreading and structural inertia loads is how close the foundation is to the initiation point (i.e., downslope end) for the lateral spreading, as it takes time for the lateral spread to propagate upslope and develop to its full extent.

The current AASHTO Guide Specifications for seismic design do allow the lateral spreading forces to be decoupled from bridge seismic inertial forces. However, the potential for some combined effect of lateral spread forces with structural inertial loads should be considered if the structure is likely to be subjected to strong shaking while the soil is in a liquefied state, especially if the foundation is located near the toe of the lateral spread or flow failure. In lieu of more sophisticated analyses such as dynamic-stress deformation analyses, 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 induced lateral ground movement has initiated), it should be assumed that the lateral spreading/flow failure forces on the foundations are combined with 25 percent of the structure inertial forces, or the plastic hinge force, whichever is less.

This timing issue also affects liquefaction-induced settlement and downdrag, in that settlement and downdrag do not generally occur until the pore pressures induced by ground shaking begin to dissipate after shaking ceases. Therefore, a de-coupled analysis is appropriate when considering liquefaction downdrag loads.

When considering the effect of liquefaction on the resistance of the soil to structure foundation loads both in the axial (vertical) and lateral (horizontal) directions, two analyses should be conducted to address the timing issue. For sites where liquefaction occurs around structure foundations, structures should be analyzed and designed in two configurations as follows:

- **Nonliquefied Configuration** – The structure should be analyzed and designed, assuming no liquefaction occurs using the ground response spectrum appropriate for the site soil conditions in a nonliquefied state, i.e., using P-Y curves derived from static soil properties.

- **Liquefied Configuration** – The structure as designed in nonliquefied configuration above should be reanalyzed assuming that the layer has liquefied and the liquefied soil provides the appropriate residual resistance for lateral and axial deep foundation response analyses consistent with liquefied soil conditions (i.e., modified P-Y curves, modulus of subgrade reaction, T-Z curves, axial soil frictional resistance). The design spectrum should be the same as that used in nonliquefied configuration. However,
this analysis does not include the lateral forces applied to the structure due to liquefaction induced lateral spreading or flow failure, except as noted earlier in this section with regard to large magnitude, long duration earthquakes.

With the approval of the State Bridge and State Geotechnical Engineers, a site-specific response spectrum (for site specific spectral analysis) or nonlinear time histories developed near the ground surface (for nonlinear structural analysis) that account for the modifications in spectral content from the liquefying soil may be developed. The modified response spectrum, and associated time histories, resulting from the site-specific analyses at the ground surface shall not be less than two-thirds of the spectrum (i.e., as applied to the spectral ordinates within the entire spectrum) developed using the general procedure described in the AASHTO Guide Specifications for LRFD Bridge Seismic Design, Article 3.4.1, modified by the site coefficients in Section 6-3.2 of this chapter. If the soil and bedrock conditions are classified as Site Class F, however, there is no AASHTO general procedure spectrum. In that case, the reduced response spectrum, and associated time histories, that account for the effects of liquefaction shall not be less than two-thirds of the site specific response spectrum developed from an equivalent linear or nonlinear total stress analysis (i.e., nonliquefied conditions), or alternatively a Site Class E response spectrum could be used for this purpose instead of the equivalent total stress analysis.

Designing structures for these two configurations should produce conservative results. Typically, the nonliquefied configuration will control the loads applied to the structure and therefore is used to determine the loads within the structure, whereas the liquefied configuration will control the maximum deformations in the structure and is therefore used to design the structure for deformation. In some cases, this approach may be more conservative than necessary, and the designer may use a more refined analysis to assess the combined effect of strong shaking and liquefaction impacts, considering that both effects may not act simultaneously. However, Youd and Carter (2005) suggest that at periods greater than 1 second, it is possible for liquefaction to result in higher spectral accelerations than occur for equivalent nonliquefied cases, all other conditions being equal. Site-specific ground motion response evaluations may be needed to evaluate this potential.

6-4.3 **Seismic Slope Instability and Deformation**

Slope instability can occur during earthquakes due to inertial effects associated with ground accelerations or due to weakening of the soil induced by the seismic shear strain. Inertial slope instability is caused by temporary exceedance of the soil strength by dynamic earthquake stresses. In general, the soil strength remains unaffected by the earthquake shaking in this case. Weakening instability is the result of soil becoming progressively weaker as shaking occurs such that the shear strength becomes insufficient to maintain a stable slope.

Seismic slope instability analysis is conducted to assess the impact of instability and slope deformation on structures (e.g., bridges, tunnels, and walls, including reinforced slopes steeper than 1.2H:1V and noise walls). However, in accordance with Section 6-1.2, slopes that do not impact such structures are generally not mitigated for seismic slope instability.

The scope of this section is limited to the assessment of seismic slope instability. The impact of this slope instability on the seismic design of foundations and walls is addressed.
in Sections 6-5.3 and 6-5.4 for foundations and Sections 15-4.10 through 15-4.12 for walls.

6-4.3.1  **Weakening Instability due to Seismic Loading**

Weakening instability occurs due to liquefaction or seismic shear strain induced weakening of sensitive fine grained soils. With regard to liquefaction induced weakening instability, earthquake ground motion induces stress and strain in the soil, resulting in pore pressure generation and liquefaction in saturated soil. As the soil strength decreases toward its liquefied residual value, two types of slope instability can occur: flow failure, and lateral spreading. These various types of weakening instability are described in the subsections that follow. How the impact of weakening instability due to liquefaction is addressed for design of structures is specified in Section 6-5.4.

**Weakening Instability not Related to Liquefaction** – This type of weakening instability depends on the sensitivity of the soil to the shear strain induced by the earthquake ground motion. Sensitive silts and clays fall into this category. For seismic stability design in this scenario, the stability shall be assessed with consideration to the lowest shear strength that is likely to occur during and after shaking. For example, glacially overconsolidated clays will exhibit a significant drop in strength to a residual value as deformation takes place (e.g., see Section 5-13.3). A seismic slope deformation analysis should be conducted to assess this potential. Since it is likely that most of the strong motion will have subsided by the time the deformation required to drop the soil to its residual strength has occurred, the seismic slope stability analysis typically does not need to include inertial forces due to seismic acceleration when seismic stability is evaluated using the residual shear strength of the sensitive silt or clay soil. However, if the deformation analysis shows that enough deformation to drop the soil shear strength to near its residual value can occur before strong motion ceases, then the slope stability analysis shall include seismic inertial forces in combination with the residual shear strength. For silts and clays with low to moderate sensitivity, a strength reduction of 10 to 15 percent to account for cyclic degradation is reasonable for earthquake magnitudes of 7.0 or more (Kavazanjian, et al. 2011). For clays with high sensitivity, cyclic shear strength tests should be conducted to assess the rate of strength reduction.

For this type of weakening instability, the minimum level of safety specified in Section 6-4.3.2 shall be met, considering the weakened state of the soil during and after shaking. Assessment of the impact of this type of instability on structures is addressed in Section 6-5.3 for foundations and Sections 15-4.10 through 15-4.12 for walls.

**Liquefaction Induced Flow Failure** – Liquefaction can lead to catastrophic flow failures driven by static shearing stresses that lead to large deformation or flow. Such failures are similar to debris flows and are characterized by sudden initiation, rapid failure, and the large distances over which the failed materials move (Kramer, 1996). Flow failures typically occur near the end of strong shaking or shortly after shaking. However, delayed flow failures caused by post-earthquake redistribution of pore water pressures can occur—particularly if liquefiable soils are capped by relatively impermeable layers.

The potential for liquefaction induced flow failures should be evaluated using conventional limit equilibrium slope stability analyses (see Section 6-4.3), using residual undrained shear strength parameters for the liquefied soil, and decoupling the analysis from all seismic inertial forces (i.e., performed with \( k_h \) and \( k_v \) equal to zero). If the limit
equilibrium factor of safety, FS, is less than 1.05, flow failure shall be considered likely. In these instances, the magnitude of deformation is usually too large to be acceptable for design of bridges or structures, and some form of mitigation will likely be needed. The exception is where the liquefied material and any overlying crust flow past the structure and the structure and its foundation system can resist the imposed loads. Where the factor of safety for this decoupled analysis is greater than 1.05 for liquefied conditions, deformation and stability shall be evaluated using a lateral spreading analysis (see the subsection “Lateral Spreading,” especially regarding cautions in conducting these types of analyses).

Residual strength values to be used in the flow failure analysis may be determined from empirical relationships (See Section 6-4.2.5) or from laboratory test results. If laboratory test results are used to assess the residual strength of the soil that is predicted to liquefy and potentially cause a flow failure, the shearing resistance may be very strain dependent. As a default, the laboratory mobilized residual strength value used should be picked at a strain of 2 percent, assuming the residual strength value is determined from laboratory testing as described in Section 6-4.2.6. A higher strain value may be used for this purpose, subject to the approval of the State Geotechnical Engineer and State Bridge Engineer, if it is known that the affected structure can tolerate a relatively large lateral deformation without collapse. Alternatively, numerical modeling may be conducted to develop the relationship between soil shear strain and slope deformation, picking a mobilized residual strength value that corresponds to the maximum deformation that the affected structure can tolerate.

With regard to flow failure prediction, even though there is a possibility that seismic inertial forces may be concurrent with the liquefied conditions (i.e., in long duration earthquakes), it is the static stresses that drive the flow failure and the deformations that result from the failure. The dynamic stresses present have little impact on this type of slope failure. Therefore, slope stability analyses conducted to assess the potential for flow failure resulting from liquefaction, and to estimate the forces that are applied to the foundation due to the movement of the soil mass into the structure, should be conducted without seismic inertial forces (i.e., k_h and k_v acting on the soil mass are set equal to zero).

**Lateral Spreading** – In contrast to flow failures, lateral spreading can occur when the shear strength of the liquefied soil is incrementally exceeded by the inertial forces induced during an earthquake or when soil stiffness degrades sufficiently to produce substantial permanent strain in the soil. The result of lateral spreading is typically horizontal movement of non-liquefied soils located above liquefied soils, in addition to the liquefied soils themselves. Lateral spreading analysis is by definition a coupled analysis (i.e., directly considers the effect of seismic acceleration), in contrast to a flow failure analysis, which is a decoupled seismic stability analysis.

If the factor of safety for slope stability from the flow failure analysis, assuming residual strengths in all layers expected to experience liquefied conditions, is 1.05 or greater, a lateral spreading/deformation analysis shall be conducted. If the liquefied layer(s) are discontinuous, the slope factor of safety may be high enough that lateral spreading does not need to be considered. This analysis also does not need to be conducted if the depth below the natural ground surface to the upper boundary of the liquefied layers is greater than 50 ft.
The potential for liquefaction induced lateral spreading on gently sloping sites or where the site is located near a free face shall be evaluated using one or more of the following empirical relationships:

- Youd et al. (2002)
- Kramer and Baska (2007)
- Zhang et al. (2004)

These procedures use empirical relationships based on case histories of lateral spreading and/or laboratory cyclic shear test results. Input into these models include earthquake magnitude, source-to-site distance, site geometry/slope, cumulative thickness of saturated soil layers and their characteristics (e.g., SPT N values, average fines content, average grain size). These empirical procedures provide a useful approximation of the potential magnitude of deformation that is calibrated against lateral spreading deformations observed in actual earthquakes. It should be noted, however, that the dataset used to develop these lateral spreading correlations is very limited for the upper end of earthquake magnitude (e.g., Mw > 8). Therefore, the potential for error in the estimate is greater for these very large magnitude earthquakes. In addition to the cited references for each method, see Kramer (2007) for details on how to carry out these methods. Kramer (2007) provides recommendations on the use of these methods which should be followed.

More complex analyses such as the Newmark time history analysis and dynamic stress deformation models, such as provided in two-dimensional, nonlinear effective stress computer programs (e.g., PLAXIS and FLAC), may also be used to estimate lateral spreading deformations. However, these analysis procedures have not been calibrated to observed performance with regard to lateral movements caused by liquefaction, and there are many complexities with regard to development of input parameters and application of the method to realistic conditions.

The Newmark time history analysis procedure is described in Anderson, et al. (2008) and Kavezanjian, et al. (2011). If a Newmark time history analysis is conducted to obtain an estimate of lateral spreading displacement, the number of cycles to initiate liquefaction for the time histories selected for analysis needs to be considered when selecting a yield acceleration to apply to the various portions of the time history. Initially, the yield acceleration will be high, as the soil will not have liquefied (i.e., non-liquefied soil strength parameters should be used to determine the yield acceleration). As the soil excess pore pressure begins to build up with additional loading cycles, the yield acceleration will begin to decrease. Once initial liquefaction or cyclic softening occurs, the residual strength is then used to determine the yield acceleration. Note that if the yield acceleration applied to the entire acceleration time history is based on residual strength consistent with liquefied conditions, the estimated lateral deformation will likely be overly conservative. To address this issue, an effective stress ground motion analysis (e.g., DEEPSOIL) should be conducted to estimate the build up of pore pressure and the development of liquefaction as the earthquake shaking continues to obtain an improved estimate of the drop in soil shear strength and yield acceleration as a function of time.

Simplified charts based on Newmark-type analyses shall not be used for estimating deformation resulting from lateral spreading. These simplified Newmark type analyses have some empirical basis built in with regard to estimation of deformation. However, they are not directly applicable to lateral spreading, as they were not developed for soil that weakens during earthquake shaking, as is the case for soil liquefaction.
If the more rigorous approaches are used, the empirically based analyses shall still be conducted to provide a baseline of comparison, to qualitatively check the reasonableness of the estimates from the more rigorous procedures. The more rigorous approaches should be used to evaluate the effect of various input parameters on deformation. See Youd, et al. (2002), Kramer (1996, 2007), Seed, et al. (2003) and Dickenson, et al. (2002) for additional background on the assessment of slope deformations resulting from lateral spreading.

A related issue is how far away the free face must be before lateral spreading need not be considered. Lateral spreading has been observed up to about 1,000 ft from the free face in past earthquakes (Youd, et al., 2002). Available case history data also indicate that deformations at L/H ratios greater than 20, where L is the distance from the free face or channel and H is the height of the free face of channel slope, are typically reduced to less than 20 percent of the lateral deformation at the free face (Idriss and Boulanger, 2008). Detailed analysis of the Youd, et al. database indicates that only two of 97 cases had observable lateral spreading deformation at L/H ratios as large as 50 to 70. If lateral spreading calculations using these empirical procedures are conducted at distances greater than 1,000 ft from the free face or L/H ratios greater than 20, additional evaluation of lateral spreading deformation using more complex or rigorous approaches should also be conducted.

At locations close to the free face (e.g., L/H < 5), displacement mechanisms more closely related to localized instabilities such as slumping could become more dominant. This should be considered when estimating displacements close to the free face.

6-4.3.2 Slope Instability Due to Inertial Effects

Even if the soil does not weaken as earthquake shaking progresses, instability can still occur due to the additional inertial forces acting on the soil mass during shaking. Inertial slope instability is caused by temporary exceedance of the soil strength by dynamic earthquake stresses.

Pseudo-static slope stability analyses shall be used to evaluate the seismic stability of slopes and embankments. The pseudo-static analysis consists of conventional limit equilibrium static slope stability analysis as described in Chapter 7 completed with horizontal and vertical pseudo-static acceleration coefficients (\(k_h\) and \(k_v\)) that act upon the critical failure mass. Kramer (1996) provides a detailed summary of pseudo-static analysis procedures.

For earthquake induced slope instability, with or without soil strength loss resulting from deformation induced by earthquake shaking (e.g., weakening instability due to strength loss in clays), the target factor of safety for the pseudo-static slope stability analysis is 1.1. When bridge foundations or retaining walls are involved, the LRFD approach shall be used, in which case a resistance factor of 0.9 shall be used for slope stability. Note that available slope stability programs produce a single factor of safety, FS. The specified resistance factor of 0.9 for slope stability is essentially the inverse of the FS that should be targeted in the slope stability program, which in this case is 1.1, making 0.9 the maximum resistance factor to be obtained when conducting pseudo-static slope stability analyses. If liquefaction effects dominate the stability of the slope and its deformation response (i.e., flow failure or lateral spreading occur), the procedures provided in Section 6-4.3.1 shall be used.
Unless a more detailed deformation analysis is conducted, a default horizontal pseudo-static coefficient, $k_h$, of 0.5$A_s$ and a vertical pseudo-static coefficient, $k_v$, equal to zero shall be used when seismic (i.e., pseudo-static) stability of slopes is evaluated, not considering liquefaction. This value of $k_h$ assumes that limited deformation of the slope during earthquake shaking is acceptable (i.e., 1 to 2 inches) and considers some wave scattering effects.

Due to the fact that the soil is treated as a rigid body in pseudo-static limit equilibrium analyses, and that the seismic inertial force is proportional to the square of the failure surface radius whereas the resistance is proportional to just the radius, the tendency is for the failure surface to move deeper and farther uphill relative to the static failure surface when seismic inertial loading is added. That is, the pseudo-static analysis assumes that the $k_h$ value applies uniformly to the entire failure mass regardless of how big the failure mass becomes. Since the soil mass is far from rigid, this can be an overly conservative assumption, in that the average value of $k_h$ for the failure mass will likely decrease relative to the input value of $k_h$ used for the stability assessment due to wave scattering effects.

The default value of $k_h$ should be increased to near 1.0 $A_s$ if a structure within or at the toe of the potentially unstable slope cannot tolerate any deformation. If slope movement can be tolerated, a reduced value of $k_h$ applied to the slope in the stability analysis may be used by accounting for both wave scattering (i.e., height) effects and deformation effects through a more detailed deformation based analysis. See Anderson, et al. (2008) and Kavezanjiam, et al. (2011) for the specific procedures to do this.

Deformation analyses should be employed where an estimate of the magnitude of seismically induced slope deformation is required, or to reduce $k_h$ for pseudo-static slope stability analysis below the default value of 0.5$A_s$ as described above. Acceptable methods of estimating the magnitude of seismically induced slope deformation are as provided in Anderson, et al. (2008) and Kavezanjian, et al. (2011), and include Newmark sliding block (time history) analysis as well as simplified procedures developed from Newmark analyses and numerical modeling. For global and sliding seismic stability analyses for walls, the procedures provided in the AASHTO LRFD Bridge Design Specifications should be used (specifically see Articles 11.6.5.2, 11.6.5.3, and Appendix A11).

### 6-4.4 Settlement of Dry Sand

Seismically induced settlement of unsaturated granular soils (dry sands) is well documented. Factors that affect the magnitude of settlement include the density and thickness of the soil deposit and the magnitude of seismic loading. The most common means of estimating the magnitude of dry sand settlement are through empirical relationships based on procedures similar to the Simplified Procedure for evaluating liquefaction susceptibility. The procedures provided by Tokimatsu and Seed (1987) for dry sand settlement should be used. The Tokimatsu and Seed approach estimates the volumetric strain as a function of cyclic shear strain and relative density or normalized SPT N values. The step by step procedure is provided in FHWA Geotechnical Engineering Circular No. 3 (Kavazanjian, et al., 2011).

Since settlement of dry sand will occur during earthquake shaking with downdrag forces likely to develop before the strongest shaking occurs, the axial forces caused by this phenomenon should be combined with the full spectral ground motion applied to the structure.
6-5  Input for Structural Design

6-5.1  Foundation Springs

Structural dynamic response analyses incorporate the foundation stiffness into the dynamic model of the structure to capture the effects of soil structure interaction. The foundation stiffness is typically represented as a system of equivalent springs using a foundation stiffness matrix. The typical foundation stiffness matrix incorporates a set of six primary springs to describe stiffness with respect to three translational and three rotational components of motion. Springs that describe the coupling of horizontal translation and rocking modes of deformation may also be used.

The primary parameters for calculating the individual spring stiffness values are the foundation type (shallow spread footings or deep foundations), foundation geometry, dynamic soil shear modulus, and Poisson's Ratio.

6-5.1.1  Shallow Foundations

For evaluating shallow foundation springs, the WSDOT Bridge and Structures Office requires values for the dynamic shear modulus, G, Poisson's ratio, and the unit weight of the foundation soils. The maximum, or low-strain, shear modulus G₀ can be estimated using index properties and the correlations presented in Table 6-2. Alternatively, the maximum shear modulus can be calculated using Equation 6-10 below, if the shear wave velocity is known:

\[ G₀ = \frac{\gamma}{g} (V_s)^2 \]  (6-10)

Where:
- \( G₀ \) = low strain, maximum dynamic shear modulus
- \( \gamma \) = soil unit weight
- \( V_s \) = shear wave velocity
- \( g \) = acceleration due to gravity

The maximum dynamic shear modulus is associated with small shear strains (typically less than 0.0001 percent). As the seismic ground motion level increases, the shear strain level increases, and dynamic shear modulus decreases. If the specification based general procedure described in Section 6-3 is used, the effective shear modulus, G, should be calculated in accordance with Table 4-7 in FEMA 356 (ASCE 2000), reproduced below as Table 6-7 for convenience. Note that \( S_{X5}/2.5 \) in the table is essentially equivalent to \( A_s \) (i.e., PGAxFPGA). This table reflects the dependence of G on both the shear strain induced by the ground motion and on the soil type (i.e., G drops off more rapidly as shear strain increases for softer or looser soils).

This table must be used with some caution, particularly where abrupt variations in soil profile occur below the base of the foundation. If the soil conditions within two foundation widths (vertically) of the bottom of the foundation depart significantly from the average conditions identified for the specific site class, a more rigorous method may be required. The more rigorous method may involve conducting one-dimensional equivalent linear ground response analyses using a program such as SHAKE to estimate the average effective shear strains within the zone affecting foundation response.
Table 6-7  Effective Shear Modulus Ratio (G/G₀)
(After ASCE 2000)

<table>
<thead>
<tr>
<th>Site Class</th>
<th>SXS/2.5 = 0</th>
<th>SXS/2.5 = 0.1</th>
<th>SXS/2.5 = 0.4</th>
<th>SXS/2.5 = 0.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>B</td>
<td>1.00</td>
<td>1.00</td>
<td>0.95</td>
<td>0.90</td>
</tr>
<tr>
<td>C</td>
<td>1.00</td>
<td>0.95</td>
<td>0.75</td>
<td>0.60</td>
</tr>
<tr>
<td>D</td>
<td>1.00</td>
<td>0.90</td>
<td>0.50</td>
<td>0.10</td>
</tr>
<tr>
<td>E</td>
<td>1.00</td>
<td>0.60</td>
<td>0.05</td>
<td>*</td>
</tr>
<tr>
<td>F</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
</tbody>
</table>

Notes: Use straight-line interpolation for intermediate values of SXS/2.5.

* Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Alternatively, site specific measurements of shear modulus may be obtained. Measured values of shear modulus may be obtained from laboratory tests, such as the cyclic triaxial, cyclic simple shear, or resonant column tests, or they may be obtained from in-situ field testing. If the specification based general procedure is used to estimate ground motion response, the laboratory or in-situ field test results may be used to calculate G₀. Then the table from FEMA 356 (ASCE, 2000) reproduced above can be used to determine G/G₀. However, caution should be exercised when using laboratory testing to obtain this parameter due to the strong dependency of this parameter on sample disturbance. Furthermore, the low-strain modulus developed from lab test should be adjusted for soil age if the footing is placed on native soil. The age adjustment can result in an increase in the lab modulus by a factor of 1.5 or more, depending on the quality of the laboratory sample and the age of the native soil deposit. The age adjustment is not required if engineered fill will be located within two foundation widths of the footing base. The preferred approach is to measure the shear wave velocity, Vₛ, through in-situ testing in the field, to obtain G₀.

If a detailed site specific ground response analysis is conducted, either Figures 6-1 and 6-2 may be used to estimate G in consideration of the shear strains predicted through the site specific analysis (the effective shear strain, equal to 65 percent of the peak shear strain, should be used for this analysis), or laboratory test results may be used to determine the relationship between G/G₀ and shear strain.

Poisson’s Ratio, v, should be estimated based on soil type, relative density/consistency of the soils, and correlation charts such as those presented in Chapter 5 or in the textbook, Foundation Analysis and Design (Bowles, 1996). Poisson’s Ratio may also be obtained from field measurements of p- and s-wave velocities.

Once G and v are determined, the foundation stiffness values should be calculated as shown in FEMA 356 (ASCE, 2000).
6-5.1.2 Deep Foundations

Lateral soil springs for deep foundations shall be determined in accordance with Chapter 8. However, if soil liquefaction is likely to occur, then the effect of liquefaction on both the shape and the magnitude of the P-Y curves provided in this section shall be followed.

Available models used to estimate P-Y curves for liquefied soil vary considerably, which may affect the accuracy of the predicted behavior during liquefaction. Typical approaches that have been used in the past to address the effect of liquefaction on P-Y curves include the following:

1. Use the soft clay P-Y model, using the undrained residual strength as the cohesive strength for development of the P-Y curve as suggested by Wang and Reese (1998);
2. Use the static sand P-Y curve model, but with the peak shear strength reduced by a p-multiplier as recommended by Brandenberg, et al. (2007b) and Boulanger, et al. (2003);
3. Assume that the liquefied soil provides no resistance to lateral movement; and
4. Liquefied sand model as developed by Rollins, et al. (2005a, 2005b), and as applied in deep foundation lateral load analysis computer programs such as LPile (Isenhower and Wang 2015).

These approaches are conceptually illustrated in Figure 6-10.

Weaver, et al. (2005) and Rollins, et al. (2005a) provided comparisons between the various methods for developing P-Y parameters for liquefied soil and the measured lateral load response of a full scale pile foundation in liquefied soil (i.e., liquefied using blast loading). They concluded that none of the simplified methods that utilize adjusted soil parameters applied to static P-Y clay or sand models (i.e., approaches 1 and 2 identified above) accurately predicted the measured lateral pile response to load due to the difference in curve shape for static versus liquefied conditions (i.e., convex, or strain softening P-Y curves that will result from approaches 1 and 2, versus concave, or strain hardening, shape that will result from approach 4, respectively). Since the strain softening model is rather steeply increasing as a function of displacement at lower stress levels, the use of that model could be unconservative for moderate earthquakes in that there is not enough load to get past the steeper portion of the P-Y curve. They also found that the third approach (i.e., assume the liquefied soil has no shear strength), was overly conservative. The concave, or strain hardening, shape most accurately modeled the observed behavior of the piles tested in liquefied conditions (Weaver, et al. 2005; Rollins, et al. 2005a).

Rollins, et al. (2005) also concluded that group reduction factors for lateral pile resistance can be neglected in fully liquefied sand (i.e., Rₘ > 0.9), and that group reduction effects reestablish quickly as pore pressures dissipate. Furthermore, they observed that group reduction factors were applicable in soil that is not fully liquefied.

Therefore, the expressions developed by Rollins, et al. (2005a, 2005b) and contained within LPile (Isenhower and Wang 2015) should be used to develop liquefied soil P-Y curves.
In general, if the liquefied P-Y curves result in foundation lateral deformations that are less than approximately 2 inches near the foundation top for the liquefied state, the liquefied P-Y curves should be further evaluated to make sure the parameters selected to create the liquefied P-Y curves represent realistic behavior in liquefied soil.

For pile or shaft groups, for fully liquefied conditions, P-Y curve reduction factors to account for foundation element spacing and location within the group may be set at 1.0. For partially liquefied conditions, the group reduction factors shall be consistent with the group reduction factors used for static loading.

6-5.2 Earthquake Induced Earth Pressures on Retaining Structures

The procedures specified in the AASHTO LRFD Bridge Design Specifications shall be used to determine earth pressures acting on retaining walls during a seismic event. Due to the high rate of loading that occurs during seismic loading, the use of undrained strength parameters in the slope stability analysis may be considered for soils other than clean coarse grained sands and gravels and sensitive silts and clays that could weaken during shaking.

6-5.3 Earthquake Induced Slope Failure Loads on Structures

If the pseudo-static slope stability analysis conducted in accordance with Section 6-4.3.2 results in a safety factor of less than 1.1 (or a resistance factor that is greater than 0.9 for LRFD), the slope shall be stabilized or the structure shall be designed to resist the slide force. For earthquake induced slope failure loads applied to structure foundations and bridge abutments, the lateral force applied to the structure is the force needed to restore the slope level of safety to the required minimum value. But this assumes that the structure and its foundations can be designed to resist the slide loading and the deformation required to mobilize the necessary resistance. If the structural designer
determines that the structure cannot resist the slide load and the deformation it causes, then the slope shall be stabilized to restore its level of safety to the required minimum values (i.e., FS > 1.1 or a resistance factor of 0.9 or less). See Section 8-6.5.2 for procedures to estimate the slide force on a foundation element.

Landslides and slope instability induced by seismic loading not induced by liquefaction should be considered to be concurrent with the structure seismic loading. Therefore, the structure seismic loads and the seismically induced landslide/slope instability forces should be coupled. Also note that when foundation elements are located within a mass that becomes unstable during seismic loading, the potential for soil below the foundation to move away from the foundation, thereby reducing its lateral support, shall be considered.

### 6-5.4 Lateral Spread and Flow Failure Loads on Structures Due to Liquefaction

Short of doing a rigorous dynamic stress-deformation analysis, there are two different approaches to estimate the lateral spread/flow failure induced load on deep foundations systems—displacement based approach and a force based approach. Displacement based approaches are more prevalent in the United States. A force based approach has been specified in the Japanese codes and is based on case histories from past earthquakes, especially the pile foundation failures observed during the 1995 Kobe earthquake. Overviews of both approaches are presented below.

#### 6-5.4.1 Displacement Based Approach


Additional background on the Caltrans procedure is provided in Ashford, et al. (2011). This procedure provides methods to evaluate deep foundation systems that partially restrain the ground movement caused by lateral spreading/flow failure (restrained case), and those foundation systems in which the ground can freely flow around them (unrestrained case). In general, the restrained case is used for bridge abutments, and the unrestrained case is used for interior bridge piers. However, to make a final determination, the spacing of the foundation elements, their stiffness as well as the stiffness of the superstructure, and the overall geometry of the structure may need to be considered.

To be consistent with the design provisions in this GDM, the Caltrans procedure shall be modified as follows:

- Assessment of liquefaction potential shall be in accordance with Section 6-4.2.2.
- Determination of liquefied residual strengths shall be in accordance with Section 6-4.2.5.
• Lateral spread deformations shall be estimated using methods provided in Section 6-4.3.1.
• The combination of seismic inertial loading and kinematic loading from lateral spreading or flow failure shall be in accordance with Section 6-4.2.7.
• Deep foundation springs shall be determined using Section 6-5.1.2.

6-5.4.2 Force Based Approaches

A force based approach to assess lateral spreading induced loads on deep foundations is specified in the Japanese codes. The method is based on back-calculations from pile foundation failures caused by lateral spreading (see Yokoyama, et al., 1997 for background on this method) The pressures on pile foundations are simply specified as follows:

• The liquefied soil exerts a pressure equal to 30 percent of the total overburden pressure (lateral earth pressure coefficient of 0.30 applied to the total vertical stress).
• The nonliquefied “crust” above the liquefied layer that moves with the liquefied layer is equal to the passive pressure of the nonliquefied layer soil moving against the foundation as later flow occurs.
• In both cases, the width of the pressure acting on the foundations is applied to the full foundation group width supporting the bridge pier. However, nothing was discussed in Yokoyama, et al. (1997) regarding the maximum center-center spacing of foundation elements that would result in the force being based in the full foundation group width. For a single foundation element supporting a bridge pier (e.g., a caisson or large diameter shaft), the width over which this lateral pressure is applied may be assumed to be the foundation width.
• Non-liquefied crustal layers exert full passive pressure on the foundation system.

Data from simulated earthquake loading of model piles in liquefiable sands in centrifuge tests indicate that the Japanese Force Method is an adequate design method (Finn, et al., 2004) and therefore may be used to estimate lateral spreading and flow failure forces on bridge foundations.

6-5.4.3 Dynamic Stress-Deformation Approaches

Seismically induced slope deformations and their effect on foundations can be estimated through a variety of dynamic stress-deformation computer models such as PLAXIS, DYNAFLOW, FLAC, and OpenSees. These methods can account for varying geometry, soil behavior, and pore pressure response during seismic loading and the impact of these deformations on foundation loading. The accuracy of these models is highly dependent upon the quality of the input parameters and the level of model validation performed by the user for similar applications.

In general, dynamic stress deformation models should not be used for routine design due to their complexity, and due to the sensitivity of deformation estimates to the constitutive model selected and the accuracy of the input parameters. If dynamic stress deformation models are used, they should be validated for the particular application. Dynamic stress-deformation models shall not be used for design on WSDOT projects without the approval of the State Geotechnical Engineer. Furthermore, independent peer review as specified in Section 6-3 shall be conducted.
6-5.5 Downdrag Loads on Structures Due to Liquefaction

Downdrag loads on foundations shall be determined in accordance with Article 3.11.8 of the AASHTO LRFD Bridge Design Specifications, GDM Chapter 8, and as specified herein.

The AASHTO LRFD Bridge Design Specifications, Article 3.11.8, recommend the use of the nonliquefied skin friction in the layers above the liquefied zone that do not liquefy but will settle, and a skin friction value as low as the residual strength within the soil layers that do liquefy, to calculate downdrag loads for the extreme event limit state. In general, vertical settlement and downdrag cannot occur until the pore pressures generated by the earthquake ground motion begin to dissipate after the earthquake shaking ceases. At this point, the liquefied soil strength will be near its minimum residual strength. At some point after the pore pressures begin to dissipate, and after some liquefaction settlement has already occurred, the soil strength will begin to increase from its minimum residual value. Therefore, the actual shear strength of soil along the sides of the foundation elements in the liquefied zone(s) may be higher than the residual shear strength corresponding to fully liquefied conditions, but still significantly lower than the nonliquefied soil shear strength. Very little guidance on the selection of soil shear strength to calculate downdrag loads due to liquefaction is available; therefore some engineering judgment may be required to select a soil strength to calculate downdrag loads due to liquefaction.

The neutral plane theory approach to assessing downdrag due to liquefaction may also be used, subject to the approval of the WSDOT State Geotechnical Engineer. See Muhunthan et al. (2017) for guidance.

6-5.6 Mitigation Alternatives

The two basic options to mitigate the lateral spread induced loads on the foundation system are to design the structure to accommodate the loads or improve the ground such that the hazard does not occur.

**Structural Options (design to accommodate imposed loads)** – See Sections 6-5.4.1 (displacement based approach) and 6-5.4.2 (force based approach) for more details on the specific analysis procedures. Once the forces and/or displacements caused by the lateral spreading have been estimated, the structural designer should use those estimates to analyze the effect of those forces and/or displacements will have on the structure to determine if designing the structure to tolerate the deformation and/or lateral loading is structurally feasible and economical.

**Ground Improvement** – It is often cost prohibitive to design the bridge foundation system to resist the loads and displacements imposed by liquefaction induced lateral loads, especially if the depth of liquefaction extends more than about 20 feet below the ground surface and if a non-liquefied crust is part of the failure mass. Ground improvement to mitigate the liquefaction hazard is the likely alternative if it is not practical to design the foundation system to accommodate the lateral loads.

The primary ground improvement techniques to mitigate liquefaction fall into three general categories, namely densification, altering the soil composition, and enhanced drainage. A general discussion regarding these ground improvement approaches is provided below. Chapter 11, Ground Improvement, should be reviewed for a more detailed discussion regarding the use of these techniques.
Densification and Reinforcement – Ground improvement by densification consists of sufficiently compacting the soil such that it is no longer susceptible to liquefaction during a design seismic event. Densification techniques include vibro-compaction, vibro-flotation, vibro-replacement (stone columns), deep dynamic compaction, blasting, and compaction grouting. Vibro-replacement and compaction grouting also reinforce the soil by creating columns of stone and grout, respectively. The primary parameters for selection include grain size distribution of the soils being improved, depth to groundwater, depth of improvement required, proximity to settlement/vibration sensitive infrastructure, and access constraints.

For those soils in which densification techniques may not be fully effective to densify the soil adequately to prevent liquefaction, the reinforcement aspect of those methods may still be used when estimating composite shear strength and settlement characteristics of the improved soil volume. See Chapter 11 for details and references that should be consulted for guidance in establishing composite properties for the improved soil volume.

If the soil is reinforced with vertical structural inclusions (e.g., drilled shafts, driven piles, but not including the structure foundation elements) but not adequately densified to prevent the soil from liquefying, the design of the ground improvement method should consider both the shear and moment resistance of the reinforcement elements. For vertical inclusions that are typically not intended to have significant bending resistance (e.g., stone columns, compaction grout columns, etc.), the requirement to resist the potential bending stresses caused by lateral ground movement may be waived, considering only shear resistance of the improved soil plus inclusions, if all three of the following conditions are met:

- The width and depth of the improved soil volume are equal to or greater than the requirements provided in Figure 6-11,
- three or more rows of reinforcement elements to resist the forces contributing to slope failure or lateral spreading are used, and
- the reinforcement elements are spaced center-to-center at less than 5 times the reinforcement element diameter or 10 feet, whichever is less.

The effect of any lateral or vertical deformation of the vertical inclusions on the structure the improved ground supports shall be taken into account in the design of the supported structure.

Figure 6-11 shows the improved soil volume as centered around the wall base or foundation. However, it is acceptable to shift the soil improvement volume to work around site constraints, provided that the edge of the improved soil volume is located at least 5 feet outside of the wall or foundation being protected. Greater than 5 feet may be needed to insure stability of the foundation, prevent severe differential settlement due to the liquefaction, and to account for any pore pressure redistribution that may occur during or after liquefaction initiation.

For the case where a “collar” of improved soil is placed outside and around the foundation, bridge abutment or other structure to be protected from the instability that liquefaction can cause, assume “B” in Figure 6-11 is equal to zero (i.e., the minimum width of improved ground is equal to D + 15 feet, but no greater than “Z”).
If the soil is of the type that can be densified through the use of stone columns, compaction grout columns, or some other means to improve the soil such that it is no longer susceptible to liquefaction within the improved soil volume, Figure 6-11 should also be used to establish the minimum dimensions of the improved soil.

If it is desired to use dimensions of the ground improvement that are less than the minimums illustrated in Figure 6-11, more sophisticated analyses to determine the effect of using reduced ground improvement dimensions should be conducted (e.g., effective stress two dimensional analyses such as FLAC). The objectives of these analyses include prevention of soil shear failure and excessive differential settlement during liquefaction. The amount of differential settlement allowable for this limit state will depend on the tolerance of the structure being protected to such movement without collapse. Use of smaller ground improvement area dimensions shall be approved of the WSDOT State Geotechnical Engineer and shall be independently peer reviewed in accordance with Section 6-3.

Another reinforcement technique that may be used to mitigate the instability caused by liquefaction is the use of geosynthetic reinforcement as a base reinforcement layer. In this case, the reinforcement is designed as described in Chapter 9, but the liquefied shear strength is used to conduct the embankment base reinforcement design.

**Figure 6-11** Minimum Dimensions for Soil Improvement Volume Below Foundations and Walls
Altering Soil Composition – Altering the composition of the soil typically refers to changing the soil matrix so that it is no longer susceptible to liquefaction. Example ground improvement techniques include permeation grouting (either chemical or micro-fine cement), jet grouting, and deep soil mixing. These types of ground improvement are typically more costly than the densification/reinforcement techniques, but may be the most effective techniques if access is limited, construction induced vibrations must be kept to a minimum, and/or the improved ground has secondary functions, such as a seepage barrier or shoring wall.

Drainage Enhancements – By improving the drainage properties of soils susceptible to liquefaction, it may be possible to prevent the build-up of excess pore water pressures, and thus liquefaction. However, drainage improvement is not considered adequately reliable by WSDOT to prevent excess pore water pressure buildup due to liquefaction for the following reasons:

- The drainage path time for pore pressure to dissipate may be too long,
- There is a potential for drainage structures to become clogged during installation and in service, and
- With drainage enhancements some settlement is still likely.

Therefore, drainage enhancements shall not be used as a means to mitigate liquefaction. However, drainage enhancements may provide some potential benefits with densification and reinforcement techniques such as stone columns.

6-6 References


6-7 Appendices

Appendix 6-A Site Specific Seismic Hazard and Site Response
Appendix 6-B High Resolution Seismic Acceleration Maps
Appendix 6-A  Site Specific Seismic Hazard and Site Response

Site specific seismic hazard and response analyses shall be conducted in accordance with Section 6-3 and the AASHTO Guide Specifications for LRFD Seismic Bridge Design. When site specific hazard characterization is conducted, it shall be conducted using the design hazard levels specified in Section 6-3.1.

6-A.1  Background Information for Performing Site Specific Analysis

Washington State is located in a seismically active region. The seismicity varies throughout the state, with the seismic hazard generally more severe in Western Washington and less severe in Eastern Washington. Earthquakes as large as magnitude 8 to 9 are considered possible off the coast of Washington State. The regional tectonic and geologic conditions in Washington State combine to create a unique seismic setting, where some earthquakes occur on faults, but more commonly historic earthquakes have been associated with large broad fault zones located deep beneath the earth’s surface. The potential for surface faulting exists, and as discussed in this appendix a number of surface faults have been identified as being potential sources of seismic ground shaking; however, surface vegetation and terrain have made it particularly difficult to locate surface faults. In view of this complexity, a clear understanding of the regional tectonic setting and the recognized seismic source zones is essential for characterizing the seismic hazard at a specific site in Washington State.

6-A.1.1  Regional Tectonics

Washington State is located at the convergent continental boundary known as the Cascadia Subduction Zone (CSZ). The CSZ lies at the boundary between two crustal tectonic plates, where the offshore Juan de Fuca plate moves northeastward, converging with and subducting beneath the continental North American plate. The CSZ extends from mid-Vancouver Island to Northern California. The interaction of these two plates results in three potential seismic source zones as depicted on Figure 6-A-1. These three seismic source zones are: (1) the shallow crustal source zone, (2) the deep CSZ Benioff or intraplate source zone, and (3) the CSZ interplate or interface source zone (i.e., the Cascadia Subduction Zone).
6-A.1.2 **Seismic Source Zones**

If conducting a site specific hazard characterization, as a minimum, the following source zones should be evaluated (all reported magnitudes are moment magnitudes):

**Shallow Crustal Source Zone** – The shallow crustal source zone is used to characterize shallow crustal earthquake activity within the North American Plate throughout Washington State. Shallow crustal earthquakes typically occur at depths ranging up to 12 miles. The shallow crustal source zone is characterized as being capable of generating earthquakes up to about magnitude 7.5. Large shallow crustal earthquakes are typically followed by a sequence of aftershocks.

Crustal seismicity is generally characterized using two types of models: known fault source models (such as the Seattle Fault zone, South Whidbey Island fault system, and the Tacoma fault), and seismicity-based background sources (which are based on historical data from earthquakes on unidentified or uncharacterized faults).
The largest known earthquakes associated with the shallow crustal source zone in Washington State include an event on the Seattle Fault about 900 AD and the 1872 North Cascades earthquake. The Seattle Fault event was believed to have been magnitude 7 or greater (Johnson, 1999), and the 1872 North Cascades earthquake is estimated to have been between magnitudes 6.8 and 7.4. The location of the 1872 North Cascades earthquake is uncertain; however, recent research suggests the earthquake's intensity center was near the south end of Lake Chelan (Bakun et al, 2002). Other large, notable shallow earthquakes in and around the state include the 1936 Milton-Freewater, Oregon earthquake (magnitude 6.1) and the North Idaho earthquake (magnitude 5.5) (Goter, 1994).

**Benioff Source Zone** – CSZ Benioff source zone earthquakes are also referred to as intraplate, intraslab, or deep subcrustal earthquakes. Benioff zone earthquakes occur within the subducting Juan de Fuca Plate between depths of 20 and 40 miles and typically have no large aftershocks. Extensive faulting results as the Juan de Fuca Plate is forced below the North American plate and into the upper mantle. Benioff zone earthquakes primarily contribute to the seismic hazard within Western Washington.

The Olympia 1949 (M = 7.1), the Seattle 1965 (M = 6.5), and the Nisqually 2001 (M = 6.8) earthquakes are considered to be Benioff zone earthquakes. The Benioff zone is characterized as being capable of generating earthquakes up to magnitude 7.5. The recurrence interval for large earthquakes originating from the Benioff source zone is believed to be shorter than for the shallow crustal and CSZ interplate source zones—anecdotally, Benioff zone earthquakes in Western Washington occur every 15 to 35 years or so, based on recent history. The deep focal depth of these earthquakes tends to dampen the shaking intensity when compared to shallow crustal earthquakes of similar magnitudes.

**CSZ Interplate Source Zone** – The Cascadia Subduction Zone (CSZ) is an approximately 650-mile long thrust fault that extends along the Pacific Coast from mid-Vancouver Island to Northern California. CSZ interplate earthquakes result from rupture of all or a portion of the convergent boundary between the subducting Juan de Fuca plate and the overriding North American plate. The fault surfaces approximately 50 to 75 miles off the Washington coast. The width of the seismogenic portion of the CSZ interplate fault is approximately 50 to 60 miles wide and varies along its length. As the fault becomes deeper, materials being faulted become ductile and the fault is unable to store mechanical stresses.

The CSZ is considered as being capable of generating earthquakes of magnitude 8 to magnitude 9. No earthquakes on the CSZ have been instrumentally recorded; however, through the geologic record and historical records of tsunamis in Japan, it is believed that the most recent CSZ event occurred in the year 1700 (Atwater, 1996 and Satake, et al, 1996). Recurrence intervals for CSZ interplate earthquakes are thought to be on the order of 400 to 600 years. Paleogeologic evidence suggests five to seven interplate earthquakes may have been generated along the CSZ over the last 3,500 years at irregular intervals.
6-A.2 Design Earthquake Magnitude

In addition to identifying the site's source zones, the design earthquake(s) produced by the source zones must be characterized for use in evaluating seismic geologic hazards such as liquefaction and lateral spreading. Typically, design earthquake(s) are defined by a specific magnitude, source-to-site distance, and ground motion characteristics.

The following guidelines should be used for determining a site's design earthquake(s):

- The design earthquake should consider hazard-compatible events occurring on crustal and subduction-related sources.
- More than one design earthquake may be appropriate depending upon the source zones that contribute to the site's seismic hazard and the impact that these earthquakes may have on site response.
- The design earthquake should be consistent with the design hazard level prescribed in Section 6-3.1.

The USGS interactive deaggregation tool (https://earthquake.usgs.gov/hazards/interactive/) provides a summary of contribution to seismic hazard for earthquakes of various magnitudes and source to site distances for a given hazard level and may be used to evaluate relative contribution to ground motion from seismic sources. Since this chapter has been updated to require the use of the 2014 maps and associated data, it is required to use the 2014 deaggregation data. Note that magnitudes presented in the deaggregation data represent contribution to a specified hazard level and should not simply be averaged for input into analyses such as liquefaction and lateral spreading. Instead, the deaggregation data should be used to assess the relative contribution to the probabilistic hazard from the various source zones. If any source zone contributes more than about 10 percent of the total hazard, design earthquakes representative from each of those source zones should be used for analyses.

For liquefaction or lateral spreading analysis, one of the following approaches should be used to account for the earthquake magnitude, in order of preference:

- Use all earthquake magnitudes applicable at the specific site (from the deaggregation) using the multiple scenario or performance based approaches for liquefaction assessment as described by Kramer and Mayfield (2007) and Kramer (2007). The hazard level used for this analysis shall be consistent with the hazard level selected for the structure for which the liquefaction analysis is being conducted (typically, a probability of exceedance of 7 percent in 75 years in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design). While performance based techniques can be accomplished using the WSLIQ software (Kramer, 2007), the performance based option in that software uses the 2002 USGS ground motions and has not been updated to include more recent ground motion data that would be consistent with the ground motions used to produce the 2014 USGS seismic maps. Until that software is updated to use the new ground motion database, the performance based option in WSLIQ shall not be used.
If a single or a few larger magnitude earthquakes dominate the deaggregation, the magnitude of the single dominant earthquake or the weighted mean of the few dominant earthquakes in the deaggregation (weighted by the percent contribution of each source) should be used.

For routine design, a default moment magnitude of 7.0 should be used for western Washington and 6.0 for eastern Washington, except within 30 miles of the coast where Cascadia Subduction zone events contribute significantly to the seismic hazard. In that case, the geotechnical designer should use a moment magnitude of 8.0. These default magnitudes should not be used if they represent a smaller hazard than shown in the deaggregation data. Note that these default magnitudes are intended for use in simplified empirically based liquefaction and lateral spreading analysis only and should not be used for development of the design ground motion parameters.

### 6-A.3 Probabilistic and Deterministic Seismic Hazard Analyses

Probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) can be completed to characterize the seismic hazard at a site. A DSHA consists of evaluating the seismic hazard at a site for an earthquake of a specific magnitude occurring at a specific location. A PSHA consists of completing numerous deterministic seismic hazard analyses for all feasible combinations of earthquake magnitude and source to site distance for each earthquake source zone. The result of a PSHA is a relationship of the mean annual rate of exceedance of the ground motion parameter of interest with each potential seismic source considered. Since the PSHA provides information on the aggregate risk from each potential source zone, it is more useful in characterizing the seismic hazard at a site if numerous potential sources could impact the site. The USGS 2014 probabilistic hazard maps on the USGS website are based on PSHA.

PSHAs and DSHAs may be required where the site is located close to a fault, long-duration ground motion is expected, or if the importance of the bridge is such that a longer exposure period is required by WSDOT. For a more detailed description and guidelines for development of PSHAs and DSHAs, see Kramer (1996), McGuire (2004), and Baker (2013).

Site specific hazard analysis should include consideration of topographic and basin effects, fault directivity and near field effects.

At a minimum, seismic hazard analysis should consider the following sources:

- Cascadia subduction zone interplate (interface) earthquake
- Cascadia subduction zone intraplate (Benioff) earthquake
- Crustal earthquakes associated with non-specific or diffuse sources (potential sources follow). These sources will account for differing tectonic and seismic provinces and include seismic zones associated with Cascade volcanism
- Earthquakes on known and potentially active crustal faults. The best source of fault information that can be considered for design is the USGS at the following website: https://earthquake.usgs.gov/hazards/qfaults
When PSHA or DSHA are performed for a site, the following information shall be included as a minimum in project documentation and reports:

Overview of seismic sources considered in analysis

Summary of seismic source parameters including length/boundaries, source type, slip rate, segmentation, maximum magnitude, recurrence models and relationships used, source depth and geometry. This summary should include the rationale behind selection of source parameters.

Assumptions underlying the analysis should be summarized in either a table (DSHA) or in a logic tree (PSHA)

The 2014 USGS probabilistic hazard maps as published herein essentially account for regional seismicity and attenuation relationships, recurrence rates, maximum magnitude of events on known faults or source zones, and the location of the site with respect to the faults or source zones. The USGS data is sufficient for most sites, and more sophisticated seismic hazard analyses are generally not required; the exceptions may be to capture the effects of sources not included in the USGS model, to assess near field or directivity influences, or to incorporate topographic impacts or basin effects.

The 2014 USGS hazard maps only capture the effects of near-fault motions (i.e., ground motion directivity or pulse effects) or bedrock topography (i.e., so called basin effects) in a limited manner. These effects modify ground motions, particularly at certain periods, for sites located near active faults (typically with 6 miles) or for sites where significant changes in bedrock topography occurs. For specific requirements regarding near fault effects, see the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

6-A.4 Selection of Attenuation Relationships

Attenuation relationships describe the decay of earthquake energy as it travels from the seismic source to the project site. Many of the newer published relationships are capable of accommodating site soil conditions as well as varying source parameters (e.g., fault type, location relative to the fault, near-field effects, etc.) In addition, during the past 10 years, specific attenuation relationships have been developed for Cascadia subduction zone sources. For both deterministic and probabilistic hazard assessments, attenuation relationships used in analysis should be selected based on applicability to both the site conditions and the type of seismic source under consideration. Rationale for the selection of and assumptions underlying the use of attenuation relationships for hazard characterization shall be clearly documented.

If deterministic methods are used to develop design spectra, the spectral ordinates should be developed using a range of ground motion attenuation relationships consistent with the source mechanisms. At least three to four attenuation relationships should be used.
6-A.5 Site Specific Ground Response Analysis

6-A.5.1 Design/Computer Models

Site specific ground response analyses are most commonly done using one-dimensional equivalent-linear or non linear procedures. A one dimensional analysis is generally based on the assumption that soils and ground surface are laterally uniform and horizontal and that ground surface motions can be modeled by vertically propagating shear wave through laterally uniform soils. The influence of vertical motions, surface waves, laterally non-uniform soil conditions, incoherence and spatial variation of ground motions are not accounted for in conventional, one-dimensional analyses (Kavazanjian, et al., 2011). A variety of site response computer models are available to geotechnical designers for dynamic site response analyses. In general, there are three classes of dynamic ground response models: 1) one dimensional equivalent linear, 2) one dimensional nonlinear, and 3) multi-dimension models. See Matasović and Hashash (2012) for a good overview of the types of models available for site specific ground response analysis, their advantages, and their limitations.

One-Dimensional Equivalent Linear Models – One-dimensional equivalent linear site response computer codes, such as ProShake (EduPro Civil Systems, 1999) or Shake2000 (Ordoñez, 2000), and DEEPSOIL (Hashash, et al. 2016) use an iterative total stress approach to estimate the nonlinear, inelastic behavior of soils. These programs use an average shear modulus and material damping over the entire cycle of loading to approximate the hysteresis loop.

The equivalent linear model provides reasonable results for small strains (less than about 1 to 2 percent) (Kramer and Paulsen, 2004). A-priori thresholds to evaluate differences between analyses and determine if a nonlinear analysis is needed (or if an equivalent linear analysis is acceptable) are provided in Kim et al. (2016). Additional information on the use and comparison of equivalent linear and nonlinear models is provided in Kaklamanos, et al. (2013, 2015), and Kim and Hashash (2013).

One-Dimensional Nonlinear Models – One-dimensional, nonlinear computer codes, such as D-MOD 2000, DESRA, and DEEPSOIL use direct numerical integration of the incremental equation of motion in small time steps and account for the nonlinear soil behavior through use of constitutive soil models. Depending upon the constitutive model used, these programs can model pore water pressure buildup and permanent deformations. The accuracy of nonlinear models depends on the proper selection of parameters used by constitutive soil model and the ability of the constitutive model to represent the response of the soil to ground shaking.

Another issue that can affect the accuracy of the model is how the $G/G_{max}$ and damping relations are modeled and the ability of the design model to adapt those relations to site specific data. Additionally, the proper selection of a Rayleigh damping value can have a significant effect on the modeling results. In general, a value of 1 to 2% is needed to maintain numerical stability. It should be recognized that the Rayleigh damping will act in addition to hysteretic damping produced by the nonlinear, inelastic soil model. Rayleigh damping should therefore be limited to the smallest value that provides the required numerical stability. The results of analyses using values greater than 1 to 2% should be interpreted with great caution. Additional information regarding Rayleigh damping as well as newer damping models is provided in Kwok, et al. (2007), and Phillips and Hashash (2009).
See Section 6-4.2.2 for specific issues related to liquefaction modeling when using one-dimensional nonlinear analysis methods.

**Two and Three Dimensional Models** – Two- and three-dimensional site response analyses can be performed using computer codes, such as QUAD4, PLAXIS, FLAC, DYNAFLOW, LSDYNA, and OPENSEES, and use both equivalent linear and nonlinear models. Many attributes of the two- and three-dimensional models are similar to those described above for the one-dimensional equivalent linear and nonlinear models. However, the two- and three-dimensional computer codes typically require significantly more model development and computational time than one-dimensional analyses. The important advantages of the two- and three-dimensional models include the ability to consider soil anisotropy, irregular soil stratigraphy, surface waves, irregular topography, and soil-structure interaction. Another advantage with the two- and three-dimensional models is that seismically induced permanent displacements can be estimated. Furthermore, these modeling platforms are better equipped for nonlinear effective stress analysis for liquefiable sites and can incorporate models that can capture large strain dilation (e.g., UBCSand). Successful application of these codes requires considerable knowledge and experience. Expert peer review of the analysis shall be conducted, in accordance with Section 6-3 unless approval to not conduct the peer review is obtained from the State Geotechnical Engineer.

6-A.5.2 **Input Parameters for Site Specific Response Analysis**

The input parameters required for both equivalent-linear and nonlinear site specific ground response analysis include the site stratigraphy (including soil layering and depth to rock or rock-like material), dynamic properties for each stratigraphic layer (including soil and rock stiffness, e.g., shear wave velocity), and ground motion time histories. Soil and rock parameters required by the equivalent linear models include the shear wave velocity or initial (small strain) shear modulus and unit weight for each layer, and curves relating the shear modulus and damping ratio as a function of shear strain (See Section 6-2.2).

The parameters required for cyclic nonlinear soil models generally consist of a backbone curve that models the stress strain path during cyclic loading and rules for loading and unloading, stiffness degradation, pore pressure generation and other factors (Kramer, 1996). More sophisticated nonlinear soil constitutive models may require definition of yield surfaces, hardening functions, and flow rules. Many of these models require specification of multiple parameters whose determination may require a significant laboratory testing program.

One of the most critical aspects of the input to a site-specific response analysis is the soil and rock stiffness and impedance values or shear wave velocity profile. Great care should be taken in establishing the shear wave velocity profile – it should be measured whenever possible. Equal care should be taken in developing soil models, including shear wave velocity profiles, to adequately model the potential range and variability in ground motions at the site and adequately account for these in the site specific design parameters (e.g., spectra). A long bridge, for example, may cross materials of significantly different stiffness (i.e., velocities) and/or soil profiles beneath the various bridge piers and abutments. Because different soil profiles can respond differently, and sometimes (particularly when very soft and/or liquefiable soils are present) very differently, great care should be taken in selecting and averaging soil profiles and properties prior to performing the site response analyses. In most cases, it is preferable to analyze the individual profiles
and then aggregate the responses rather than to average the soil properties or profiles and analyze only the averaged profile.

A suite of ground motion time histories is required for both equivalent linear and nonlinear site response analyses as described in Section 6-A.6. The use of at least three input ground motions is required and seven or more is preferred for site specific ground response analysis (total, regardless of the number of source zones that need to be considered. Guidelines for selection and development of ground motion time histories are also described in Section 6-A.6.

6-A.6 Analysis Using Acceleration-Time Histories

The site specific analyses discussed in Section 6-3 and in this appendix are focused on the development of site specific design spectra and use in other geotechnical analyses. However, site specific time histories may be required as input in nonlinear structural analysis.

Time history development and analysis for site-specific ground response or other analyses shall be conducted as specified in the AASHTO Guide Specifications for LRFD Seismic Bridge Design. For convenience, Article 3.4.4 and commentary of the AASHTO Guide Specifications are provided below:

Earthquake acceleration time histories will be required for site-specific ground motion response evaluations and for nonlinear inelastic dynamic analysis of bridge structures. The time histories for these applications shall have characteristics that are representative of the seismic environment of the site and the local site conditions, including the response spectrum for the site.

Response-spectrum-compatible time histories shall be developed from representative recorded earthquake motions. Analytical techniques used for spectrum matching shall be demonstrated to be capable of achieving seismologically realistic time series that are similar to the time series of the initial time histories selected for spectrum matching. The recorded time histories should be scaled to the approximate level of the design response spectrum in the period range of significance unless otherwise approved by the Owner. At least three response-spectrum-compatible time histories shall be used for representing the design earthquake (ground motions having 7 percent probability of exceedance in 75 years) when conducting dynamic ground motion response analyses or nonlinear inelastic modeling of bridges.

- For site-specific ground motion response modeling single components of separate records shall be used in the response analysis. The target spectrum used to develop the time histories is defined at the base of the soil column. The target spectrum is obtained from the USGS/AASHTO Seismic Hazard Maps or from a site-specific hazard analysis as described in Article 3.4.3.1.

- For nonlinear time history modeling of bridge structures, the target spectrum is usually located at or close to the ground surface, i.e., the rock spectrum has been modified for local site effects. Each component of motion shall be modeled. The issue of requiring all three orthogonal components (x, y, and z) of design motion to be input simultaneously shall be considered as a requirement when conducting a nonlinear time-history analysis. The design actions shall be taken as the maximum response calculated for the three ground motions in each principal direction.
If a minimum of seven time histories are used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction. For near-field sites (D < 6 miles) the recorded horizontal components of motion selected should represent a near-field condition and that they should be transformed into principal components before making them response-spectrum-compatible. The major principal component should then be used to represent motion in the fault-normal direction and the minor principal component should be used to represent motion in the fault-parallel direction.

Characteristics of the seismic environment of the site to be considered in selecting time-histories include: tectonic environment (e.g., subduction zone; shallow crustal faults in western United States or similar crustal environment; eastern United States or similar crustal environment); earthquake magnitude; type of faulting (e.g., strike-slip; reverse; normal); seismic-source-to-site distance; basin effects, local site conditions; and design or expected ground-motion characteristics (e.g., design response spectrum; duration of strong shaking; and special ground-motion characteristics such as near-fault characteristics). Dominant earthquake magnitudes and distances, which contribute principally to the probabilistic design response spectra at a site, as determined from national ground motion maps, can be obtained from deaggregation information on the U.S. Geological Survey website: https://earthquake.usgs.gov/hazards/interactive.

It is desirable to select time-histories that have been recorded under conditions similar to the seismic conditions at the site listed above, but compromises are usually required because of the multiple attributes of the seismic environment and the limited data bank of recorded time-histories. Selection of time-histories having similar earthquake magnitudes and distances, within reasonable ranges, are especially important parameters because they have a strong influence on response spectral content, response spectral shape, duration of strong shaking, and near-source ground-motion characteristics. It is desirable that selected recorded motions be somewhat similar in overall ground motion level and spectral shape to the design spectrum to avoid using very large scaling factors with recorded motions and very large changes in spectral content in the spectrum-matching approach. If the site is located within 6 miles of an active fault, then intermediate-to-long-period ground-motion pulses that are characteristic of near-source time-histories should be included if these types of ground motion characteristics could significantly influence structural response. Similarly, the high short-period spectral content of near-source vertical ground motions should be considered.

Ground-motion modeling methods of strong-motion seismology are being increasingly used to supplement the recorded ground-motion database. These methods are especially useful for seismic settings for which relatively few actual strong-motion recordings are available, such as in the central and eastern United States. Through analytical simulation of the earthquake rupture and wave-propagation process, these methods can produce seismologically reasonable time series.

Response spectrum matching approaches include methods in which time series adjustments are made in the time domain (Lilhanand and Tseng, 1988; Abrahamson, 1992) and those in which the adjustments are made in the frequency domain (Gasparini and Vannmarcke, 1976; Silva and Lee, 1987; Bolt and Gregor, 1993). Both of these approaches can be used to modify existing time-histories to achieve a close match to the design response spectrum while maintaining fairly well the basic time-domain character of the recorded or simulated time-histories. To minimize changes to the time-domain
characteristics, it is desirable that the overall shape of the spectrum of the recorded time-history not be greatly different from the shape of the design response spectrum and that the time-history initially be scaled so that its spectrum is at the approximate level of the design spectrum before spectrum matching.

When developing three-component sets of time histories by simple scaling rather than spectrum matching, it is difficult to achieve a comparable aggregate match to the design spectra for each component of motion when using a single scaling factor for each time-history set. It is desirable, however, to use a single scaling factor to preserve the relationship between the components. Approaches for dealing with this scaling issue include:

- Use of a higher scaling factor to meet the minimum aggregate match requirement for one component while exceeding it for the other two,
- Use of a scaling factor to meet the aggregate match for the most critical component with the match somewhat deficient for other components, and
- Compromising on the scaling by using different factors as required for different components of a time-history set.

While the second approach is acceptable, it requires careful examination and interpretation of the results and possibly dual analyses for application of the horizontal higher horizontal component in each principal horizontal direction.

The requirements for the number of time histories to be used in nonlinear inelastic dynamic analysis and for the interpretation of the results take into account the dependence of response on the time domain character of the time histories (duration, pulse shape, pulse sequencing) in addition to their response spectral content.

Additional guidance on developing acceleration time histories for dynamic analysis may be found in publications by the Caltrans Seismic Advisory Board Adhoc Committee (CSABAC) on Soil-Foundation-Structure Interaction (1999) and the U.S. Army Corps of Engineers (2000). CSABAC (1999) also provides detailed guidance on modeling the spatial variation of ground motion between bridge piers and the conduct of seismic soil-foundation-structure interaction (SFSI) analyses. Both spatial variations of ground motion and SFSI may significantly affect bridge response. Spatial variations include differences between seismic wave arrival times at bridge piers (wave passage effect), ground motion incoherence due to seismic wave scattering, and differential site response due to different soil profiles at different bridge piers. For long bridges, all forms of spatial variations may be important. For short bridges, limited information appears to indicate that wave passage effects and incoherence are, in general, relatively unimportant in comparison to effects of differential site response (Shinozuka et al., 1999; Martin, 1998). Somerville et al. (1999) provide guidance on the characteristics of pulses of ground motion that occur in time histories in the near-fault region.

In addition to the information sources cited above, Kramer (1996), Bommer and Acevedo (2004), NEHRP (2011), and Kramer, et al. (2012), should be consulted for specific requirements on the selection, scaling, and use of time histories for ground motion characterization and dynamic analysis.
Final selection of time histories to be used will depend on two factors:

- How well the response spectrum generated from the scaled time histories matches the design response spectrum, and
- Similarity of the fault mechanisms for the time histories to those of recognized seismic source zones that contribute to the site’s seismic hazard. Also, if the earthquake records are used in the site specific ground response model as bedrock motion, the records should be recorded on sites with bedrock characteristics. The frequency content, earthquake magnitude, and peak bedrock acceleration should also be used as criteria to select earthquake time histories for use in site specific ground response analysis.

The requirements in the first bullet are most important to meet if the focus of the seismic modeling is structural and foundation design. The requirements in the second bullet are most important to meet if liquefaction and its effects are a major consideration in the design of the structure and its foundations. Especially important in the latter case is the duration of strong motion.

Note that a potential issue with the use of a spectrum-compatible motion that should be considered is that in western Washington, the uniform hazard spectrum (UHS) may have significant contributions from different sources that have major differences in magnitudes and site-to-source distances. The UHS cannot conveniently be approximated by a single earthquake source. For example, the low period (high frequency) part of the UHS spectrum may be controlled by a low-magnitude, short-distance event and the long period (low frequency) portion by a large-magnitude, long-distance event. Fitting a single motion to that target spectrum will therefore produce an unrealistically energetic motion with an unlikely duration. Using that motion as an input to an analysis involving significant amounts of nonlinearity (such as some sort of permanent deformation analysis, or the analysis of a structure with severe loading) can lead to overprediction of response (soil and/or structural). However, if the soil is overloaded by this potentially unrealistically energetic prediction of ground motion, the soil could soften excessively and dampen a lot of energy (large strains), more than would be expected in reality, leading to an unconservative prediction of demands in the structure.

To address this potential issue, time histories representing the distinctly different seismic sources (e.g., shallow crustal versus subduction zone) should be spectrally matched or scaled to correspondingly distinct, source-specific spectra. A source-specific spectrum should match the UHS or design spectrum over the period range in which the source is the most significant contributor to the ground motion hazard, but will likely be lower than the UHS or design spectrum at other periods for which the source is not the most significant contributor to the hazard. However, the different source-spectra in aggregate should envelope the UHS or design spectrum. Approval by the State Geotechnical Engineer and State Bridge Engineer is required for use of source-specific spectra and time histories.
Seismic Zones and Peak Horizontal Acceleration (%g) for 7% Probability of Exceedance in 75 years - Site Class - B/C Boundary - 1000 Year Seismic Event -

Legend
- Seismic Zones and Peak Horizontal Acceleration (%g)
  - 0.131 - 0.150
  - 0.151 - 0.180
  - 0.181 - 0.210
  - 0.211 - 0.230
  - 0.231 - 0.260
  - 0.261 - 0.280
  - 0.281 - 0.300
  - 0.301 - 0.320
  - 0.321 - 0.340
  - 0.341 - 0.360
  - 0.361 - 0.390
  - 0.391 - 0.410
  - 0.411 - 0.430
  - 0.431 - 0.460
  - 0.461 - 0.480
  - 0.481 - 0.500
  - 0.501 - 0.530
  - 0.531 - 0.560
  - 0.561 - 0.600
- Major Lakes
- Major Shorelines
- U.S. Interstate
- U.S. Highway
- State Route
Horizontal Spectral Response Acceleration of 0.2 - second Period (%g) for 7% Probability of Exceedance in 75 years - with 5% of Critical Damping
- 1000 Year Seismic Event -
Horizontal Spectral Response Acceleration of 1.0-second Period (%) for 7% Probability of Exceedance in 75 years - with 5% of Critical Damping - 1000 Year Seismic Event -

Legend

<table>
<thead>
<tr>
<th>Spectral Response Acceleration (%)</th>
<th>0.000 - 0.100</th>
<th>0.101 - 0.120</th>
<th>0.121 - 0.140</th>
<th>0.141 - 0.160</th>
<th>0.161 - 0.180</th>
<th>0.181 - 0.200</th>
<th>0.201 - 0.220</th>
<th>0.221 - 0.240</th>
<th>0.241 - 0.260</th>
<th>0.261 - 0.280</th>
<th>0.281 - 0.300</th>
<th>0.301 - 0.320</th>
<th>0.321 - 0.340</th>
<th>0.341 - 0.360</th>
<th>0.361 - 0.380</th>
<th>0.381 - 0.400</th>
<th>0.401 - 0.420</th>
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July 2019
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.
of the manufacturer's plans and details would need to be incorporated into the contract documents. Several manufacturers may need to be contacted to maintain competitive bidding. More information is available in Chapters 610 and 730 of the Design Manual M 22-01.

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

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

15-3  Required Information

15-3.1  Site Data and Permits

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

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

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

- Identify areas of concern, risk, or potential variability in subsurface conditions.
- Develop likely sequence and phases of construction as they may affect retaining wall and reinforced slope selection.
- Identify design and constructability requirements or issues such as:
  - Surcharge loads from adjacent structures
  - 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
Chapter 15 Abutments, Retaining Walls, and Reinforced Slopes

- Identify engineering analyses to be performed:
  - Bearing resistance
  - Settlement
  - Global stability
  - Internal stability

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

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

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

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

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

<table>
<thead>
<tr>
<th>Geotechnical Issues</th>
<th>Engineering Evaluations</th>
<th>Required Information for Analyses</th>
<th>Field Testing</th>
<th>Laboratory Testing</th>
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<tr>
<td>Cut Walls</td>
<td>• internal stability</td>
<td>• subsurface profile</td>
<td>• test cut to evaluate stand-up time</td>
<td>• triaxial tests</td>
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<td>• external stability</td>
<td>(soil, ground water, rock)</td>
<td>• well pumping tests</td>
<td>• unconfined compression</td>
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<td>• excavation stability</td>
<td>• shear strength of soil</td>
<td>• piezometers</td>
<td>• direct shear</td>
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<td>• global and compound</td>
<td>• horizontal earth</td>
<td>• SPT</td>
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<td>stability</td>
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<td>• Atterberg limits</td>
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<td>• chemical compatibility of wall/soil</td>
<td>(soil and reinforcement)</td>
<td>• dilatometer</td>
<td>• pH, resistivity tests</td>
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<td>• lateral earth pressure</td>
<td>• hydraulic conductivity of soil</td>
<td>• pullout tests (anchors, nails)</td>
<td>• organic content</td>
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<td>• down-drag on wall</td>
<td>• geologic mapping</td>
<td>• geophysical testing</td>
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<td>including orientation</td>
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<td>and characteristics of rock</td>
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<td>subsidence (karst,</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 as measured vertically from wall bottom to top or from slope toe to crest, as shown in Figure 15-1, shall be investigated in accordance with Sections 15.3.4.1 and 15.3.4.2. For all retaining walls and reinforced slopes greater than 10 feet in exposed height, the field exploration shall be completed in accordance with the AASHTO LRFD Bridge Design Specifications and this manual.

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

![Exposed Height (H) for a Retaining Wall or Slope](image)

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

- Geological formation(s).
- Location and thickness of soil and rock units.
- Engineering properties of soil and rock units, such as unit weight, shear strength and compressibility.
- Ground water conditions.
- Ground surface topography.
- Local considerations (e.g., liquefiable, expansive or dispersive soil deposits, underground voids from solution weathering or mining activity, or slope instability potential).
In areas underlain by heterogeneous soil deposits and/or rock formations, it will probably be necessary to perform more investigation to capture variations in soil and/or rock type and to assess consistency across the site area. In a laterally homogeneous area, drilling or advancing a large number of borings may be redundant, since each sample tested would exhibit similar engineering properties. In all cases, it is necessary to understand how the design and construction of the geotechnical feature will affect the soil and/or rock mass in order to optimize the exploration. The following minimum guidelines for frequency and depth of exploration shall be used. Additional exploration may be required depending on the variability in site conditions, wall/slope geometry, wall/slope type, and the consequences should a failure occur.

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

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

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

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

For retaining walls and slopes more than 100 feet in length, exploration points should in general be spaced at 100 to 200 feet, but may be spaced at up to 500 feet in uniform, dense soil conditions. Even closer spacing than 100 to 200 feet should be used in highly variable and potentially unstable soil conditions. Where possible, locate at least one boring where the maximum height occurs. Explorations should be completed on the alignment of the wall face or approximately midpoint along the reinforced slope, i.e., where the height is 0.5H. Additional borings to investigate the toe slope for walls or the toe catch for reinforced slopes may be required to assess overall stability issues.
A key to the establishment of exploration frequency for walls is the potential for the subsurface conditions to impact the construction of the wall, the construction contract in general, and the long-term performance of the finished project. The exploration program should be developed and conducted in a manner that these potential problems, in terms of cost, time, and performance, are reduced to an acceptable level. The boring frequency described above may need to be adjusted by the geotechnical designer to address the risk of such problems for the specific project.

15-3.4.2 Walls and Slopes Requiring Additional Exploration

15-3.4.2.1 Soil Nail Walls

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

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

15-3.4.2.2 Walls With Ground Anchors or Deadman Anchors

Walls with ground anchors or deadman anchors should have additional geotechnical borings completed to explore the soil conditions within the anchor/deadman zone. For retaining walls more than 100 feet in length, exploration points should in general be spaced at 100 to 200 feet, but may be spaced at up to 500 feet in uniform, dense soil conditions. Even closer spacing than 100 to 200 feet should be used in highly variable and potentially unstable soil conditions. The borings should be completed outside the no-load zone of the wall in the bond zone of the anchors or at the deadman locations. The depth of the borings shall be sufficient to explore the full depth of soils where ground anchors or deadman anchors are likely to be installed, and deep enough to address overall stability issues.
15-3.4.2.3 **Wall or Slopes With Steep Back Slopes or Steep Toe Slopes**

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

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

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

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

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

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

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

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

In soft soils, CPT tests or in situ 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.
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 are tied to both wall faces, the walls shall be considered unable to slide to reduce the acceleration to be applied. Therefore, the full ground acceleration shall be used in the walls in that case.

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

15-4.4 Walls on Slopes

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

All walls and abutments should meet the minimum embedment criteria in AASHTO. The final embedment depth required shall be based on geotechnical bearing and stability requirements provided in the AASHTO LRFD specifications, as determined by the geotechnical designer (see also Section 15.4.4). Walls that have a sloping ground line at the face of wall may need to have a sloping or stepped foundation to optimize the wall embedment. Sloping foundations (i.e., not stepped) shall be 6H:1V or flatter. Stepped foundations shall be 1.5H:1V or flatter determined by a line through the corners of the steps. The maximum feasible slope of stepped foundations for walls is controlled by the maximum acceptable stable slope for the soil in which the wall footing is placed. Concrete leveling pads constructed for MSE walls shall be sloped at 6H:1V or flatter or stepped at 1.5H:1V or flatter determined by a line through the corners of the steps. As MSE wall facing units are typically rectangular shapes, stepped leveling pads are preferred.

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

15-4.6 Wall Height Limitations

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

Some types of walls may have more stringent height limitations. Walls that have more stringent height limitations include full height propped precast concrete panel MSE walls (Section 15.5.3.7), 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(_{100}) ≤ 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(_{100}) ≤ 2 in</td>
<td>Ensure structure can tolerate settlement</td>
</tr>
<tr>
<td>ΔH &gt; 2.5 in</td>
<td>ΔH(_{100}) &gt; 2 in</td>
<td>Obtain Approval(^1) prior to proceeding with design and Construction</td>
</tr>
</tbody>
</table>

\(^1\)Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

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

<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(_{100}) ≤ 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(_{100}) ≤ 3 in</td>
<td>Ensure structure can tolerate settlement</td>
</tr>
<tr>
<td>ΔH &gt; 4 in</td>
<td>ΔH(_{100}) &gt; 3 in</td>
<td>Obtain Approval(^1) prior to proceeding with design and Construction</td>
</tr>
</tbody>
</table>

\(^1\)Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

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

<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(_{50}) ≤ 3 in</td>
<td>Design and Construct</td>
</tr>
<tr>
<td>4 in &lt; ΔH ≤ 12 in</td>
<td>3 in &lt; ΔH(_{50}) ≤ 9 in</td>
<td>Ensure structure can tolerate settlement</td>
</tr>
<tr>
<td>ΔH &gt; 12 in</td>
<td>ΔH(_{50}) &gt; 9 in</td>
<td>Obtain Approval(^1) prior to proceeding with design and Construction</td>
</tr>
</tbody>
</table>

\(^1\)Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

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

Note that more stringent tolerances may be necessary to meet aesthetic requirements for the walls.
15-4.8 **Active, Passive, At-Rest Earth Pressures**

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

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

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

15-4.9 **Surcharge Loads**

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

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

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

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

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

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

If fine grained soils are present behind the wall, the seismic earth pressure shall be determined accounting for the effect of earthquake shaking and displacement on the soil shear strength. For sensitive silts and clays (see also Section 6.4.3), the shear strength used to calculate the seismic earth pressure shall be reduced to account for the strength loss caused by the shaking. If over-consolidated cohesive soils (e.g., “Seattle Clays” as described in Section 5.13.3) are present behind the wall and the wall is designed to allow displacement, the residual drained friction angle rather than the peak friction angle in accordance with Chapter 5, should be used to determine the seismic lateral earth pressure. To justify a design shear strength greater than its residual value, a wall displacement analysis shall be conducted and shall demonstrate that the magnitude of the wall deflections allowed are too small to drop the shear strength to its residual value. See Chapter 5 for additional requirements regarding the shear strength issue, and Chapter 6 and the AASHTO LRFD Bridge Design Specifications for design methods and additional requirements to estimate the wall deflection.

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

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

15-4.11 Liquefaction

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

For walls that retain liquefiable soils, and for which ground improvement is not feasible or cost effective to mitigate the liquefiable soils, the Generalized Limit Equilibrium (GLE) Method should be used to estimate the seismic active earth pressure as specified in the AASHTO LRFD Bridge Design Manual, specifically Article 11.6.5.3. Two analyses are required when a wall retains soil layers that may liquefy. These two analyses include: (1) a pseudo-static wall design as specified in Section 15.4.10, and (2) an analysis in which the soil has liquefied. For sites where more than 20 percent of the hazard contributing to the peak ground acceleration is from an earthquake with a magnitude of 7.5 or more (i.e., a long duration earthquake where there is potential for strong motion to occur after liquefaction has occurred), it should be assumed that the additional earth pressure behind the wall due to liquefaction occurs simultaneously with the earthquake ground motion. In this case, \( k_h \) shall be as specified in the previous section (i.e., Section 15.4.10). For earthquakes in which the magnitude is less than 7.5, it can be assumed that \( k_h = 0 \) when the soil is liquefied.

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

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

All retaining walls and reinforced slopes shall be designed 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.
15-4.14 Utilities

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

15-4.15 Guardrail and Barrier

Guardrail and barrier shall meet the requirements of the Design Manual M 22-01, Bridge Design Manual, Standard Plans, and the AASHTO LRFD Bridge Design Specifications. In no case shall guardrail be placed through MSE wall or reinforced slope soil reinforcement closer than 3 feet from the back of the wall facing elements. Furthermore, the guardrail 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, or clays in general, the level of safety obtained in past WSDOT practice shall continue to be used (i.e., \(FS = 1.5\)). For anchors in other soils (e.g., sands, gravels, glacial tills), the level of safety obtained when applying the national practice (i.e., \(FS = 1.33\)) should be used.

The AASHTO LRFD Bridge Design Specifications specifically addresses anchor testing. The AASHTO specifications recommend that the test loads used in past allowable stress design practice be reduced by the load factor applicable to the limit state that controls the maximum factored design load for the anchor. For the strength limit state, a load factor \( \gamma_{EH} \) of 1.35 is typically applied to the lateral earth pressure acting on the wall. If the seismic design (i.e., Extreme Event I) controls the factored load acting on the anchor, then the load factor is only 1.0. However, due to the extreme nature of the loading for this limit state, the extra margin of safety used to design in the strength limit state is not needed for the seismic load case, as past allowable stress design practice used a FS of 1.0.
To be consistent with previous WSDOT practice, for the Strength Limit State, verification tests, if conducted, shall be performed to 1.5 times the factored design load (FDL) for the anchor. Proof and performance tests shall be performed to 1.15 times the factored design load (FDL) for anchors installed in clays, and to 1.00 times the factored design load (FDL) for anchors in other soils and rock. The geotechnical designer should make the decision during design as to whether or not a higher test load is required for anchors in a portion of, or all of, the wall due to the presence of clays or other problematic soils. These proof, performance, and verification test loads assume that a load factor, $\gamma_{EH}$, of 1.35 is applied to the apparent earth pressure used to design the anchored wall. If the Extreme Event I limit state controls the design, the same loading sequence and magnitude as used for the strength limit state should be used for all anchor tests.

The following shall be used for verification tests:

<table>
<thead>
<tr>
<th>Strength Limit State Controls</th>
<th>Load</th>
<th>Hold Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>AL</td>
<td>1 Min.</td>
<td></td>
</tr>
<tr>
<td>0.25FDL</td>
<td>10 Min.</td>
<td></td>
</tr>
<tr>
<td>0.50FDL</td>
<td>10 Min.</td>
<td></td>
</tr>
<tr>
<td>0.75FDL</td>
<td>10 Min.</td>
<td></td>
</tr>
<tr>
<td>1.00FDL</td>
<td>10 Min.</td>
<td></td>
</tr>
<tr>
<td>1.15FDL</td>
<td>60 Min.</td>
<td></td>
</tr>
<tr>
<td>1.25FDL</td>
<td>10 Min.</td>
<td></td>
</tr>
<tr>
<td>1.50FDL</td>
<td>10 Min.</td>
<td></td>
</tr>
<tr>
<td>AL</td>
<td>1 Min.</td>
<td></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>
<th>Load</th>
<th>Hold Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>AL</td>
<td>1 Min.</td>
<td></td>
</tr>
<tr>
<td>0.25FDL</td>
<td>1 Min.</td>
<td></td>
</tr>
<tr>
<td>0.50FDL</td>
<td>1 Min.</td>
<td></td>
</tr>
<tr>
<td>0.75FDL</td>
<td>1 Min.</td>
<td></td>
</tr>
<tr>
<td>1.00FDL</td>
<td>1 Min.</td>
<td></td>
</tr>
<tr>
<td>1.15FDL</td>
<td>10 Min.</td>
<td></td>
</tr>
<tr>
<td>AL</td>
<td>1 Min.</td>
<td></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><strong>Strength Limit State Controls</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Load</strong></td>
<td><strong>Hold Time</strong></td>
</tr>
<tr>
<td>AL</td>
<td>1 Min.</td>
</tr>
<tr>
<td>0.25FDL</td>
<td>1 Min.</td>
</tr>
<tr>
<td>0.50FDL</td>
<td>1 Min.</td>
</tr>
<tr>
<td>0.75FDL</td>
<td>1 Min.</td>
</tr>
<tr>
<td><strong>1.00FDL</strong></td>
<td><strong>10 Min.</strong></td>
</tr>
<tr>
<td>AL</td>
<td>1 Min.</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></td>
<td></td>
</tr>
<tr>
<td>0.75FDL</td>
<td>0.75FDL</td>
<td>0.75FDL</td>
<td></td>
<td>0.75FDL</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.00FDL</td>
<td>1.00FDL</td>
<td>1.15FDL</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).
Abutments, Retaining Walls, and Reinforced Slopes

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

**EFFECT OF ANCHOR LOCATION RELATIVE TO THE WALL**

- Anchor block left of b provides no resistance.
- Anchor block right of b provides full resistance with no load transferred to wall.
- Anchor block between b and d provides partial resistance and transfers load \( \Delta P \) to base of wall.

**VECTOR DIAGRAM FOR FREE BODY OF DEAD MAN WHERE \( P_a \) = ACTIVE FORCE ON BACK OF DE AT ANCHOR BLOCK.**

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

- Forces per linear foot of anchor wall
- Anchor wall left of cc
- Anchor wall right of cc

**EFFECT OF DEPTH AND SPACING OF ANCHOR BLOCKS**

1. Anchor resistance for \( h_1 \leq \frac{b}{2} \)
2. Individual anchors:
   - Ultimate \( A_{pc/d} = 1.5P_aP_b \) where \( A_{pc/d} \) is anchor resistance and \( P_a, P_b \) taken per linear foot of wall.
   - For \( b > 1.5h \), ultimate \( A_{pc/d} = 1.5P_aP_b + 2P_aP_b \tan \phi \), where \( P_C \) is resultant force of soil at rest on vertical area \( c'd'e' \) or \( c'd'e' \).
   - For this condition \( L' = h' \).
   - Anchor resistance for \( h_1 < \frac{b}{2} \)
   - Ultimate \( A_{pc/d} \) or \( A_{pc/d} \) equals bearing capacity of strip footing of width \( h_1 \) and surcharge load \( y \)

**GENERAL REQUIREMENTS:**
1. Allowable value of \( A_{pc} \) and \( A_{pc/d} \) are ultimate values divided by factor of safety of 2 against failure.
2. Values of \( K_a \) and \( K_p \) are for cohesionless materials. If backfill has both \( \phi \) and \( c \) strengths, compute active and passive forces according to Figures 7 and 9.
3. Soils within passive wedge of anchorage should not be used at the anchorage.
4. Soils within passive wedge of anchorage shall be compacted to no less than 95% of maximum unit weight (ASTM D698 test).
5. Tie rod is designed for allowable \( A_p \) or \( A_{pc} \). Tie rod connections to wall and anchorage are designed for allowable \( A_p \) or \( A_{pc} \).
6. Tie rod connection to anchorage is made at the location of the resultant earth pressures acting on the vertical face of the anchorage.
Mechanically Stabilized Earth Walls

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

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

The Stiffness Method is in general less conservative, but more accurate, than the other two methods. For this reason, the load and resistance factors provided in the current AASHTO LRFD Bridge Design Specifications (2017), which are based on levels of safety used in previous long-term design practice, are not directly applicable to the Stiffness Method, requiring that the Stiffness Method be calibrated using reliability theory to achieve the target minimum reliability (see Allen et al. 2005). Therefore, the calibrated load and resistance factors provided in Section 15.5.3.10.2 for the Stiffness Method shall be used.

Note that load and resistance factors are not provided for the Stiffness Method in Section 15.5.3.10.2 for MSE walls with steel (i.e., inextensible) reinforcement. Calibration of the Stiffness Method load and resistance factors for steel reinforced systems are still in progress and therefore are not available at the time of this update. Until that calibration work is complete, the Stiffness Method is only approved for routine use for MSE walls with extensible reinforcement. This method may be used for steel reinforced MSE walls only if the reinforcement layers are instrumented such that the reinforcement loads are measured, subject to approval by the State Geotechnical Engineer. However, the Coherent Gravity and Simplified methods, using the load and resistance factors provided in the AASHTO LRFD Bridge Design Manual, should be used for inextensible steel reinforced MSE walls, considering long-term successful design practice.

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

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

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

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

Horizontal spacings as large as 3.3 ft have been used in typical design and construction practice for MSE walls. Back-analysis of instrumented MSE walls indicates that reinforcement load prediction accuracy is not adversely compromised with horizontal spacing of this magnitude when the reinforcement elements are directly attached to rigid facings such as precast concrete panels. However, for flexible facings such as welded wire, large horizontal spacing of the reinforcement has 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 15E) 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, 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, $T_{\text{ref}}$, 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

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**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, but no less than 3 feet of reinforced soil, shall extend over the top of the existing structure or steep rock face an adequate distance to insure adequate pullout resistance. The minimum length of these upper two reinforcement layers should be 0.7H, 5 feet behind the face of the existing structure or rock face, or the minimum length required to resist the pullout forces applied to those layers, whichever results in the greatest reinforcement length. Note that to accomplish this, it may be necessary to remove some of the top of the existing structure or rock face if the existing structure is nearly the same height as the new wall. The minimum clearance between the top of the existing structure or rock face and the first reinforcement layer extended beyond the top of the existing structure should be 6 in to prevent stress concentrations.

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

• 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.

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

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

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

Figures 15-8, 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 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 15-8](image-url)
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)

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 $\pm \frac{1}{16}$ in or less for walls built as part of WSDOT projects, even though the current ASTM requirements for these types of blocks have been relaxed to $\pm \frac{1}{8}$ in), poor block placement technique, soil reinforcement placed between the blocks that create too much unevenness between
the block surfaces, some forms of shimming to make facing batter adjustments, and inconsistencies in the block concrete properties. See Figure 15-11 for illustrations of potential causes of block cracking. Another tall block faced wall problem encountered by others includes shearing of the back portion of the blocks parallel to the wall, possibly face due to excessive buildup of downdrag forces immediately behind the blocks. This problem, if it occurs, has been observed in the bottom 5 to 7 feet of walls that have a hinge height of approximately 25 to 30 feet (total height of 35 feet or more) and may have been caused by excessive downdrag forces due to backfill soil compressibility immediately behind the facing.

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

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

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

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

15-5.3.10 Internal Stability Using the Stiffness Method

The Stiffness Method, as described by Allen and Bathurst (2015, 2018), may be used as an alternative to the Simplified and Coherent Gravity methods 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.
For the Stiffness Method, $T_{\text{max}}$ is calculated as follows:

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

(15-1)

where,

- $S_v$ = tributary vertical thickness for reinforcement layer (ft)
- $H$ = height of wall (ft)
- $H_{\text{ref}}$ = reference wall height = 20 ft
- $\gamma_f$ = unit weight of soil in wall reinforcement zone (lbs/ft$^3$)
- $S$ = average soil surcharge thickness over reinforcement (ft)
- $\gamma_i$ = unit weight of soil in wall in surcharge above wall (lbs/ft$^3$)
- $D_{\text{max}}$ = $T_{\text{max}}$ distribution factor (dim)
- $k_{\text{avg}}$ = 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 $D_{\text{max}}$ is illustrated in Figure 15-12. In the figure, depths below the wall top have been normalized by the wall height, $H$. $T_{\text{max}}$ 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 $D_{\text{max}}$ 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), $\Phi$ shall 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)

and,

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

where,

- $J_{ave} = $ average secant tensile stiffness of all “n” reinforcement layers (kips/ft)
- $J_i = $ secant tensile stiffness of reinforcement layer i considering the horizontal spacing, i.e., the coverage ratio $R_c$, of the reinforcement (kips/ft)
- $n = $ number of reinforcement layers in wall section (dim)

For geogrids and geotextiles, the reinforcement stiffness 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 $\Phi_{fs}$ shall be determined as follows:

$$\Phi_{fs} = \eta \left( \frac{S_{global}}{P_a} \right)^\kappa \left( F_f \right)^\lambda$$  \hspace{1cm} (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)
\[
F_t = \frac{1.5H^3 p_s}{E b^3 (h_{\text{eff}}/H)}
\]

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 \(\Phi_g = 1\). For full height and incremental panel walls, \(h_{\text{eff}} = H\) and panel height, respectively. Since the facing stiffness factor \(\Phi_g\) 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 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 \(\Phi_g = 1.0\) in that case). The facing stiffness factor \(\Phi_g\) 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.

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. For the complete Stiffness Method, \(\Phi\) in Equation 15-4 is expanded as follows:

\[
\Phi = \Phi_g \Phi_{fs} \Phi_{fb} \Phi_{local} \Phi_c
\]

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)
Φₙ and Φₛ are determined as shown in equations 15-5 and 15-7. Φ₀ₙ shall be determined as follows:

\[
Φ₀ₙ = \left( \frac{K_{abh}}{K_{avh}} \right)^d
\]

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

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

The local stiffness factor, Φₙₗₗₗ, shall be determined as follows:

\[
Φₗₗₗ = \left( \frac{S_{ₗₗₗ}}{S_{ₗₗₗave}} \right)^a
\]

where,
\[a = \text{empirical exponent} = 0.50 \text{ for extensible reinforcement (e.g., geotextiles, geogrids, polymer straps)}\]
\[S_{ₗₗₗ} = \text{local reinforcement stiffness determined as follows:}\]
\[S_{ₗₗₗ} = J_i/S_v\]

where,
\[J_i \text{ and } S_v \text{ are as defined previously}\]

\[S_{ₗₗₗave} \text{ shall be determined as follows:}\]

\[
S_{ₗₗₗave} = \frac{\sum_{i=1}^{n} J_i / S_v}{n}
\]

where,
\[\text{all variables are as defined previously}\]

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

\[
Φₗ = e^{\lambda (c/(γ_r H))}
\]

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 Appendix 15-E, Table 15-E-2, for selecting soil parameters for design and how soil cohesion should be handled. In general, soil backfill cohesion should be assumed to be zero for design.
However, if soil cohesion (i.e., “true cohesion” as identified in Table 15-E-2) is present, $\Phi_c$ may be used to assess the potential for post-construction deformation and reinforcement load increase. See Appendix 15-E for additional information on this subject.

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

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

\[
T_{max} = S_c \sigma_c K = S_c(K/S_v) \gamma_S \gamma_h (K/K_v)
\]

\[
T_{max} = (S_c(K/S_v) \gamma_S \gamma_h (K/K_v)
\]

15-5.3.10.2  Load and Resistance Factors for the Stiffness Method

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

![Diagram showing the relationship between Simplified Method and Stiffness Method equations.]

### Table 15-5  Load and Resistance Factors for the Stiffness Method

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Reinforcement Type</th>
<th>Load Factor, $\gamma_{p-EV}$ and $\gamma_{p-EVr}$</th>
<th>Live Load, $\gamma_{LL}$</th>
<th>Resistance Factor $\phi_{p}^r$, $\phi_{cr}$, $\phi_{po}$ and $\phi_{sf}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement rupture, $\gamma_{p-EVr}$, and connection failure, $\gamma_{p-con}$ (strength limit)</td>
<td>Geogrids and geotextiles</td>
<td>1.35</td>
<td>1.75</td>
<td>0.80</td>
</tr>
<tr>
<td>Soil failure, $\gamma_{p-EVsf}$ (service limit)</td>
<td>Polymer straps</td>
<td>1.35</td>
<td>1.75</td>
<td>0.55</td>
</tr>
<tr>
<td>Pullout, $\gamma_{p-EV}$ (strength limit - default model in AASHTO 2017)</td>
<td>All geosynthetics</td>
<td>1.20</td>
<td>1.0</td>
<td>1.0</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.
2. AASHTO (2017); 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 2017 are used.
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.

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:

$$
\varepsilon_{\text{rein}} = \gamma_{p-EVsf} T_{\text{max}} / \phi_{sf} J_i < \varepsilon_{\text{max}}
$$

where,

- $\varepsilon_{\text{rein}} = $ the reinforcement strain in any individual reinforcement layer corresponding to $T_{\text{max}}$ (%)
- $\gamma_{p-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)
- $J_i = $ stiffness for layer (kips/ft)
- $\varepsilon_{\text{max}} = $ maximum acceptable strain in the wall cross-section corresponding to $T_{\text{max}}$ in any reinforcement layer (%)

$J_i$, the per layer secant stiffness, should be determined at a strain of 2% for geogrids and geotextiles. $J_i$ should be determined at 1,000 hrs or the estimated time to complete the wall, as specified in AASHTO R-69 (2015). $\phi_{sf}$ is as specified in Table 15-5.

If multiple load sources are acting on the reinforced soil backfill, they shall be added to $T_{\text{max}}$ as determined using Eq. 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 2.5% strain for stiff faced walls and 3.0% strain for flexible faced walls. The average strain from all layers (i.e., average of the $\varepsilon_{\text{rein}}$ values from all the layers) should not exceed 2% and 2.5% for stiff and flexible faced walls, respectively. These criteria have the objective of preventing the development of a contiguous shear surface though the reinforced soil zone.

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, the value of J in Equation 15-14 must be corrected to \( J = J_{2\%} \times R_c \), where \( J_{2\%} \) 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:

\[
L_e \geq \frac{\gamma_{\text{p-EV}} T_{\text{max}}}{\phi_{\text{po}} F^* \sigma_v C R_c}
\]

\[ (15-15) \]

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)
- \( \gamma_{\text{p-EV}} \) = load factor for vertical earth pressure specified in Table 15-5 (dim.)
- \( \phi_{\text{po}} \) = 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, \( \gamma_{\text{p-EV}} T_{\text{max}} \) should be replaced with \( T_{\text{totalf}} \) calculated using superposition, as follows:

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

\[ (15-16) \]

where,

- \( \gamma_{\text{p-EV}} \) = load factor for vertical earth pressure specified in Table 15-5 (dim.)
- \( \gamma_{\text{p-ES}} \) = 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)
- \( k_o \) = active lateral earth pressure coefficient (dim)

Note that Eq. 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:

$$
\gamma_p \cdot EV \cdot T_{\text{max}} \leq \phi T_{\text{al}} R_c
$$

(15-17)

where,

- $T_{\text{max}}$ = applied load in the reinforcement as specified in Section 15.5.3.10.1 (kips/ft)
- $\gamma_p \cdot EV$ = 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.)
- $T_{\text{al}}$ = 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 $\gamma_p \cdot EV \cdot T_{\text{max}}$ with $T_{\text{totalf}}$ calculated as shown below:

$$
T_{\text{totalf}} = \gamma_p \cdot EV \cdot T_{\text{max}} + (\gamma_{LS}) \gamma_f h_{eq} < \phi T_{\text{al}} R_c
$$

(15-18)

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.)
- $\gamma_f$ = unit weight of soil used to calculate live load surcharge, LS (lbs/ft³)
- $h_{eq}$ = equivalent height of soil for live load surcharge (ft)

If multiple load sources other than traffic live load are present, use Eq. 15-16 to determine $T_{\text{totalf}}$.

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 69-15. 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 $T_{\text{ultconn}}$ for modular block facings or ASTM D4884 for seam connections. The connection strength requirements provided for the specific wall systems identified in the appendices to Chapter 15 shall be used.

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

The requirements in the AASHTO LRFD Bridge Design Manual, Article 11.10.7.2, apply, except that $T_{\text{max}}$ 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.
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.

15-5.7 Soil Nail Walls

Soil nail walls shall be designed in accordance with the following manual:


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 such as those underpinning abutments.

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

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

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

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

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

Furthermore, nails installed in soils with strong corrosion potential, defined as:
- pH < 4.5 or > 10 (AASHTO T289),
- Resistivity < 2000 ohm-cm (AASHTO T-288),
- Sulphates > 200 ppm (AASHTO T290), or
- Chlorides > 100 ppm (AASHTO T291)

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

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


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 $\gamma = 130$ 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.

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

15-7 Temporary Cut Slopes and Shoring

This section addresses the design requirements for temporary cut slopes and shoring, both separately and in combination. For temporary cuts and shoring, construction submittals are required in accordance with the Standard Specifications for Road, Bridge, and Municipal Construction M 41-10 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. The resistance factor for global stability of the shoring system should be 0.75 (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 FHWA manuals identified herein shall be used until the next edition of the AASHTO LRFD Bridge Design Manual is published (i.e., the 9th edition), at which point the AASHTO specifications shall be used.

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

15-7.3.3 **Design Loads**

The active, passive, and at-rest earth pressures used to design temporary shoring shall be determined in accordance with the procedures outlined in Article 3.11.5 of the AASHTO LRFD Bridge Design Specifications or Section 5 of the AASHTO Standard Specifications for Highway Bridges (2002) for wall types in which LRFD procedures are not available. Surcharge loads on temporary shoring shall be estimated in accordance with the procedures presented in Article 3.11.6 of the AASHTO LRFD Specifications, or Section 5 of the AASHTO Standard Specifications for Highway Bridges (2002) for wall types in which LRFD procedures are not available. It is important to note that temporary shoring systems often are subject to surcharge loads from stockpiles and construction equipment, and these surcharges loads can be significantly larger than typical vehicle surcharge loads often used for design of permanent structures. The design of temporary shoring must consider the actual construction-related loads that could be imposed on the shoring system. As a minimum, the shoring systems shall be designed for a live load surcharge of 250 psf to address routine construction equipment traffic above the shoring system. For unusual temporary loadings resulting from large cranes or other large equipment placed above the shoring system, the loading imposed by the equipment shall be specifically assessed and taken into account in the design of the shoring system. For the case where large or unusual construction equipment loads will be applied to the shoring system, the construction equipment loads shall still be considered to be a live load, unless the dynamic and transient forces caused by use of the construction equipment can be separated from the construction equipment weight as a dead load, in which case, only the dynamic or transient loads carried or created by the use of the construction equipment need to be considered live load.
As described previously, temporary structures are typically not designed for seismic loads, provided the design life of the shoring system is 3 years or less. Similarly, geologic hazards, such as liquefaction, are not mitigated for temporary shoring systems.

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

15-7.3.4 Design Property Selection

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

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

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

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

15-7.4 Special Requirements for Temporary Cut Slopes

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

WAC 296-155 presents maximum allowable temporary cut slope inclinations based on soil or rock type, as shown in Table 15-8. WAC 296-155 also presents typical sections for compound slopes and slopes combined with trench boxes. The allowable slopes presented in the WAC are applicable to cuts 20 feet or less in height. The WAC requires that slope inclinations steeper than those specified by the WAC or for slope heights greater than 20 feet, as well as slopes in soils or rock not meeting the requirements to be classified as stable rock, or Type A, B, or C soil, shall be designed by a registered professional engineer. As a minimum, the design by or under the supervision of the registered professional engineer shall include a geotechnical slope stability analysis (i.e., Chapter 7) that is based on a knowledge of the subsurface conditions present, including soil and rock stratigraphy, engineering data that can be used to estimate soil and rock properties, and ground water conditions, and with consideration to the loading conditions on or above the slope that could affect its stability. The design shall be conducted in accordance with the requirements in this GDM and referenced documents. Engineering recommendations based upon field observations alone shall not be considered to be an engineering design as defined in the WAC and this GDM.

**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>( \frac{3}{4}H:1V )</td>
</tr>
<tr>
<td>Type B Soil</td>
<td>1H:1V</td>
</tr>
<tr>
<td>Type C Soil</td>
<td>1( \frac{1}{2} )H: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 excessive 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?
2. Performance Objectives for the Shoring System
   a. Is the anticipated design life of the shoring system identified?
   b. Are objectives regarding what the shoring system is to protect, and how to protect it, clearly identified?
   c. Does the shoring system stay within the constraints at the site, such as the right of way limits, boundaries for temporary easements, etc?

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

4. Shoring system loading
   a. Have the anticipated loads on the shoring system been correctly identified, considering all applicable limit states?
   b. If construction or public traffic is near or directly above the shoring system, has a minimum traffic live load surcharge of 250 psf been applied?
   c. If larger construction equipment such as cranes will be placed above the shoring system, have the loads from that equipment been correctly determined and included in the shoring system design?
   d. If the shoring system is to be in place longer than three years, have seismic and other extreme event loads been included in the shoring system design?
5. Shoring system design
   a. Have the correct design procedures been used (i.e., the GDM and referenced design specifications and manuals)?
   b. Have all appropriate limit states been considered (e.g., global stability of slopes above and below wall, global stability of wall/slope combination, internal wall stability, external wall stability, bearing capacity, settlement, lateral deformation, piping or heaving due to differential water head)?

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

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

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

9. Shoring System Removal
   a. Have any elements of the shoring system to be left in place after construction of the permanent structure is complete been identified?
   b. Has a plan been provided regarding how to prevent the remaining elements of the shoring system from interfering with future construction and performance of the finished work (e.g., will the shoring system impede flow of ground water, create a hard spot, create a surface of weakness regarding slope stability)?
15-8 References


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

Appendix 15-H Preapproved Wall Appendix: Specific Requirements and Details for Eureka Reinforced Soil Concrete Panel Walls
Appendix 15-I  Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

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

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

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

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

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

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

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

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

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

Appendix 15-S  Chapter 15 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:

1. Very tall walls, as defined for each wall system in Appendix 15-D.
2. 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.
3. 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.
4. 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 for the face when measured along a 10 feet straight edge at the midpoint of each wall layer shall not exceed:</td>
<td>3 inches</td>
<td>5 inches</td>
</tr>
<tr>
<td>Deviation from the overall design batter per 10 feet of wall height shall not exceed:</td>
<td>2 inches</td>
<td>3 inches</td>
</tr>
<tr>
<td>Maximum outward bulge of the face between backfill reinforcement layers shall not exceed:</td>
<td>4 inches</td>
<td>6 inches</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 Division should be contacted prior to submittal to discuss specific requirements.

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

Responses should be organized in the order shown and referenced to the given numbering system. Additionally, duplication of information is not needed or wanted. A simple statement referencing another section is adequate.

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
GB6 covers welded wire faced structural earth wall materials, GSP 130202.GB covers
precast concrete panel faced structural earth wall materials, and GSP 130203. GB6 covers
concrete block faced structural earth wall materials. All wall components falling into the
categories currently defined by WSDOT should meet the WSDOT material requirements.

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

All geosynthetic reinforced wall systems shall use a soil reinforcement product listed
in the WSDOT Qualified Product List (QPL). Inclusion of geosynthetic reinforcement
products on the QPL will be a necessary prerequisite to wall system approval.
1. For facing units, provide the following information:
   • Standard dimensions and tolerances
   • Joint sizes and details
   • Facing unit to facing unit shear resistance
   • Bearing pads (joints)
   • Spacers
   • Connectors (pins, etc.)
   • Joint filler requirements: geotextile or graded granular
   • Other facing materials, such as for reinforced slopes, or other materials not specifically identified above

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

3. For the connection between the facing units and the soil reinforcements (applies to structural earth walls and reinforced slopes), provide the following information:
   • Photographs/drawings that illustrate the connection
   • 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>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>
<td></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 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>Hilfiker Retaining Walls</td>
<td>Welded Wire Retaining Wall Appendix 15-G</td>
<td>Welded wire facing that is continuous with welded wire soil reinforcement</td>
<td>ASD/LFD</td>
<td>33 feet</td>
<td>Unknown</td>
<td>Approved 11/9/04 (submitted 9/15/03)</td>
</tr>
<tr>
<td>Hilfiker Retaining Walls</td>
<td>Eureka Reinforced Soil Wall Appendix 15-H</td>
<td>Precast concrete 5’×5’ facing panels and welded wire mat soil reinforcement</td>
<td>ASD/LFD</td>
<td>33 feet</td>
<td>Unknown</td>
<td>Approved 11/9/04 (submitted 10/5/04)</td>
</tr>
<tr>
<td>Tensar Earth Technologies, Inc.</td>
<td>ARES Wall Appendix 15-J</td>
<td>Precast concrete 5’×5’ facing panels and Tensar geogrid soil reinforcement</td>
<td>ASD/LFD</td>
<td>33 feet</td>
<td>1998</td>
<td>Approved 11/9/04 (submitted 8/6/04)</td>
</tr>
<tr>
<td>Tensar Earth Technologies, Inc.</td>
<td>MESA Wall Appendix 15-K</td>
<td>Modular dry cast concrete block facing with Tensar geogrid soil reinforcement</td>
<td>ASD/LFD</td>
<td>33 feet</td>
<td>2000</td>
<td>Approved 11/9/04 (submitted 4/19/04 and 9/22/04)</td>
</tr>
<tr>
<td>Tensar Earth Technologies, Inc.</td>
<td>Welded Wire Form Wall Appendix 15-L</td>
<td>Tensar geogrid wrapped face wall with welded wire facing form</td>
<td>ASD/LFD</td>
<td>33 feet²</td>
<td>2006</td>
<td>Approved 3/3/06 (submitted 11/26/05)</td>
</tr>
<tr>
<td>SSL, LLC</td>
<td>MSEPlus Wall Appendix 15-M</td>
<td>Precast concrete 5’×5’ facing panels and steel welded wire strip soil reinforcement</td>
<td>LRFD</td>
<td>33 feet</td>
<td>1999</td>
<td>Approved 8/5/13 (submitted 5/28/13)</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.
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-N</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)</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-Q</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-S</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.
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:

\[
T_{\text{max}} = S_v \left[ H \gamma_f D_{\text{tm}ax} + \left( \frac{H_{\text{ref}}}{H} \right) S \gamma_f \right] K_{\text{avh}} \phi_f \phi_g \phi_{fs} \phi_{local} \phi_c
\]  

(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),
- \( \gamma_f \) = unit weight of wall backfill soil (kcf),
- \( \gamma_f \) = unit weight of surcharge soil (kcf),
- \( D_{\text{tm}ax} \) = \( T_{\text{max}} \) distribution factor,
- \( \phi_g \) = global stiffness influence factor,
- \( \phi_{fs} \) = facing stiffness factor,
- \( \phi_{fb} \) = facing batter factor,
- \( \phi_{local} \) = local stiffness influence factor, and
- \( \phi_c \) = soil cohesion influence 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, M41-10 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 should not be used for final design, 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.
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>$D_{t_{\text{max}}}$</td>
<td>T_{\text{max}} distribution factor</td>
<td>$z_b = C_n \times (H)^y \times \Phi_{f_b}$</td>
<td>$C_n$ (for H in m)</td>
<td>0.40</td>
</tr>
<tr>
<td>$\Phi_g$</td>
<td>Global stiffness factor</td>
<td>$\Phi_g = a \left( \frac{S_{\text{global}}}{P_a} \right)^{\beta}$</td>
<td>$\alpha$</td>
<td>0.16</td>
</tr>
<tr>
<td>$S_{\text{global}}$</td>
<td>Global reinforcement stiffness</td>
<td>$S_{\text{global}} = \frac{J_{\text{ave}}}{(H/n)} = \frac{\sum_{i=1}^{n} J_i}{H}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Phi_{f_{\text{local}}}$</td>
<td>Local stiffness factor</td>
<td>$\Phi_{f_{\text{local}}} = \left( \frac{S_{\text{local}}}{S_{\text{localave}}} \right)^a$</td>
<td>&quot;a&quot; for steel</td>
<td>0</td>
</tr>
<tr>
<td>$S_{\text{local}}$</td>
<td>Local reinforcement stiffness</td>
<td>$S_{\text{local}} = J/S_v$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$S_{\text{localave}}$</td>
<td>Average local reinforcement stiffness</td>
<td>$S_{\text{localave}} = \frac{\sum_{i=1}^{n} J_i}{S_v}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Phi_{f_{b}}$</td>
<td>Facing batter stiffness factor</td>
<td>$\Phi_{f_{b}} = \left( \frac{K_{\text{abh}}}{K_{\text{avh}}} \right)^d$</td>
<td>$d$</td>
<td>0.4</td>
</tr>
<tr>
<td>Coefficient of active earth pressure</td>
<td>$K_{\text{abh}} = \frac{\cos^2(\phi_r + \omega)}{\sin^2(\phi_r / \cos \omega)}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_{\text{avh}} = K_a = (1 - \sin \phi_i)/(1 + \sin \phi_i)$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Phi_{f_{s}}$</td>
<td>Facing stiffness factor</td>
<td>$\Phi_{f_{s}} = \eta \left( \frac{S_{\text{global}}}{P_a} \right)^{\kappa}$</td>
<td>$\eta$</td>
<td>0.57</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>$\Phi_{c} = e^{\lambda(c/(\gamma_r H))}$</td>
<td>$\lambda$</td>
<td>-16</td>
</tr>
</tbody>
</table>

Notes:

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

- $T_{\text{max}}$ = 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)
- $H_{\text{ref}}$ = 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)
- $h_{\text{eff}}$ = equivalent height of an un-jointed facing column that is approximately 100% efficient in transmitting moment through the height of the facing column (ft)
- $K_{\text{abh}}$ = horizontal component of active earth pressure coefficient accounting for wall face batter
- $K_{\text{avh}}$ = horizontal component of active earth pressure coefficient assuming the wall is vertical ($\omega = 0$)
- $J_{\text{2\%}}$ = secant tensile stiffness of geosynthetic reinforcement at 2% strain and 1000 h on a per unit width of reinforcement basis (obtained from laboratory testing) (kips/ft)
- $J_i = J_{\text{2\%}} \times R_c$ = secant tensile stiffness of geosynthetic reinforcement at 2% strain and 1000 h on a per width of wall basis (layer i) (kips/ft)
- $J_{\text{ave}}$ = average secant tensile stiffness of all "n" geosynthetic reinforcement layers (kips/ft)
- $R_c$ = reinforcement coverage ratio
- $\alpha_v$ = 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)
- $q$ = $S \gamma_f$ = average vertical pressure due to soil surcharge on the top of the reinforced soil mass up to a maximum width of 70% of the wall height $H$ (ksf)
- $z$ = depth below wall top measured at the back of the facing (ft)
- $K_a$ = active earth pressure coefficient
- $S$ = average soil surcharge depth above the wall top using a soil surcharge unit weight $\gamma_f$ (ft)
- $\phi_r$ = friction angle of the reinforced soil backfill (degrees)
- $\omega$ = wall face batter in clockwise direction from the vertical (degrees). In AASHTO (2017) the face batter $\theta$ is taken clockwise from the horizontal, hence $\omega = \theta - 90^\circ$
### Table 15-E-2  Soil shear strength parameters recommended for design using the Stiffness Method (after Allen and Bathurst 2018)

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

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 \( \varepsilon_{\text{max},L} \), 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 2014).

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 reinforced soil layers to prevent the development of a contiguous shear surface through the reinforced soil zone.

It may also be useful to estimate the tensile strengths required to achieve the target reinforcement stiffness values. This can be accomplished using the AASHTO NTPEP report stiffness and tensile strength data applicable to the product line in question (e.g., NTPEP 2014). In some cases, the tensile strength to achieve the required reinforcement stiffness may be greater than the tensile strength required to prevent reinforcement rupture and connection failure.

### 15-E-2.2 Reinforcement Strength Design

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

\[
RF = RF_{ID} \times RF_{CR} \times RF_{D} \tag{15-E-3}
\]

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

The long-term (nominal) design strength is:

\[
T_{\text{alt}} = \frac{T_{\text{ult}}}{RF} = \frac{T_{\text{ult}}}{RF_{ID}RF_{CR}RF_{D}} \tag{15-E-4}
\]

where,
- \( T_{\text{ult}} \) = 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 \( T_{\text{alt}} \).

Equation 15-E-1 is the equation to calculate the unfactored reinforcement load \( T_{\text{max}} \) 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:

\[
\gamma_{p-EV} T_{\text{max}} < \phi_{rr} T_{\text{alt}} R_c \tag{15-E-5}
\]

where,
- \( \phi_{rr} \) = the resistance factor for reinforcement rupture.
All parameters are as defined previously.

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

\[
T_{al\text{ (required)}} = \left( \frac{\gamma_{p-EV}}{\phi_{PRC}} \right) T_{\text{max}} = \left( \frac{\gamma_{p-EV}}{\phi_{PRC}} \right) S_v \left[ H \gamma_r D_{t_{\text{max}}} + \left( \frac{H_{\text{ref}}}{H} \right) S_f g \right] \\
\left( \frac{H_{\text{ref}}}{H} \right) S_f g \right]
\]

where,

\[ T_{al\text{ (required)}} \] = 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:

\[
T_{al\text{ (required)}} = \left( \frac{\gamma_{p-EV}}{\phi_{PRC}} \right) T_{\text{max}} = \left( \frac{\gamma_{p-EV}}{\phi_{PRC}} \right) S_v \left[ H \gamma_r D_{t_{\text{max}}} + \left( \frac{H_{\text{ref}}}{H} \right) S_f g \right] \\
\left( \frac{H_{\text{ref}}}{H} \right) S_f g \right] LL \left[ \frac{\gamma_{LL}}{\gamma_{p-EV}} \right] K_{avh} \Phi_f b \Phi_f g \Phi_f s \Phi_{\text{local}} \Phi_c
\]

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:

\[
T_{al\text{ (required)}} = \left( \frac{\gamma_{p-EV} T_{\text{max}} + \gamma_{p-ES} S_v (K_a \Delta \sigma_v + \Delta \sigma_H)}{\phi_{PRC}} \right) S_v \left[ H \gamma_r D_{t_{\text{max}}} + \left( \frac{H_{\text{ref}}}{H} \right) S_f g \right] \\
\left( \frac{H_{\text{ref}}}{H} \right) S_f g \right] LL \left[ \frac{\gamma_{LL}}{\gamma_{p-EV}} \right] K_{avh} \Phi_f b \Phi_f g \Phi_f s \Phi_{\text{local}} \Phi_c
\]

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)

15-E-2.3 Connection Strength Design

AASHTO (2017) specifies that \( T_o \) is equal to 1.0 x \( T_{\text{max}} \), 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 (2017) 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:

\[ \gamma_{P-EvC} T_0 = \phi_c T_{\infty} R_c \]  \hspace{1cm} (15-E-9)

where,
- \( T_{\infty} \) = nominal long-term reinforcement/facing connection strength per unit of reinforcement width (kips/ft)
- \( \gamma_{P-EvC} \) = connection load factor,
- \( R_c \) = connection resistance factor for rupture or pullout of the reinforcement at the connection to the wall face, and
- \( T_0 \) = the reinforcement load at the connection, which is equal to \( 1.0T_{\text{max}} \) and \( T_{\text{max}} \) is determined using the Stiffness Method.

For geosynthetic block-faced walls, the reference (short-term) ultimate connection strength \( (T_{\text{ult} \text{conn}}) \) 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:

\[ T_{\text{ult} \text{conn}} = c_{\text{conn}} + N \tan \phi_{\text{conn}} \]  \hspace{1cm} (15-E-10)

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

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

\( T_{\infty} \) is determined as follows:

\[ T_{\infty} = \frac{T_{\text{ult}} \times CR_{cr}}{RF_D} \]  \hspace{1cm} (15-E-11)

where,
- \( T_{\text{ult}} \) = 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<sub>cr</sub> is determined using RF<sub>CR</sub> 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<sub>cr</sub> shall be based on short-term connection tests and shall be determined as follows:

\[
CR_{cr} = \frac{T_{ultconn}}{RF_{CR}T_{lot}} 
\]

(15-E-12)

where,

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

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

\[
T_{ac} (\text{required}) = \frac{T_{ultconn}}{(RF_D \times RF_{CR})} \geq \left( \frac{\gamma_{con}}{\phi_s R_c} \right) \left( T_{max} + LL \left( \frac{\gamma_{ul}}{\gamma_{con}} \right) S_{K_m} \phi_\sigma \phi_\alpha \phi_\beta \phi_{local} \phi_v \right)
\]

(15-E-13)

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

\[
T_{ult} (\text{required}) = \frac{T_{ultconn}}{CR_u} \geq \left( \frac{\gamma_{con}}{\phi_s R_c} \right) \left( T_{max} + LL \left( \frac{\gamma_{ul}}{\gamma_{con}} \right) S_{K_m} \phi_\sigma \phi_\alpha \phi_\beta \phi_{local} \phi_v \right)
\]

(15-E-14)

All variables are defined previously. For other types of connections, minor modifications to these equations may be needed; see AASHTO (2017) for guidance on handling other facing connection systems.

### 15-E.2.4 Pullout Resistance Limit State Design

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

\[
\gamma_{p-EVTmax} T_{\text{max}} = \phi_{po} P_c
\]

(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:

\[
P_c = C(F^* \alpha) \sigma_v L_e R_c
\]

(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 (2017), Article 11.10.6.3.2.
15-E-2.5 Design Process for the Stiffness Method

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

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

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1 The cohesion factor $\phi_c$ can be calculated here if $\Pi_t > 6$, but in this flow chart example $c = 0$ and therefore $\phi_c = 1.0$.

2 Could multiply $T_{af}$ by $RF$ to get $T_{af}$, and then compare to $T_{af}$ (available). $3$ $3\%$ for flexible faced wall. $4$ $2.5\%$ for flexible faced wall
Appendix 15-E  MSE Wall Design Using the Stiffness Method

References


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

15-F-1  Fill Applications

While most temporary retaining systems are used in cut applications, some temporary retaining systems are also used in fill applications. Typical examples include the use of MSE walls to support preload fills that might otherwise encroach into a wetland or other sensitive area, the use of modular block walls or wrapped face geosynthetic walls to support temporary access road embankments or ramps, and the use of temporary wrapped face geosynthetic walls to support fills during intermediate construction stages.

MSE walls, including wrapped face geosynthetic walls, are well suited for the support of preload fills because they can be constructed quickly, are relatively inexpensive, are suitable for retaining tall fill embankments, and can tolerate significant settlements. Modular block walls without soil reinforcement (e.g., ecology block walls) are also easy to construct and relatively inexpensive; however they should only be used to support relatively short fill embankments and are less tolerant to settlement than MSE walls. Therefore, block walls are better suited to areas with firm subgrade soils where the retained fill thickness behind the walls is less than 15 feet.

15-F-2  MSE Walls

MSE walls are described briefly in Section 15.5.3, and extensively in Publication No. FHWA-NHI-00-043 (Elias, et al., 2001). In general, MSE walls consist of strips or sheets of steel or polymeric reinforcement placed as layers in backfill material and attached to a facing. Facings may consist of concrete blocks or panels, gabions, or a continuation of the reinforcement layer.

15-F-3  Prefabricated Modular Block Walls

Prefabricated modular block walls without soil reinforcement are discussed in Section 15.5.4 and should be designed as gravity retaining structures. Concrete blocks used for gravity walls typically consist of 2½- by 2½- by 5-foot solid rectangular concrete blocks designed to interlock with each other. They are typically cast from excess concrete at concrete batch plants and are relatively inexpensive. Because of their rectangular shape they can be stacked a variety of ways. Because of the tightly fitted configuration of a concrete block wall, oversized blocks will tend to fit together poorly. Occasionally, blocks from a concrete batch plant are found to vary in dimension by several inches.

15-F-4  Common Cut Applications

A wide range of temporary shoring systems are available for cut applications. Each temporary shoring system has advantages and disadvantages, conditions where the system is suitable or not suitable, and specific design considerations. The following sections provide a brief overview of many common temporary shoring systems for cut applications. The “Handbook of Temporary Structures in Construction” (Ratay, 1996) is another useful resource for information on the design and construction of temporary shoring systems.
15-F-5 Trench Boxes

Trench boxes are routinely used to protect workers during installation of utilities and other construction operations requiring access to excavations deeper than 4 feet. Trench boxes consist of two shields connected by internal braces and have a fixed width and height. The typical construction sequence consists of excavation of a trench and then setting the trench box into the excavation prior to allowing workers to gain access to the protected area within the trench box. For utility construction, the trench box is commonly pulled along the excavation by the excavator as the utility construction advances. Some trench boxes are designed such that the trench boxes can be stacked for deeper excavations.

The primary advantage of trench boxes is that they provide protection to workers for a low cost and no site specific design is generally required. Another advantage is that trench boxes are readily available and are easy to use. One disadvantage of trench boxes is that no support is provided to the soils—where existing improvements are located adjacent to the excavation, damage may result if the soils cave-in towards the trench box. Therefore, trench boxes are not suitable for soils that are too weak or soft to temporarily support themselves. Another disadvantage of trench boxes is the internal braces extend across the excavation and can impede access to the excavation. Finally, trench boxes provide no cutoff for groundwater; thus, a temporary dewatering system may be necessary for excavations that extend below the water table for trench boxes to be effective.

Trench boxes are most suitable for trenches or other excavations where the depth is greater than the width of the excavation and soil is present on both sides of the trench boxes. Trench boxes are not appropriate for excavations that are deeper than the trench box.

15-F-6 Sheet Piling

Sheet piling is a common temporary shoring system in cut applications and is particularly beneficial as the sheet piles can act as a diaphragm wall to reduce groundwater seepage into the excavation. Sheet piling typically consists of interlocking steel sheets that are much longer than they are wide. Sheets can also be constructed out of vinyl, aluminum, concrete, or wood; however, steel sheet piling is used most often due to its ability to withstand driving stresses and its ability to be removed and reused for other walls. Sheet piling is typically installed by driving with a vibratory pile driving hammer. For sheet piling in cut applications, the piling is installed first, then the soil in front of the wall is excavated or dredged to the design elevation. There are two general types of sheet pile walls: cantilever, and anchored/braced.

Sheet piling is most often used in waterfront construction; although, sheet piling can be used for many upland applications. One of the primary advantages of sheet piling is that it can provide a cutoff for groundwater flow and the piles can be installed without lowering the groundwater table. Another advantage of sheet piling is that it can be used for irregularly shaped excavations. The ability for the sheet piling to be removed makes sheet piling an attractive shoring alternative for temporary applications. The ability for sheet piling to be anchored by means of ground anchors or deadman anchors (or braced internally) allows sheet piling to be used where deeper excavations are planned or where large surcharge loading is present. One disadvantage of sheet piling is that it is installed by vibrating or driving; thus, in areas where vibration sensitive improvements or soils are
present, sheet piling may not be appropriate. Another disadvantage is that where very dense soils are present or where cobbles, boulders or other obstructions are present, installation of the sheets is difficult.

15-F-7  Soldier Piles

Soldier pile walls are frequently used as temporary shoring in cut applications. The ability for soldier piles to withstand large lateral earth pressures and the proven use adjacent to sensitive infrastructure make soldier piles an attractive shoring alternative. Soldier pile walls typically consist of steel beams installed in drilled shafts; although, drilled shafts filled with steel cages and concrete or precast reinforced concrete beams can be used. Following installation of the steel beam, the shaft is filled with structural concrete, lean concrete, or a combination of the two. The soldier piles are typically spaced 6 to 8 feet on center. As the soil is excavated from in front of the soldier piles, lagging is installed to retain the soils located between adjacent soldier piles. The lagging typically consists of timber; however, reinforced concrete beams, reinforced shotcrete, or steel plates can also be used as lagging. Ground anchors, internal bracing, rakers, or deadman anchors can be incorporated in soldier pile walls where the wall height is higher than about 12 feet, or where backslopes or surcharge loading are present.

Soldier piles are an effective temporary shoring alternative for a variety of soil conditions and for a wide range of wall heights. Soldier piles are particularly effective adjacent to existing improvements that are sensitive to settlement, vibration, or lateral movement. Construction of soldier pile walls is more difficult in soils prone to caving, running sands, or where cobbles, boulders or other obstructions are present; however, construction techniques are available to deal with nearly all soil conditions. The cost of soldier pile walls is higher than some temporary shoring alternatives. In most instances, the steel soldier pile is left in place following construction. Where ground anchors or deadman anchors are used, easements may be required if the anchors extend outside the right-of-way/property boundary. Where ground anchors are used and soft soils are present below the base of the excavation, the toe of the soldier pile should be designed to prevent excessive settlements.

15-F-8  Prefabricated Modular Block Walls

In general, modular blocks (see Section 15.6.6.1.2) for cut applications require the soil deposit to have adequate standup time such that the excavation can be made and the blocks placed without excessive caving. Otherwise large temporary backcuts and subsequent backfill placement may be required. A key advantage to modular block walls is that the blocks can be removed and reused after the temporary structure is no longer needed. One disadvantage to using modular blocks in cut applications is that the blocks are placed in front of an excavation and the soils are initially not in full contact with the blocks unless the areas is backfilled. Some movement of the soil mass is required prior to load being applied to the blocks—this movement can be potentially damaging to upslope improvements.
15-F-9  Braced Cuts

Braced cuts are used in applications where a temporary excavation is required that provides support to the retained soils in order to reduce excessive settlement or lateral movement of the retained soils. Braced cuts are generally used for trenches or other excavations where soil is present on both sides of the excavation and construction activities are not affected by the presence of struts extending across the excavation. A variety of techniques are available for constructing braced cuts; however, most include a vertical element, such as a sheet pile, metal plate, or a soldier pile, that is braced across the excavation by means of struts. Many of the considerations discussed below for soldier pile walls and sheet piling apply to braced cuts.

15-F-10  Soil Nail Walls

The soil nail wall system consists of drilling and grouting rows of steel bars or "nails" behind the excavation face as it is excavated and then covering the face with reinforced shotcrete. The placement of soil nails reinforces the soils located behind the excavation face and increases the soil's ability to resist a mass of soil from sliding into the excavation. Soil nail walls are typically used in dense to very dense granular soils or stiff to hard, low plasticity, fine-grained soils. Soil nail walls are less cost effective in loose to medium dense sands or soft to medium stiff/high plasticity fine-grained soils.

The soils typically are required to have an adequate standup time (to allow placement of the steel wire mesh and/or reinforcing bars to be installed and the shotcrete to be placed). Soils that have short standup times are problematic for soil nailing. Many techniques are available for mitigating short standup time, such as installation of vertical elements (vertical soil nails or light steel beams set in vertical drilled shafts placed several feet on center along the perimeter of the excavation), drilling soil nails through soil berms, use of slot cuts, and flash-coating with shotcrete. Easements may be required if the soil nails extend outside the right-of-way/property boundary.

15-F-11  Uncommon Shoring Systems for Cut Applications

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

15-F-11.1  Diaphragm/Slurry Walls

Diaphragm/slurry walls are constructed by excavating a deep trench around the proposed excavation. The trench is filled with a weighted slurry that keeps the excavation open. The width of the trench is at least as wide as the concrete wall to be constructed. The slurry trench is completed by installing steel reinforcement cages and backfilling the trench with tremied structural concrete that displaces the slurry. The net result is a continuous wall that significantly reduces horizontal ground water flow. Once the concrete cures, the soil is excavated from in front of the slurry wall. Internal bracing and/or ground anchors can be incorporated into slurry walls. Diaphragm/slurry walls can be incorporated into a structure as permanent walls.

Diaphragm/slurry walls are most often used where groundwater is present above the base of the excavation. Slurry walls are also effective where contaminated groundwater is to be contained. Slurry walls can be constructed in dense soils where the use of sheet piling is difficult. Other advantages of slurry walls include the ability to withstand significant
vertical and lateral loads, low construction vibrations, and the ability to construct slurry walls in low-headroom conditions. Slurry walls are particularly effective in soils where high groundwater and loose soils are present, and dewatering could lead to settlement related damage of adjacent improvements, assuming that the soils are not so loose or soft that the slurry is inadequate to prevent squeezing of the very soft soil.

In addition to detailed geotechnical design information, diaphragm/slurry walls require jobsite planning, preparation and control of the slurry, and contractors experienced in construction of slurry walls. For watertight applications, special design and construction considerations are required at the joints between each panel of the slurry wall.

15-F-11.2 Secant Pile Walls

Secant pile walls are another type of diaphragm wall that consist of interconnected drilled shafts. First, every other drilled shaft is drilled and backfilled with low strength concrete without steel reinforcement. Next, structural drilled shafts are installed between the low strength shafts in a manner that the structural shafts overlap the low strength shafts. The structural shafts are typically backfilled with structural concrete and steel reinforcement. The net result is a continuous wall that significantly reduces horizontal ground water flow while retaining soils behind the wall.

Secant pile walls are typically more expensive than many types of cut application temporary shoring alternatives; thus, the use of secant pile walls is limited to situations where secant pile walls are better suited to the site conditions than other shoring alternatives. Conditions where secant pile walls may be more favorable include high groundwater, the need to prevent migration of contaminated groundwater, sites where dewatering may induce settlements below adjacent improvements, sites with soils containing obstructions, and sites where vibrations need to be minimized.

15-F-11.3 Cellular Cofferdams

Sheet pile cellular cofferdams can be used for applications where internal bracing is not desirable due to interference with construction activities within the excavation. Cellular cofferdams are typically used where a dewatered work area or excavation is necessary in open water or where large dewatered heads are required. Cellular cofferdams consist of interlocking steel sheet piles constructed in a circle, or cell. The individual cells are constructed some distance apart along the length of the excavation or area to be dewatered. Each individual cell is joined to adjacent cells by arcs of sheet piles, thus providing a continuous structure. The cells are then filled with soil fill, typically granular fill that can be densified. The resulting structure is a gravity wall that can resist the hydrostatic and lateral earth pressures once the area within the cellular cofferdam is dewatered or excavated. As a gravity structure, cellular cofferdams need adequate bearing; therefore, sites where the cellular cofferdam can be founded on rock or dense soil are most suitable for these structures.

Cellular cofferdams are difficult to construct and require accurate placement of the interlocking sheet piles. Sites that require installation of sheet piles through difficult soils, such as through cobbles or boulders are problematic for cellular cofferdams and can result in driving the sheets out of interlock.
15-F-11.4 Frozen Soil Walls (Ground Freezing)

Frozen soil walls can be used for a variety of temporary shoring applications including construction of deep vertical shafts and tunneling. Frozen soil walls are typically used where conventional shoring alternatives are not feasible or have not been successful. Frozen soil walls can be constructed as gravity structures or as compressive rings. Ground freezing also provides an effective means of cutting of groundwater flows. Frozen soil has compressive strengths similar to concrete. Installation of a frozen soil wall can be completed with little vibration and can be completed around existing utilities or other infrastructure. Ground freezing is typically completed by installing rows of steel freeze pipes along the perimeter of the planned excavation. Refrigerated fluid is then circulated through the pipes at temperatures typically around -20°C to -30°C. Frozen soil forms around each freeze pipe until a continuous mass of frozen soil is present. Once the frozen soil reaches the design thickness, excavation can commence within the frozen soil.

Frozen soil walls can be completed in difficult soil and groundwater conditions where other shoring alternatives are not feasible. Frozen soil walls can provide an effective cutoff for groundwater and are well suited for containment of contaminated groundwater. Frozen soil walls are problematic in soils with rapid groundwater flows, such as coarse sands or gravels, due to the difficulty in freezing the soil. Flooding is also problematic to frozen soil walls where the flood waters come in contact with the frozen soil—a condition which can lead to failure of the shoring. Special care is required where penetrations are planned through frozen soil walls to prevent groundwater flows from flooding the excavation. Accurate installation of freeze pipes is required for deeper excavations to prevent windows of unfrozen soil. Furthermore, ground freezing can result in significant subsidence as the frozen ground thaws. If settlement sensitive structures are below or adjacent to ground that is to be frozen, alternative shoring means should be selected.

15-F-11.5 Deep Soil Mixing

Deep soil mixing (DSM) is an in-situ soil improvement technique used to improve the strength characteristics of panels or columns of native soils. DSM utilizes mixing shafts suspended from a crane to mix cement into the native soils. The result is soil mixed panels or columns of improved soils. Two types of DSM walls can be constructed: gravity walls and diaphragm-type walls. Gravity type DSM walls consist of columns or panels of improved soils configured in a pattern capable of resisting movement of soil into the excavation. Diaphragm-type DSM walls are constructed by improving the soil along the perimeter of the excavation and inserting vertical reinforcement into the improved soil immediately after mixing cement into the soil. The result is a low permeability structural wall that can be anchored with tiebacks, similar to a soldier pile wall, where the improved soil acts as the lagging.

Advantages with deep soil mixing gravity walls include the use of the native soils as part of the shoring system and reduced or no reinforcement. However, a significant volume of the native soils needs to be improved over a wide area to enable the improved soil to act as a gravity structure. Advantages with soil mixed diaphragm walls include the ability to control groundwater seepage, construction of the wall facing simultaneously with placement of steel soldier piles, and a thinner zone of improved soils compared to gravity DSM walls.
DSM walls can be installed top-down by wet methods where mechanical mixing systems combine soil with a cementitious slurry or through bottom up dry soil mixing where mechanical mixing systems mix pre-sheared soil with pneumatically injected cement or lime. DSM is generally appropriate for any soil that is free of boulders or other obstructions; although, it may not be appropriate for highly organic soils. DSM can be completed in very soft to stiff cohesive soils and very loose to medium dense granular soils.

15-F-11.6 Permeation Grouting

Permeation grouting involves the pressurized injection of a fluid grout to improve the strength of the in-situ soils and to reduce the soil’s permeability. A variety of grouts are available—micro-fine cement grout and sodium silicate grout are two of the more frequently used types in permeation grouting. To be effective, the grout must be able to penetrate the soil; therefore, permeation grouting is not applicable in cohesive soils or granular soils with more than about 20 percent fines. Disadvantages of permeation grouting is the expense of the process and the high risk of difficulties. Permeation grouting, like ground freezing or jet grouting, can be used to create gravity retaining walls consisting of improved soils or can be used to create compression rings for access shafts or other circular excavations.

In addition to characterizing the soils gradation and stratigraphy, it is important to characterize the permeability of the soils to evaluate the suitability of permeation grouting.

15-F-11.7 Jet Grouting

Jet grouting is a ground improvement technique that can be used to construct temporary shoring walls and groundwater cutoff walls. Jet grouting can also be used to form a seal or strut at the base of an excavation. Jet grouting is an erosion based technology where high velocity fluids are injected into the soil formation to break down the soil structure and to mix the soil with a cementitious slurry to form columns of improved soil. Jet grouting can be used to construct diaphragm walls to cutoff groundwater flow and can be configured to construct gravity type shoring systems or compressive rings for circular shafts. Jet grouting is applicable to most soil conditions; however, high plasticity clays or stiff to hard cohesive soils are problematic for jet grouting.

Advantages with jet grouting include the ability to use of the native soils as part of the shoring system. A significant volume of the native soils needs to be improved over a wide area to enable the improved soil to act as a gravity structure. The width of the improved soil column is difficult to control, thus the final face of a temporary shoring wall may be irregular or protrude into the excavation.

15-F-12 Factors Influencing Choice of Temporary Shoring

A multitude of factors will influence the choice of temporary shoring systems for a particular application. The most common considerations are cost, subsurface constraints (i.e. difficult driving conditions, the need to cutoff groundwater seepage, etc.), site constraints (i.e. limited access, impacts to adjacent infrastructure, etc.), and local practice. The sections below, while not all-inclusive, provide a brief discussion of several of the factors that influence selection of temporary shoring systems.
15-F-12.1 Application

The first screening criteria for alternative temporary shoring options will be the purpose of the shoring—will it retain an excavation or support a fill.

15-F-12.2 Cut/fill Height

Some retaining systems are more suitable for supporting deep excavations/fill thicknesses than others. Temporary modular block walls are typically suitable only for relatively short fill embankments (less than 15 feet), while MSE walls can be designed to retain fills several tens of feet thick.

In cut applications, the common cantilever retaining systems (sheet piling and soldier piles) are typically most cost effective for retained soil heights of 12 to 15 feet or less. Temporary shoring walls in excess of 15 feet typically require bracing, either external (struts, rakers, etc.) or internal (ground anchors or dead-man anchors).

15-F-13 Soil Conditions

15-F-13.1 Dense Soils and Obstructions

Dense subsurface conditions, such as presented by glacial till or bedrock, result in difficult installation conditions for temporary shoring systems that are typically driven or vibrated into place (sheet piling). Cobbles, boulders and debris within the soil also often present difficult driving conditions. It is often easier to use drilling methods to install shoring in these conditions. However, oversize materials and dense conditions may also hinder conventional auger drilling, resulting in the need for specialized drilling equipment. Methods such as slurry trenches and grouting may become viable in areas with very difficult driving and drilling conditions.

15-F-13.2 Caving Conditions

Caving conditions caused by a combination of relatively loose cohesionless soils and/or groundwater seepage may result in difficult drilling conditions and the need to use casing and/or drilling slurry to keep the holes open.

15-F-13.3 Permeability

Soil permeability is based primarily on the soil grain size distribution and density. It influences how readily groundwater flows through a soil. If soils are very permeable and the excavation will be below the water level, then some sort of groundwater control will be required as part of the shoring system; this could consist of traditional dewatering methods or the use of shoring systems that also function as a barrier to seepage, such as sheet piling and slurry trench methods.

15-F-13.4 Groundwater, Bottom Heave and Piping

The groundwater level with respect to the proposed excavation depth will have a substantial influence on the temporary shoring system selected. Excavations that extend below the groundwater table and that are underlain by relatively permeable soils will require either dewatering, shoring systems that also function as a barrier to groundwater seepage, or some combination thereof. If the anticipated dewatering volumes are high,
issues associated with treating and discharge of the effluent can be problematic. Likewise, large dewatering efforts can cause settlement of nearby structures if they are situated over compressible soils, or they may impact nearby contamination plumes, should they exist. Considerations for barrier systems include the depth to an aquitard to seal off groundwater flow and estimated flow velocities. If groundwater velocity is high, some barrier systems such as frozen ground and permeation grouting will not be suitable.

Bottom heave and piping can occur in soft/loose soils when the hydrostatic pressure below the base of the excavation is significantly greater than the resistance provided by the floor soils. In this case, temporary shoring systems that can be used to create a seepage barrier below the excavation, thus increasing the flow path and reducing the hydrostatic pressure below the base, may be better suited than those that do not function as a barrier. For example, sheet piling can be installed as a seepage barrier well below the base of the excavation, while soldier pile systems cannot. This is especially true if an aquitard is situated below the base of the excavation where the sheet piles can be embedded into the aquitard to seal off the groundwater flow path.

15-F-13.5  High Locked in Lateral Stresses

Glacially consolidated soils, especially fine-grained soils, often have high locked in lateral stresses because of the overconsolidation process (i.e. Ko can be much greater than a typical normally consolidated soil deposit). The Seattle Clay is an example of this type of soil, and much has been written about the performance of cuts into this material made to construct Interstate 5 (Peck, 1963; Sherif, 1966; Andrews, et al., 1966; and Strazer, et al., 1974). When cuts are made into soils with high locked in lateral stresses, they tend to rebound upon the stress relief, which can open up joints and fractures. Hydrostatic pressure buildup in the joints and fractures can function as a hydraulic jack and move blocks of soil, and movement can quickly degrade the shear strength of the soil. Therefore, for excavations into virgin material suspected of having high locked in lateral stresses, temporary shoring methods that limit the initial elastic rebound are required. For example, anchored shoring systems that are loaded and locked-off before the excavation will likely perform better than passive systems that allow the soil move, such as soil nails.

15-F-13.6  Compressible Soils

Compressible soils are more likely to impact the selection of temporary walls used to retain fills. MSE walls are typically more settlement tolerant than other fill walls, such as modular block walls.

15-F-13.7  Space Limitations

Space limitations include external constraints, such as right-of-way issues and adjacent structures, and internal constraints such as the amount of working space required. If excavations are required near existing right-of-ways, then temporary construction easements may be required to install the shoring system. Permanent easements may be required if the shoring systems include support from ground anchors or dead-man anchors that may remain after construction is complete. To minimize the need for temporary and permanent easements, cantilever walls or walls with external bracing (e.g. struts or rakers) should be considered. However, if the work space in front of the excavation needs to be clear, then shoring systems with external support may not be appropriate.
Existing infrastructure, such as underground utilities that cannot be relocated, may have the same impact on the choice of temporary shoring system as nearby right-of-ways.

15-F-13.8  Adjacent Infrastructure

The location of infrastructure adjacent to the site and the sensitivity of the infrastructure to settlement and/or vibrations will influence the selection of temporary shoring. For example, it may be necessary to limit dewatering or incorporate recharge wells if the site soils are susceptible to consolidation if the water table is lowered. If the adjacent infrastructure is brittle or supported above potentially liquefiable soils, it may be necessary to limit vibrations, which may exclude the selection of temporary shoring systems that are driven or vibrated into place, such as sheet piling.

The shoring system itself could also be sensitive to adjacent soil improvement or foundation installation activities. For example, soil improvement activities such as the installation of stone columns in loose to medium dense sands immediately in front of a shoring structure could cause subsidence of the loose sands and movement, or even failure, of the shoring wall. In such cases, the shoring wall shall be designed assuming that the soil immediately in front of the wall could displace significantly, requiring that the wall embedment be deepened and ground anchors be added.

15-F-14  References


Appendix 15-G  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 all permanent walls, the welded wire shall be galvanized in accordance with the AASHTO LRFD specifications. For temporary walls, galvanization is not required, but the life of the wire shall be designed to be adequate for the intended life.

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

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

The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet. Larger diameter culverts or obstructions are not considered preapproved. This wall is also preapproved for use with traffic barriers.

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

The following standard details shall be used for the Hilfiker Welded Wire Faced Wall system:
Appendix 15-H  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

NOTE:
1. REINFORCEMENT BARS SHALL COMPLY TO THE ASTM A615, GRADE 60.
   REINFORCEMENT MILL WILL BE DETAILLED AND DRAWN
   ON PANEL SHOP DRAWINGS.
2. 1/2" x 1/2" CHAMBER SHALL BE PROVIDED ON ALL EXPOSED EDGES (FRONT FACE ONLY).
3. ALL PANEL TYPES AND OTHER RELATED ELEMENTS
   WILL BE DETAILLED ON SHOP DRAWINGS.
4. PANEL DESIGN STRUCTURAL THICKNESS IS 1.0" MINIMAL.
   THIS THICKNESS MUST ACCOMMODATE ANY
   ARCHITECTURAL, SCULPTURED PANELS.
5. ACTUAL LOCATION OF REPAIRS WILL BE ADJUSTED TO
   ACCOMMODATE PANEL CASTING.
6. PANEL REINFORCEMENT SHALL BE PLACED WITH A MINIMUM
   2.0" CLEARANCE FROM THE TE STRIPS. IF MESH
   REINFORCEMENT IS USED, THE TE STRIP LOCATION SHALL BE
   ADJUSTED TO PROVIDE THE MINIMUM REQUIRED CLEARANCE
   OF 1.0".

7. CONCRETE FOR PANELS SHALL HAVE A MINIMUM COMPREHENSIVE
   STRENGTH AFTER 28 DAYS OF 4000 PSI.
8. TIE STRIPS SHALL BE ASTM A615, GRADE 60.
9. VERTICAL REINFORCEMENT BARS SHALL BE 2" MIN.
   CLEAR FROM THE BACK FACE OF THE PANEL.
10. ALL PANELS SHALL BE STAPLED AT 12" CLEAR FROM ANY EDGE OF PANEL.
    PANELS WITH ABSOLUTELY NO REINFORCEMENT DRAWINGS
    SHALL BE PROVIDED ALONG WITH THE ASSEMBLED PANEL.
11. ALL INDIVIDUAL FABRICATION DRAWINGS ARE SHOWN BACK FACE.
12. IN THE CASE OF DOMES AND PANELS WITH HOLES.
    ADDITIONAL REINFORCEMENT SHOWN ON THE SHOP DRAWINGS
    SHALL BE PROVIDED ALONG WITH THE ASSEMBLED PANEL.
13. ALL PANELS SHALL HAVE TWO 2-TON LIFTING HOOKS, EXCEPT.
    FOR "B" AND "A" PANELS. THE "A" AND "B" PANELS SHALL HAVE TWO
    2-TON LIFTING HOES ADDED TO THE 2-TON DECK UNITS.
C.I.P. TRAFFIC BARRIER
OVER SLIP JOINT COVER

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

* SEE WALL ELEVATION

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<th>DESCRIPTION</th>
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Appendix 15-I  Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

C.I.P. TRAFFIC BARRIER
OVER SLIP JOINT COVER

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

* SEE WALL ELEVATION

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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.) EMBEDDED 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"

The Reinforced Earth Company

DESCRIPTION
CLIP ANGLE DETAIL
DATE: 3/04
SHEET NO. 0025
Splice Connection Detail A

Scale: NTS

NOTES:
1. Splice plate connections required on all reinforcing strips between length of 32 feet and 40 feet.

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<td>SINGLE BOLT SPAlice CONNECTION DETAIL</td>
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Appendix 15-L Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

PLAN VIEW

SECTION A–A

SPLOCE CONNECTION DETAIL B

SCALE 1:2

NOTES:

1. SPLOCE PLATE CONNECTIONS REQUIRED ON ALL REINFORCING STRIPS EXCEEDING LENGTH OF 40 FEET.

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<td>DOUBLE BOLTED SPLOCE CONNECTION DETAIL W/ PLATES</td>
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Sheet No. 0027

July 2019
PRECAST COPING SECTION TYPE 1

NOTE:
STANDARD COPING UNIT IS 10'-0" LONG.
Appendix 15-I  Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

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

WSDOT Geotechnical Design Manual  M 46-03.08 Page 15-L-21

October 2013

The Reinforced Earth Company

DESCRIPTION
PRECAST COPING FABRICATION DRAWING - TYPE 1

DATE: 3/04
SHEET NO. 0029

Geotechnical Design Manual  M 46-03.12 Page 15-I-21
July 2019
PRECAST COPING SECTION TYPE 2

NOTE:
STANDARD COPING UNIT IS 10'-0" LONG WITH SQUARE ENDS.
Appendix 15-I: Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

DESCRIPTION
PRECAST COPING FABRICATION DRAWING - TYPE 2

DATE: 3/04
SHEET NO: 0031

The Reinforced Earth Company

Note: Standard coping units are 10'-0" long with square ends. All shorter lengths requiring beveled ends shall be field cut by the contractor.
**SLIP JOINT COVER DETAIL**

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

C.I.P. CONC. COPING W/DITCH

SCALE: 1" = 1'-0"

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NOTES:
1. CONC. = 4000 psi
2. STEEL = GRADE 60
3. ALL LONGITUDINAL BARS ARE #4
4. EXPANSION JOINTS (1/2") EVERY 2 PANEL UNITS
C.I.P. CONC. COPING W/ FENCE

Scale: 1" = 1'-0"

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

1/2" x 1/2" CHAMFER

#4 @ AT DOWEL LOCATION

2 - #4 FOLLOW SLOPE LINE

2" CLR.

#4 PARALLEL W/TOP OF PANEL

2" (LAP)

2" CLAR.

3 - #4 DOWELS 2'-0" LONG EMBEDDED IN PANEL (SEE PARTIAL ELEVATION)

FRONT FACE OF WALL PANEL AND HORIZ. CONTROL LINE

4" MIN.

5 1/2"

C.I.P. CONC. COPING

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

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DESCRIPTION
C.I.P. COPING DETAIL
DATE: 3/04
SHEET NO. 0009
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 % 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.
Appendix 15-I  Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

CONC VERTICAL COPING DETAIL

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

* TO MATCH PRECAST COPING

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October 2013
COPING ENCLOSURE DETAIL

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

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

**SECTION A–A**

SCALE: 3/4" = 1'– 0"

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<td>SHEET NO. 0016B</td>
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OBTUSE BUTT JOINT DETAIL

SCALE: 3/4” = 1’-0”

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The Reinforced Earth Company
2641 Pacific Drive, P.O. Box 565, Puyallup, WA 98373 (P) 253-845-3700
Appendix 15-I  Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

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

90° CORNER ELEMENT DETAIL

* 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.

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<th>DESCRIPTION</th>
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Geotechnical Design Manual  M 46-03.12
July 2019
90° BUTT JOINT DETAIL

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

FRONT FACE OF WALL PANEL

REINFORCING STRIPS

GEOTEXTILE FABRIC
18" WIDE PLACED AS SHOWN (TYPE FX-45HS OR EQUAL)

WORKING POINT
3/4" ± 3/8" OPEN JOINT

Q PANEL JOINT

5 1/2"
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.
Appendix 15-L Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

The Reinforced Earth Company

**DESCRIPTION**

C.I.P. BARRIER PARTIAL ELEVATION
SQUARE PANELS

**DATE:** 3/04

**SHEET NO.:** 0035

NOTE:

JOINTS IN PAVEMENT OR JUNCTION SLAB SHALL CONSIDER WITH JOINTS IN BARRIER.

C.I.P. TRAFFIC BARRIER

PARTIAL ELEVATION

SCALE: 3'/16" = 1'-0"
PARTIAL WALL PLAN AT LIGHT POLE

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

The Reinforced Earth Company

*DESCRIPTION: PARTIAL PLAN AT LIGHT POLE*

*DATE:* 11/04

*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"

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<td>PARTIAL PLAN AT GENERAL OBSTRUCTION</td>
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Appendix 15-I  Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

The Reinforced Earth Company

**Pipe Penetration at Wall Face**

*Description*

- **Place**: 4" x 4" maximum openings, galvanized 9 gage minimum wire mesh around pipe as shown.
- **Preparation**: Form concrete around pipe over exp. jt. mat'l as shown. Allow concrete to set prior to backfilling behind wall.
- **Application**: Wrap and secure filter cloth around pipe over joint prior to backfilling behind wall.
- **Instructions**: Apply bond breaker between precast panels and concrete.

**Front Face of Wall Panel**

- **5 1/2"**
- **4" min.**
- **1" thick expansion joint material banded around pipe as shown**

**Scale**: 1" = 1'-0"
PIPE PENETRATION AT WALL FACE WITH CATCH BASIN DETAIL

DESCRIPTION
PIPE PENETRATION AT WALL FACE W/CATCH BASIN

DATE: 3/04
SHEET NO. 0039
Appendix 15-I  Preapproved Wall Appendix: Specific Requirements and Details for Reinforced Earth (RECO) Concrete Panel Walls

The Reinforced Earth Company

34" - C.I.P. Conc. Traffic Barrier Partial Plan

August 2019

3/04 SHEET NO. 0040
Appendix 15-J  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 ($T_{\text{L}}$) 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-J-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>$T_{ul}$(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>
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<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>
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<td>0.87</td>
<td>3.4</td>
<td>2,530</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>
</tbody>
</table>

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

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

Where:

- $RF = RF_{ID} \times RF_{CR} \times RF_{D}$

and,

- $T_{MARV}$ = The minimum average roll value for the ultimate geosynthetic strength $T_{ult}$
- $CR_u$ = The ultimate connection strength $T_{ult,conn}$ divided by the lot specific ultimate tensile strength, $T_{lot}$ (i.e., the lot of material specific to the connection testing)
- $RF_{ID}$ = Reduction factor for installation damage
- $RF_{CR}$ = Creep reduction factor for the geosynthetic
- $RF_{D}$ = The durability reduction factor for the geosynthetic
Approved details for the Tensar ARES wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- For all plan sheets, the full height panel details are not preapproved. Full height panels may be used by special design, with the approval of the State Bridge Design Engineer and the State Geotechnical Engineer, provided a complete wall design with detailed plans are developed and included in the construction contract (i.e., full height panel walls shall not be submitted as shop drawings in design-bid-build projects).

- In plan sheet 3 of 19, there should be a minimum cover of 4 inches of soil between the geogrid and the traffic barrier reaction slab.

- In plan sheet 8 of 19, the strength of the geogrid and connection available shall be reduced by 10% to account for the skew of the geogrid reinforcement. The skew angle relative to the perpendicular from the wall face shall be no more than 10°.

- In plan sheets 10 and 14 of 19, regarding the filter fabric shown, WSDOT reserves the right to require the use of 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

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<td>ARES Articulated Panels</td>
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<td>Panel Coping</td>
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<td>17.</td>
<td>32 Inch Type &quot;F&quot; Traffic Barrier Standard</td>
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<td></td>
<td></td>
<td>18.</td>
<td>42 Inch Type &quot;F&quot; Traffic Barrier Standard</td>
</tr>
<tr>
<td></td>
<td></td>
<td>19.</td>
<td>42 Inch Type &quot;F&quot; Traffic Barrier Standard</td>
</tr>
</tbody>
</table>

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**CONSTRUCTION DRAWINGS**  
Prepared For  
STATE OF WASHINGTON  
DEPARTMENT OF TRANSPORTATION  

**Tensar**  

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**Preapproved Wall Appendix Specific Requirements and Details for Tensar ARES Walls**  
Appendix 15-M  
Page 15-M-4  
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Preapproved Wall Appendix: Specific Requirements and Details for Tensar ARES Walls

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October 2013

Full Height Panel Typical Corner Element Detail
Not to Scale

Articulated Panel Typical Corner Element Detail
Not to Scale

SHEET Dwg

REVISIONS / ISSUE

STANDARD ARES PRECAST PANEL RETAINING WALL DETAILS

CORNER/SLIP JOINT ELEMENTS

Sheet Number
7 of 19
**Panel Connection Detail Section (A-A)**

**Panel Connection - Bookin Detail**

1. **Reinforcement Steel or Welded Wire Mesh**
   - Max. 3" Embayment
   - Precast Concrete Panel

2. **Primary Geogrid Reinforcement**
   - Pull Tab to Remove Slack

3. **Structural Geogrid Tab**
   - Positioned at Panel Reinforcement and Cast with Panel (Place Transverse Bar Against Reinforcing Steel)

4. **Articulated Panel Connection Detail - Plan View**

5. **Full Height Panel Connection Detail - Plan View**

**Revisions / Issue**

- **0 9/8/12 Signed for Review**

**Standard Ares Precast Panel Retaining Wall Details**

**Geogrid Panel Connection**

Sheet Number 9 of 19
Appendix 15-K  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 ($T_{\text{long}}$) 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, CRu, 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.

### Table 15-K-1  Approved Connection Strength Design Values for Tensar MESA Walls

<table>
<thead>
<tr>
<th>Tensar Geogrid Product</th>
<th>Tult (MARV) for Geogrid per ASTM D6637 in HITEC Report (lbs/ft)</th>
<th>Tult (MARV) for Geogrid per ASTM D6637 for 2003 Product (lbs/ft)</th>
<th>CRu from HITEC Report</th>
<th>*CRu if 2003 Tult (MARV) Values Used</th>
<th>RFCR</th>
<th>CRu if 2003 Tult (MARV) Values Used</th>
<th>Tac (lbs/ft)</th>
</tr>
</thead>
<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>
</tr>
<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.

\[
T_{ac} = \frac{T_{MARV} \cdot CR_u}{RFCR \cdot RF_D} \tag{15-K-1}
\]

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 DOT³ 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-K-1  MESA DOT System Connector and Block

Figure 15-K-2  MESA DOT System Connector and Block as Assembled
Figure 15-K-3 MESA DOT System Connector and Block as Assembled, With Block Placed on Top

Block connectors for block courses with geogrid reinforcement shall be glass fiber reinforced high-density polypropylene conforming to the following minimum material specifications:

<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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<tr>
<td>11.</td>
<td>32 Inch Type &quot;F&quot; Barrier Standard</td>
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<td>12.</td>
<td>42 Inch Type &quot;F&quot; Barrier Standard</td>
</tr>
<tr>
<td>13.</td>
<td>42 Inch Type &quot;F&quot; Barrier Standard</td>
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</tbody>
</table>
Appendix 15-K  Preapproved Wall Appendix: Specific Requirements and Details for Tensar MESA Walls

Installation Procedure

1. Install each course of the Mesa facing units between the top of the last geotextile placed and the bottom of the next layer of geogrid above as shown on the approved construction drawings. The facing units shall be aligned and level in accordance with the wall installation guide.

2. Prior to placing the select backfill, the geotextile shall be placed behind the units such that a minimum of six (6) inches of material is turned into the fill at the top and the bottom. The geotextile shall then be adjusted to prevent a visibly smooth surface.

3. The select backfill shall then be placed and compacted in accordance with the approved construction drawings and project specifications.

4. After the select backfill has been compacted and properly graded for the installation of the next layer of geogrid reinforcement, the geotextile on the top will be canted in position on the facing unit (see Stage 4 detail) or be pulled back onto the geogrid (contractor's option).

5. Install the geogrid reinforcement and repeat the process commencing with Item 1.

6. After the last layer of primary geogrid reinforcement has been placed, install the remaining courses of geotextile facing units except for the last standard course and the cap units, in accordance with the details on the approved construction drawings.

7. Place a line of an approved construction adhesive along the top of the Mesa Standard units approximately one (1) inch behind the face as shown in Detail S1.

8. Place the eight (8) oz. geotextile 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 30 minutes (Detail S1).

9. Install the geotextile connectors in the slots as shown in the wall installation guide. Connectors will push the geotextile into the slot as shown in the Typical Section, this sheet.

10. Install the last course of Mesa Standard units and align and level as required in the installation guide.

11. Place a line of adhesive in the depressed area between the connector slot and the face of the unit per Detail S2.

12. Install the eight (8) oz. geotextile that the leading edge of the material just contacts the line of the depressed area as shown in the Typical Section, this sheet. The bottom of the geotextile shall extend a minimum of six (6) inches beyond the geotextile in the Mesa Standard unit.

13. Place a line of adhesive along the top of the Standard units just behind the face per Detail S3.

14. Install the cap units as shown in Detail S4.

Geotextile widths required for the detail:

- MASA W28 Class 1: 35 inch

**VISIT MESA**

**MESA WALLS & CONSTRUCTION NOTES**

**REVISIONS / ISSUE**

**STANDARD MESA DETAILS & CONSTRUCTION NOTES**

**FABRIC SEPARATOR**

Sheet Number 8 of 13
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-L-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 of 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.
TOP OF WALL

CUT TOP MOST WIRE FORM TO CONFORM WITH FINISHED GRADE OR NEST INTO UNDERLYING WIRE FORM

2” - 4” FREE DRAINING STONE

AASHTO M286 CLASS 3 GEOTEXTILE

LIMIT OF REINFORCED FILL

TENSAR UNIAXIAL STRUCTURAL GEORIDGE

WALL HEIGHT VARIES

WELDED WIRE FACING UNIT

BX1120

SUPPORT STRUT

BODKIN CONNECTION (REQUIRED IF LAYERS ARE COMPOSED OF DIFFERENT GEORIDGE)

FOUNDATION SOIL

GEOGRID EMBEDMENT LENGTH VARIES

TYPICAL CROSS-SECTION

DESCRIPTION: TYPICAL CROSS-SECTION
FILE NAME: WWFSSS6.DWG

Tensar Earth Technologies Inc.
Appendix 15-L  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.
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.

TYPICAL DETAIL
NOTES:
1. SEE WELDED WIRE FACING UNIT DETAIL FOR MATERIAL AND DIMENSIONS.
2. ALL FACING UNITS SHALL BE GALVANIZED AS PER ASTM A123 AFTER FABRICATION.

ALTERNATE WELDED WIRE FACING DETAIL (1" - 2" FACE FILL)

NOT TO SCALE

DESCRIPTION ALT WELD WIRE FACING DETAIL (1" - 2" FACE FILL)

FILE NAME: WWFSS2e020305.DWG

Tensar Earth Technologies Inc.

TYPICAL DETAIL
LIMIT OF PLANTABLE FILL SHALL NOT EXTEND BELOW
THE WELDED WIRE FORM FACING UNIT ABOVE.

WELDED WIRE FORM
FACING UNIT

36" (MIN.)
TOP AND BOTTOM

VARES
(6" MIN.)

TENSA BK1120 GEOGRID

3" (MIN. TRM LENGTH EXTENDING BELOW
THE WELD WIRE FORM FACING UNIT ABOVE)

SUPPORT STRUT

TENSA UNIAXIAL GEOGRID
IN ACCORDANCE WITH ELEVATION VIEW

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

Tensar Earth Technologies Inc.
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 8'-8" WITH 4" OVER LAPPING AT ENDS.
4. STRUT LENGTH AND CROSS-SECTIONAL FORM DIMENSIONS TO BE PROVIDED IN FABRICATORS SHOP DRAWINGS.

Tensar Earth Technologies Inc.

TYPICAL DETAIL
NOTES:
BEND OR CUT BASKETS TO FIT FIELD CONDITIONS
ENSURE THAT GEOTEXTILE AND BIAXIAL GEORGRID OVERLAP 1' MINIMUM

90° OUTSIDE CORNER DETAIL
NOT TO SCALE

Tensar Earth Technologies Inc.

DESCRIPTION: OUTSIDE CORNER DETAIL
FILE NAME: WWFC1.DWG
NOTE:
BEND, BUTT OR CUT BASKETS TO FIT FIELD CONDITIONS

90° INSIDE CORNER DETAIL
NOT TO SCALE

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

DESCRIPTION: GEOGRID PLACEMENT ON CURVES
FILE NAME: GPOC2.DWG

Tensar Earth Technologies Inc.

TYPICAL DETAIL
3" SOIL FILL REQUIRED BETWEEN OVERLAPPING GEORIDS FOR PROPER ANCHORAGE

WALL CORNER DETAIL
NOT TO SCALE

DESCRIPTION: WALL CORNER DETAIL
FILE NAME: GPW4.DWG

Tensar Earth Technologies Inc.
GEOGRID 90° CORNER DETAIL
NOT TO SCALE

Tensar Earth Technologies Inc.
ELEVATION VIEW

PLAN VIEW

NOTES:
1. CUT WIRE FACING AS CLOSE AS POSSIBLE TO PIPE PENETRATION.
2. CUT OR TERMINATE GEOGRIDS 3 INCHES OR LESS FROM PIPE.
3. WRAP ENTIRE PIPE WITH AASHTO M288 CLASS 3 NON-WOVEN DRAINAGE GEOTEXTILE. ENSURE THAT WRAP EXTENDS AT LEAST 12 INCHES BEHIND WIRE FACING AT PENETRATION TO ENSURE NO LOSS OF FILL.
4. FOR GEOGRID LAYOUT REFER TO ELEVATION VIEW FOR LENGTH, TYPE AND LOCATION

PIPE PENETRATION DETAIL AT WELDED WIRE FACE SYSTEM
NOT TO SCALE

DESCRIPTION
PIPE PENETRATION DETAIL AT WELDED WIRE FACE SYSTEM

FILE NAME: GPP10.DWG

Tensar Earth Technologies Inc.

Tensar
GEOGGRID PLACEMENT AT PIPE

NOT TO SCALE

DESCRIPTION: GEOGGRID 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

DESCRIPTION: GEOGRID PENETRATION
FILE NAME: GP2.DWG

TYPICAL DETAIL
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
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 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-M Preapproved Wall Appendix: Specific Requirements and Details for SSL Concrete Panel Walls

Panel Reinforcement Table

<table>
<thead>
<tr>
<th>Panel</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.80 m²</td>
<td>1.40 m²</td>
</tr>
<tr>
<td>B1</td>
<td>0.56 m²</td>
<td>0.77 m²</td>
</tr>
<tr>
<td>B2</td>
<td>0.66 m²</td>
<td>1.10 m²</td>
</tr>
</tbody>
</table>

Max. Spacing Between Bars

Note: Lifting insert shown is typical. Other lifting inserts consist of greater than what is shown in drawings or may be substituted.

Lifting Insert Details

Panel Segments:
- 90° Panel Corners
- 180° Diagonal Panels

Overall Dimensions:
- Standard Panel: 3' x 12' 4" x 10'
- Top: 3' x 12' 4" x 8'
- Bottom: 3' x 12' 4" x 6'

Connection Details:
- Embeds: Vertical
- Embeds: Horizontal

Plan View

Elevation View

Standard Details 1 of 3

Appendix 15-Q

Page 15-Q-4 WSDOT Geotechnical Design Manual  M 46-03.08

October 2013
TYPICAL DRAINAGE DETAILS
(CENTER LINE OF INLET TO BE RELIGATED TO PANEL JOINT)
SCALE: 1"=20'

PLACE OBSTRUCTED ROWS OF MESH IN OUTSIDE CONNECTION OPENINGS TO AVOID OBSTRUCTION.

MESH BELOW THE OBSTRUCTION SHALL BE PLACED IN THE STANDARD CONFIGURATION.

WALL PANEL CONNECTOR FOR ALL PANEL TYPES; SEE DETAILS ON SHEET SD-06 FOR SECTION A-A.

DRAINAGE INLET SHALL BE CENTERED ON THE CLOSEST OBSTRUCTED PANEL JOINT; SEE ELEVATIONS FOR INLET STATION.
PROPER STORAGE AND HANDLING OF WELDED WIRE SOIL REINFORCING PANELS

1. The panels should be stacked one on one, separated by non-staining damage, with a width greater than or equal to the amount of panels per stack. This creates the amount of panels per stack. This creates the amount of panels per stack.

2. Damage should be aligned in the vertical direction. Some panels may be damaged, but all must be supported by the provided pallets shown in the diagram.

3. During panel handling, panels shall be lifted and set by the use of the lifting points located in the top of each panel. The use of the lifting points located in the top of each panel.

4. When lifting panels from the stack, make sure that the additional piece of damage is below the bottom edge of the panel. The additional piece of damage is below the bottom edge of the panel.

5. Lifting one must be vertical to avoid damage to panels.

PROPER STORAGE AND HANDLING OF WELDED WIRE SOIL REINFORCING PANELS

1. Soil reinforcement arrives to the site on a turned in panel, using at least two pick points spaced for more than 7 feet apart. Each panel is designed to be handled directly on the ground.

2. Place pick points on the ground, making sure that the soil reinforcement is not damaged. Place pick points on the ground, making sure that the soil reinforcement is not damaged.

3. Figure that the damage of the soil reinforcement is not directly on the ground. The damage of the soil reinforcement is not directly on the ground.

4. Damage that the damage of the soil reinforcement is not directly on the ground. The damage of the soil reinforcement is not directly on the ground.

FIGURE 2

FIGURE 3
6.0 ERECTION SEQUENCE

6.1 PREPARATION

a. Ensure the top of the pre-tensioned concrete panels is clean and free of debris.

b. Ensure the wall is plumb before starting the erection process.

c. Check the alignment of the panels using a laser level.

6.2 ERECTION

a. The first step in the erection of the wall is to establish the reference lines. These lines will be set using a laser level and a transit instrument.

b. The second step is to establish the top and bottom reference lines. These lines will be set using a laser level and a transit instrument.

c. The third step is to establish the side reference lines. These lines will be set using a laser level and a transit instrument.

d. The fourth step is to establish the corner reference lines. These lines will be set using a laser level and a transit instrument.

6.3 INSTALLATION

a. The panels are installed in the following order:

1. Panel A
2. Panel B
3. Panel C
4. Panel D
5. Panel E
6. Panel F
7. Panel G
8. Panel H
9. Panel I
10. Panel J

b. The panels are installed using a temporary scaffolding and safety harnesses.

6.4 TIGHTENING

a. The panels are tightened using a hydraulic jack and a screw jack.

b. The hydraulic jack is used to apply pressure to the panels.

6.5 FINISHING

a. The finish work is performed after the panels are tight and the scaffolding is removed.

b. The finish work includes grouting and sealing.

6.6 QUALITY CONTROL

a. The quality control process includes visual inspection and testing.

b. The visual inspection includes checking the alignment and finish of the panels.

c. The testing includes the use of a laser level and a transit instrument.

6.7 DOCUMENTATION

a. The documentation process includes recording the erection sequence and the final dimensions of the wall.

b. The documentation process includes the use of photographs and videos.

6.8 CERTIFICATION

a. The wall is certified by an independent testing agency.

b. The certification process includes the use of a laser level and a transit instrument.

c. The certification process includes the use of a laser level and a transit instrument.

6.9 CLEANUP

a. The cleanup process includes removing the scaffolding and safety harnesses.

b. The cleanup process includes removing the temporary reference lines.

6.10 REMOVAL

a. The panels are removed using a hydraulic jack and a screw jack.

b. The panels are removed using a hydraulic jack and a screw jack.
### Appendix 15-M Preapproved Wall Appendix: Specific Requirements and Details for SSL Concrete Panel Walls

**TYPE "X" PANEL**

**SHOWN FROM BACK FACE**

<table>
<thead>
<tr>
<th>O/L</th>
<th>Item Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>VERTICAL BAR</td>
<td>W20 Wire - Grade 60</td>
</tr>
<tr>
<td>2</td>
<td>HORIZONTAL BAR</td>
<td>W20 Wire - Grade 60</td>
</tr>
<tr>
<td>3</td>
<td>LIFTING INSERT</td>
<td>1 TON INSERT</td>
</tr>
<tr>
<td>4</td>
<td>LOOP EMBOS</td>
<td>8 CONNECTION EMBOS</td>
</tr>
</tbody>
</table>

USE W20 SOIL REINFORCEMENT PER LAYER

---

**TYPE "X2" PANEL**

**SHOWN FROM BACK FACE**

**TABLE**

<table>
<thead>
<tr>
<th>O/L</th>
<th>Item Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LOOP EMBOS</td>
<td>B CONNECTION EMBOS</td>
</tr>
</tbody>
</table>

USE W20 SOIL REINFORCEMENT PER LAYER

---

**Design by:**

- DATE: 06/17/12
- NAME: B. JACOBS

**Panel Rebar Details:**

**Type:**

- DATE: 05/13/13
- NAME: D. MITCHELL
LEVELING PAD DETAIL

LEVELING PAD CORNER DETAIL

LEVELING PAD STEP DETAIL

LEVELING PAD DETAILS

PREAPPROVED WALL APPENDIX: SPECIFIC REQUIREMENTS AND DETAILS FOR SSL CONCRETE PANEL WALLS

APPENDIX 15-Q

GEOENHICAL DESIGN MANUAL M 46-03.12

PAGE 15-Q-25

JULY 2019
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-N-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>$T_{ultconn}/T_{lot}$ (lbs/ft)</th>
<th>Creep Reduction Factor applicable to the Connection (use for $RF_{CR}$ in Eq. 1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight</td>
<td>Miragrid 5XT</td>
<td>2800</td>
<td>3844</td>
<td>0.73</td>
<td>$T_{ultconn}/9456$</td>
<td>1.45*</td>
</tr>
<tr>
<td>Block</td>
<td>Miragrid 8XT</td>
<td>4000</td>
<td>6564</td>
<td>0.61</td>
<td>$T_{ultconn}/6564$</td>
<td>1.45*</td>
</tr>
<tr>
<td>Tapered</td>
<td>Miragrid 5XT</td>
<td>2837 + N*Tan 16°</td>
<td>9456</td>
<td>$T_{ultconn}/9456$</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>Block</td>
<td>Miragrid 8XT</td>
<td>4250 + N*Tan 5°</td>
<td>6564</td>
<td>$T_{ultconn}/6564$</td>
<td>1.45*</td>
<td></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>
<td></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.
\[ T_{ac} \text{, the long-term connection strength, shall be calculated as follows:} \]

\[ T_{ac} = \frac{T_{\text{MARV}} \times CR_u \times CR_{\text{cr}} \times CR_{\text{cr}} \times RF_{\text{D}}}{RF_{\text{CR}}} \]

where,

- \( T_{\text{MARV}} \) = 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),
- \( RF_{\text{CR}} \) = creep reduction factor for the geosynthetic, and
- \( RF_{\text{D}} \) = the durability reduction factor for the geosynthetic.

\( RF_{\text{CR}} \) and \( RF_{\text{D}} \) 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 \( \frac{1}{6} \) inch is allowed, but that Section 15.5.3.8 recommends a tighter dimensional tolerance of \( \frac{1}{8} \) 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:

1. Preparing and installing Anchor Landmark modular retaining wall units to the fines 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 for Sampling and Testing Concrete Masonry Units

2. ASTM D 448 Standard Classification for Sizes of Aggregate for Road and Bridge Construction

3. ASTM C 668 Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.46lb Rammer and 154c, Drop. (Standard Practice)

4. ASTM C 1262 Standard Test Method for Evaluating the Freeze-Thaw Durability of Manufactured Concrete Masonry Units and Related Concrete Units

5. ASTM C 1372 Standard Specification for Segmental Retaining Wall Units

6. ASTM D 1556 Standard Test Method for Density of Soil in Place by the Sand Cone Method

7. ASTM D 1557 Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 104lb Rammer and 154c, Drop. (Modified Practice)


9. ASTM D 2522 Standard Test Method for Density of Soil and Soil-Aggregate-In Place by Nuclear Methods (Shallow Depth)

10. ASTM D 4325 Practice for Sampling of Geosynthetics for Testing

11. ASTM D 4596 Test Method for Tensile Properties of Geosynthetics by the Wide Wash Test Method

12. ASTM D 4759 Practice for Determining Specification Conformance of Geosynthetics


14. ASTM D 5262 Creep Limit Strength of Geosynthetics

B. American Association of State Highway Officials


2. AASHTO M 278 Standard Specification for Glass PSA Polyvinyl Chloride (PVC) Pipe

3. AASHTO M 304 Standard Specification for Poly (Vinyl Chloride) (PVC) Profile Wall Drain Pipe and Fittings Based on Controlled Variable Diameter


C. Geosynthetic Research Institute

1. GRI GC4: Allowable Design Strength of Geosynthetics


1.03 SUBMITTALS

A. Submit the following in accordance with Section:

1. Manufacturer’s literature: Materials description

2. Shop drawings: Retaining wall system design, including wall heights, geosynthetic reinforcement layout, drainage provisions and other pertinent details. The shop drawings shall be signed by a registered professional engineer licensed in the state of wall installation.

3. Certificate of Compliance letter in accordance with Section ______ of the Standard Specifications and Section ______ of the Standard Specifications

4. Design calculations demonstrating satisfactory safety factors for:

   1. Overall stability
   2. Internal stability
   3. Bearing capacity
   4. Shifting

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.

C. Store above ground on wood pallets or skids, remove damaged or otherwise unsatisfactory material, when so determined, from the site.

D. The contractor shall prevent excessive moist, wet cement, spay or the material from coming in contact with the modular units and reinforcement.

E. Geosynthetic materials shall remain in protective wrapping until placed in the wall. Follow manufacturer’s recommendations regarding protection from direct sunlight.

F. 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 Mats are a fine draping material used as a drainage media.

D. Reinforced Sandfilled Walls 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 Subsoil is a perforated PVC pipe, generally of 4 inch 100 mm diameter, used to drain water from soil.

G. Filter Fabric is a nonwoven geosynthetic material used for isolation and to allow for the long term passage of water into a subsurface drain system while retaining the in-situ soil.

H. The Landmark Lock Bar is a specifically manufactured polymer based material, supplied by Anchor, and used to mechanically connect the reinforcement products to the Landmark units.

1.06 DISCREPANCIES

Should discrepancies exist between the plans and specifications, plans shall take precedence over the specifications.
PART 2 PRODUCTS

2.01 MATERIALS

A. Modular Block Facing Units: "Landmark" as manufactured under license from Anchor Wall Systems, No substitutions allowed. Landmark units shall meet requirements of ASTM C1372 except as modified by FHWA NH-00-043, section 4.10.

1. Unit height dimensions shall not vary more than ±1/16 inch (1.6mm) from that specified.
2. Unit length dimensions shall not vary more than ±1/32 inch (3.2mm) from that specified.
3. Minimum required compressive strength shall be 4,000 psi (28MPa).
4. Maximum water absorption shall be 5%.
5. Cylinder tests shall be submitted on a monthly basis for approval by the resident engineer.
6. Texture: Offset Split Face
7. The concrete units shall include an integral concrete shear connector to the facing module.
8. Geosynthetic reinforcement: InfraMax 35T X 8T X 10T 4.0. No substitutions allowed.
9. Connections: The Landmark toe bar as supplied by Anchor Wall Systems shall be manufactured from CPVC and PVC to the dimensions shown on the plans.

D. Permeable material: Shall conform to the provisions in Section 1.06.04.01 (Earth Retaining Structures) of the Special Specifications for Permeable Materials.

E. Reinforced Backfill Soil: Shall conform to the provisions in Section 1.06.04.01 (Earth Retaining Structures) of the Special Specifications for Structure Backfill.

F. Sand subbase: Shall conform to the provisions in Section 1.06.04.01 (Sand) of the Standard Specifications and the Special Provisions for this project.

G. Filter fabric: Shall conform to the provisions in Section 1.06.04.01 (Filter Fabric) of the Standard Specifications and the Special Provisions for this project.

H. Levelling Pads: A leveling pad of unreinforced concrete shall be placed to facilitate first course placement. Concrete levelling pads shall have a minimum thickness of 5 inches (125mm) and a minimum width of 24 inches (600mm) and shall have a minimum 28 day compressive strength of 3,000 psi (20.7 MPa).

PART 3 EXECUTION

3.01 EXAMINATION

A. The contractor shall examine the areas and conditions under which the retaining wall is to be erected and notify the owner's representative in writing of conditions detrimental to the proper and timely completion of the work. Contractor shall examine the retaining wall as directed by the owner's representative and notify the owner in writing of conditions detrimental to the proper and timely completion of the work.

B. Foundation soil and rock tests shall be performed by the project geotechnical engineer or technician to ensure that the actual soil properties are consistent with those listed on the construction plans. Leverage test pits shall be performed for soil classification and water seepage.

3.02 EXCAVATION

A. The contractor shall excavate to the lines and grades shown on the construction plans. Overexcavation not approved by the owner's representative shall not be paid for and replacement with approved compacted fill and/or soil system components will be required at the contractor's expense. Do not disturb base beyond the lines shown on the plans.

CERTIFICATE OF AUTHORIZATION 2055

DEAN LAUGHLIN
WAHINGTON DEPARTMENT OF TRANSPORTATION

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. If the project geotechnical engineer shall examine the foundation and select soils to ensure that the soil strength and types meet or exceed those required as shown on the foundation drawings. Foundation soil shall not be acceptable for proper alignment or settlement, shall be remediated at the direction of the engineer.

3.04 BASE COURSE PREPARATION

A. Levelling pad materials shall be as shown on the construction drawings on the pre-approved foundation. Levelling pad materials shall be used as directed by the Engineer.

B. Levelling pad materials shall be installed upon undisturbed soils, or foundation soils prepared in accordance with Section 3.03.

C. Concrete levelling pads shall be allowed to cure for 12 hours prior to placement of the next course of modular units.

D. Levelling pad materials shall be prepared to provide intimate contact with the modular units.

E. Levelling 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 levelling pads. Units shall be checked for horizontal alignment with a string line placed at the back of the units and vertical alignment front to back and side to side with a level. The top of all units in the base course shall be at the same elevation.

B. Ensure that concrete wall units are in full contact with base. A 1 inch (25 mm) gap between 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. Levelling pads shall be placed at the lowest level possible to maintain gravity flow of water to outside of the reinforced zone. Wet subbase shall be placed adjacent to the structure at the top of all units and at 50-foot (15 m) intervals along the wall.

E. Remove all excess fill from top of units and from the back chanel of the top of the units and install next course.

F. Subsequent courses of modular units shall be placed side by side for full length of wall alignment. Vertical gap of 1/8 inch (3.2 mm) is allowable between units. Alignment should be checked by using a string line at the back of the units. Adjust units as needed for proper alignment.

G. If required, a minimum of 12 inches (300 mm) of compacted 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 wall materials.

I. Ensure permeable material and base are compacted after installation of each succeeding course.

J. Install each succeeding course, backfill as each course is completed and prior to placement of the next course. Pull the units forward until the backfilling of the unit contacts the backfilling of the units in the preceding course.

K. Check vertical alignment of each course, adjust units as necessary with reinforcement arms to maintain proper alignment and level.

L. Permeable material or reinforced soil 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 course in the top of the units prior to reinforcement placement.

N. Install geosynthetic reinforcement at locations and elevations shown on the construction drawings.

1. The geosynthetic reinforcement has a primary strength 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 butted 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 fabrics are removed before reinforced units are placed. Panels shall be stapled or anchored as necessary to maintain these conditions.

4. Trenched vehicles may operate directly on geosynthetic reinforcement at speeds less than 10 mph permitted by the reinforcement manufacturer. Scrapped and tearing shall not be allowed.

5. The Landmark toe bar shall be placed at each geosynthetic elevation. Toe bar is not required on courses where geosynthetic reinforcement is not placed. Gaps between adjacent sections of soil shall be no greater than 3 inches (75 mm). The toe bar shall be placed side up, with the angled sides to the back of the unit, as shown on the construction drawings. The reinforcement must be maintained within ±1/2 inch (20 mm) of the face of the modular units below.
3.05 BACKFILL PLACEMENT
A. Special care shall be taken during compaction below the first reinforcement layer to maintain unit level and alignment.
B. All horizontal layers of soil reinforcement shall be thoroughly loosened to an elevation approximately 1" (25 mm) above the level of the ensuing unit before placing the next reinforcement layer.
C. Clean up any debris at the top of the units and from within the channel in the top of the units before placing the next reinforcement layer.
D. 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 backfill, pull the reinforcement taut and anchor in place with staples or tie plates at the back of the reinforcement.
F. Place the reinforced backfill into the channel and spread in a direction parallel to the wall face. Reinforced backfill shall be placed, spread, and compacted in a manner that will minimize slippage or settlement from forming in the reinforcement.
G. Place a minimum of 8" (150 mm) of backfill prior to operating equipment above the reinforcement. Avoid sudden braking or turning on fill placed over the reinforcement.
H. Fill in the reinforced soil zone shall be placed and compacted in lifts not to exceed 6" to 8" inches (150 to 200 mm) in loose thickness where hand-operated compaction equipment is used, and not exceeding 12 inches (300 mm) in loose thickness where heavy, self-propelled compaction equipment is used.
I. Only lightweight, hand-operated compaction equipment shall be allowed within 3 feet (0.9 m) of the back of the Landmark units.
J. All fill placed in the reinforced zone must be compacted in accordance with the project specifications and the project engineer.
K. Compaction tests shall be taken in the reinforced zone. A minimum frequency of one test within the reinforced soil zone per every 5 feet (1.5 m) of wall height for every 100 feet (30 m) of wall is recommended.
L. Prior to turning of construction activity, the reinforced backfill should be graded to drain away from the wall face. Trenches or trenches will be needed to control surface drainage in the vicinity of the retained cut slope, reinforced backfill or wall toe area.

3.07 CAP UNIT INSTALLATION (Where required)
A. Brush clean the top of the upper course of units. Place cap units, cutting as necessary on curved wall portions, prior to setting the cap units.
B. Mortar the preferred material to adhere the cap units to the upper course of modular units.
C. Lay mortar onto 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 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.

END OF SECTION
Appendix 15-N Preapproved Wall Appendix: Specific Requirements and Details for Landmark Reinforced Soil Wall

Geotechnical Design Manual M 46-03.12
July 2019
Appendix 15-O Preapproved Wall Appendix: Specific Requirements and Details for Allan Block Walls

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

Facing Blocks – Blocks acceptable for use with this wall system include, AB Classic, and AB Vertical. These blocks are for a facing batter of 1°, 3°, and 6° degrees. Considering the currently approved block dimensions, the maximum vertical spacing of reinforcement allowed to meet the requirements in the AASHTO Specifications is 2 ft.

Soil Reinforcement – Only geosynthetic reinforcement listed in the WSDOT QPL and which has been evaluated for connection strength with the Allan Block wall system shall be used. For walls with a face batter of 1 degrees or more (i.e., facing blocks, AB Classic, and AB Vertical), this includes the following specific products that are approved for use with this wall system:

- Miragrid 3XT
- Stratagrid SG200
- Miragrid 5XT
- Stratagrid SG350

Reinforcement pullout shall be calculated based on the default values for geogrid reinforcement provided in the AASHTO Specifications.

Reinforcement/Facing Block Connection Requirements – Connection testing was done for the range of blocks and geosynthetic reinforcements preapproved for this wall system. The connection between Allan Block facing units and the geosynthetic reinforcement is essentially a frictional connection. That being the case, the connection resistance is strongly dependent on the normal force between blocks and in the gravel in-fill inside the blocks and less dependent on the roll or lot specific tensile strength, T_{lol}, as well as the long-term effect of creep on the connection strength. However, neither T_{lol} for each test (only T_{MARV} values for the tested geogrids were provided), nor connection creep tests, were provided. Since no connection creep tests were provided, as required in the AASHTO LRFD Bridge Design manual, RF_{CR} must be used to obtain T_{ac}. Therefore, the long-term connection strength (i.e., T_{ac}) equation provided in the AASHTO LRFD Bridge Design Manual will need to be simplified to the equation shown below:

\[ T_{ac} = \frac{T_{ultconn}}{RF_{CR} \times RF_{D}} \]  

(15-O-1)

where,

- \( T_{ultconn} \) is the ultimate connection strength from the product specific connection strength tests, the results of which are provided in Table 15-S-1,
- \( RF_{CR} \) = creep reduction factor for the geosynthetic, and
- \( RF_{D} \) = the durability reduction factor for the geosynthetic.
RF<sub>CR</sub> and RF<sub>D</sub> shall be as provided in the WSDOT QPL, Appendix D.

### Table 15-O-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&lt;sub&gt;ultconn&lt;/sub&gt; (lbs/ft) Facing Batter = 1° or 3°</th>
<th>T&lt;sub&gt;ultconn&lt;/sub&gt; (lbs/ft) Facing Batter = 6°</th>
</tr>
</thead>
<tbody>
<tr>
<td>AB Classic and AB Vertical</td>
<td>Miragrid 3XT</td>
<td>N ≤ 2474, N &gt; 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, N &gt; 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</td>
<td>N ≤ 2474, 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, N &gt; 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 ⅛ inch is allowed, but that WSDOT GDM Section 15-5.3.8 recommends a tighter dimensional tolerance of ¼ 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 ¼ 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

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<th>SHEET NO.</th>
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<td>2</td>
<td>WALL CONSTRUCTION DETAILS (1 OF 2)</td>
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<tr>
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<td>10</td>
<td>WALL PENETRATION DETAILS (2 OF 2)</td>
</tr>
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<td>11</td>
<td>CURVET PENETRATION DETAILS</td>
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<tr>
<td>12</td>
<td>TRANSITION AND SLIP JOINT DETAILS</td>
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</table>
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
7.1: ALLAN BLOCK TYPICAL SECTION – GUIDE RAIL

7.2: ALLAN BLOCK TYPICAL SECTION – PARKING GUARD RAIL

7.3: CEMENT CONCRETE GUTTER DETAIL
Preapproved Wall Appendix: Specific Requirements and Details for Allan Block Walls

Appendix 15-O

GENERAL NOTES:
1. IN ORDER TO ENSURE PROPER DRAINAGE IN AREA SURROUNDING THE VANKOOL CONTRACTOR SHOULD PROVIDE ADDITIONAL DRAINAGE ROCK TO 10'-0" ON ALL SIDES OF WALL STRUCTURE.
2. WITHIN 15' HORIZONTAL FEET OF THE CATCH BASIN EACH BLOCK COURSE ABOVE THE BASE OF THE CATCH BASIN SHOULD CONTAIN A LAYER OF GEORED OF THE SAME TYPE AND LENGTH SPECIFIED ON THE APPROVED CROSS SECTION.
3. REFER TO ALLAN BLOCK SECTIONS FOR ALL OTHER NOTES, DETAILS AND SPECIFICATIONS.

11.1: ALLAN BLOCK TYPICAL SECTION - LARGE GRID OBSTRUCTION BEHIND WALL

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>$T_{ulc,conn}$ (lbs/ft)</th>
<th>$T_{lot}$ (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-P  
Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls

\[ T_{ac} \text{, the long-term connection strength, shall be calculated as follows:} \]

\[
T_{ac} = \frac{T_{MARV} \cdot CR_u}{RF_{CR} \cdot RF_D} \tag{15-P-1}
\]

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),
- \( RFCR \) = creep reduction factor for the geosynthetic, and
- \( RDF \) = the durability reduction factor for the geosynthetic.

\( RFCR \) and \( RDF \) shall be as provided in the WSDOT QPL, Appendix D.

Approved details for the Redi-Rock Positive Connection wall system are provided in the following plan sheets. Exceptions and additional requirements regarding these approved details are as follows:

- Retaining wall heights up to a maximum of 33 feet.
- Retaining walls having a wall face batter of one degree to five degrees.
- The culvert penetration and obstruction avoidance details are preapproved up to a diameter of 4 feet.
- The pipe penetration details for pipes oriented up to a 45 degree skew angle as measured from perpendicular to the wall face are preapproved for pipe diameters of 18 inches or less.
- The cast-in-place concrete to be constructed around pipes that are protruding through the wall face is considered non-preapproved. Detailed stamped drawings and stamped engineering calculations are to be submitted for approval on a project specific basis.
- Reinforcement pullout design shall be calculated based on the default values for geogrid reinforcement provided in the latest edition of the AASHTO LRFD Bridge Design Specifications.
TYPICAL APPURTENANCE INSTALLATION WITH REDI-ROCK WALLS

CONNECTION OPTION #1
- Expansion Anchor or Core Into the 28" Top Block
  - Spacing as Required for Appurtenance
  - Mass of Single Block Available to Resist Overturning Forces

CONNECTION OPTION #2
- Grout Posts in V-Shaped Opening Between 28" Top Blocks
  - Spacing in Multiples of 46 1/8" Increments
  - Mass of 2 Adjacent Blocks Available to Resist Overturning Forces

CONNECTION OPTION #3
- Core Through Top Block and Grout Posts in V-Shaped Opening Between Blocks in Second Course Down
  - Spacing in Multiples of 46 1/8" Increments
  - Mass of 2 Adjacent Blocks in Second Level Down and 3 Top Row Blocks Available to Resist Overturning Forces

Side View
- "Top" Block
- "Middle" Block
- "Bottom" Block
- Connection Details

Front View
- "Top" Block
- "Middle" Block
- "Bottom" Block
- Connection Details

Top View
- "Top" Block
- "Middle" Block
- "Bottom" Block
- Connection Details

REVISED BY:
M. WALZ 05-16-11
C. HINES 05-07-15

Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls
Appendix 15-P
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.
Typical Reinforced Wall with 28" PC (Positive Connection) Blocks

- 12" Wide Strip of Geogrid Wrapped Through Block and Cut to Length of Block
- Perforated Sock Drain As Specified by Engineer (L)
- 12" Wide Strip of Geogrid Wrapped Through Block and Extending Full Length (Typical)
- Move Blocks Forward During Installation to Engage Shear Knobs (Typical)
- Grade to Drain Surface Water Away From Wall

Typical Gravity Wall with 41" Bottom Block Unit

- 1 Degree or Zero Degree Batter Angle Walls are Available Using Blocks with 7 1/2" or 6 3/4" Knobs (Specialty Items)
- 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 Available Using Blocks with 7 1/2" or 6 3/4" Knobs (Specialty Items)
Positive Connection (PC) Details

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

Free Draining Backfill
Crushed No. 57 Stone per WSDOT 9-03.14/C
To Extend at Least 12" Behind Wall

Isometric View of Back of Blocks

Notes:
- 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.

Volume and Center of Gravity (C of G) calculations are based on the blocks as shown.
Center of Gravity is measured from the back of the block.
Half blocks may include a forklift slot on one side.
Actual weights and volumes may vary. Weight shown is based on 143 pcf concrete.

Top - 28" PC Block
Volume = 8.38 cft
Weight = ±1800 lbs
C of G = 15.5"

Middle - 28" PC Block
Volume = 10.77 cft
Weight = ±1540 lbs
C of G = 14.4"

Middle - 41" PC Block
Volume = 15.34 cft
Weight = ±2195 lbs
C of G = 20.7"

Bottom - 28" PC Block
Volume = 11.50 cft
Weight = ±1645 lbs
C of G = 14.5"

Middle - 41" Block
Volume = 17.37 cft
Weight = ±2483 lbs
C of G = 21.3"

Bottom - 41" Block
Volume = 17.37 cft
Weight = ±2483 lbs
C of G = 21.3"
**Geotechnical Design Manual**

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

**Appendix 15-P**

### MOMENT SLAB AND TRAFFIC BARRIER

#### Installation for Sloping Grade

- **Steelfies Per Traffic Barrier Design**: #5 bars at 9" O.C.
- **Cast-In-Place Traffic Barrier**: Sections
  - **30'-0" Sections**
  - **2'-10" Minimum**
- **Concrete Strip**: Refer to contract documents for subgrade requirements
- **Transverse Reinforcement**: #4 bars at 11.5" O.C., top and bottom
- **Dowels at Contraction and Expansion Joints**: 1" expanded polystyrene foam per WSDOT 5-04-6
- **Expansion Joints**: Every 90'-0"
- **Contraction Joints**: Every 30'-0" between expansion joints
- **Concrete for Cast-In-Place Moment Slab**: DOT Standard Structure Mix. Minimum 28 day compressive strength shall be 4,000 psi or higher as specified.
- **Concrete for Cast-In-Place Level-Up**: Manufactured in accordance with ASTM C94 and shall have a minimum 28 day compressive strength of 4,000 psi.
- **Reinforcing Steel**: Conform to ASTM A706 or AASHTO M31 Grade 60. All reinforcing steel shall be epoxy-coated in accordance with ASTM A775 or A934.
- **Materials for Cast-In-Place Moment Slab**: Be DOT Standard Structure Mix. Minimum 28 day compressive strength shall be 4,000 psi or higher as specified. Reinforcing Steel shall conform to ASTM A706 or AASHTO M31 Grade 60. All reinforcing steel shall be epoxy-coated in accordance with ASTM A775 or A934.

**Design**

- **Moment Slab Shown**: Is dimensioned based on an equivalent static load of 10,000 lbs per NCHRP Report 693.

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.

**Redi-Rock PC Series**

*TRAFFIC BARRIER & MOMENT SLAB*

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

**Appendix 15-P**

### MOMENT SLAB AND TRAFFIC BARRIER

#### Installation for Level Grade (0° Slope)

- **Steelfies Per Traffic Barrier Design**: #5 bars at 9" O.C.
- **Cast-In-Place Traffic Barrier**: Sections
  - **30'-0" Sections**
  - **8'-0" Minimum**
- **Concrete Strip**: Refer to contract documents for subgrade requirements
- **Transverse Reinforcement**: #4 bars at 11.5" O.C., top and bottom
- **Dowels at Contraction and Expansion Joints**: 1" expanded polystyrene foam per WSDOT 5-04-6
- **Expansion Joints**: Every 90'-0"
- **Contraction Joints**: Every 30'-0" between expansion joints
- **Concrete for Cast-In-Place Moment Slab**: DOT Standard Structure Mix. Minimum 28 day compressive strength shall be 4,000 psi or higher as specified.
- **Concrete for Cast-In-Place Level-Up**: Manufactured in accordance with ASTM C94 and shall have a minimum 28 day compressive strength of 4,000 psi.
- **Reinforcing Steel**: Conform to ASTM A706 or AASHTO M31 Grade 60. All reinforcing steel shall be epoxy-coated in accordance with ASTM A775 or A934.
- **Materials for Cast-In-Place Moment Slab**: Be DOT Standard Structure Mix. Minimum 28 day compressive strength shall be 4,000 psi or higher as specified. Reinforcing Steel shall conform to ASTM A706 or AASHTO M31 Grade 60. All reinforcing steel shall be epoxy-coated in accordance with ASTM A775 or A934.

**Design**

- **Moment Slab Shown**: Is dimensioned based on an equivalent static load of 10,000 lbs per NCHRP Report 693.

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.

**Redi-Rock PC Series**

*TRAFFIC BARRIER & MOMENT SLAB*
90° OUTSIDE CORNER DETAIL
WITH SPECIALTY CORNER BLOCK

SPECIALTY CORNER BLOCK
(NO SCALE)

4" x 6" x 2" HIGH OVAL KNOB CENTERED ON BLOCK.

TOP VIEW
(+/- 2") (TEXTURE VARIES)

BOTTOM VIEW

ISOMETRIC VIEW
OF CORNER
(NO SCALE)

NOTE:
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)

SPECIALTY CORNER BLOCK

TOP VIEW OF BOTTOM TWO ROWS
(NO SCALE)

Overlap Blocks at Corner to Provide Full Engagement of Shear Knobs, Typical

20" Redi-Rock PC Middle Blocks

Butt Face Of Overlapped Block Directly to side of Adjacent Block, Typical

ISOMETRIC VIEW
OF CORNER
(NO SCALE)

ORDER!

*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 and shop drawings address both structural and aesthetic aspects. Any content that is not applicable to a specific application will be noted for the applicability.

REDI-ROCK PC SERIES
INSIDE & OUTSIDE 90° CORNERS

+/- 23" (TEXTURE VARIES)

18"x46-1/8"x23"
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**Appendix 15-P**

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**Legend**

- **GEOGRID STRIP REINFORCEMENT AROUND BLOCK ON CURRENT LAYER**
- **GEOGRID STRIP REINFORCEMENT AROUND BLOCK ONE LAYER DOWN**

---

**REINFORCEMENT PLACEMENT FOR CONVEX CORNERS**

1. **L**
2. **PRINCIPLE**
3. **REINFORCEMENT DIRECTION**
4. **GEOGRID STRIPS**
5. **SQUARED CORNER**
6. **3" OF SOIL REQUIRED BETWEEN OVERLAPPING REINFORCEMENT**
7. **PLACE 18" HIGH PIECE OF NON-WOVEN GEOTEXTILE FABRIC (WSDOT 9-33.2(2) - TABLE 7)**
8. **IN JOINT BETWEEN BLOCKS (TYP.)**

---

**ENLARGED JOINT DETAIL**

- **PLACE STONE IN JOINT BETWEEN BLOCKS**

---

**REINFORCEMENT PLACEMENT FOR CONCAVE CORNERS**

1. **L**
2. **PRINCIPLE**
3. **REINFORCEMENT DIRECTION**
4. **GEOGRID STRIPS**
5. **GEOGRID STRIP REINFORCEMENT**
6. **AROUND BLOCK ONE LAYER DOWN**
7. **GEOGRID STRIP REINFORCEMENT**
8. **AROUND BLOCK ON CURRENT LAYER**
9. **REDI-ROCK PC SERIES INSIDE & OUTSIDE RADII**

---

**NOTE:**

90° Corners are the Only Corners that can be Constructed. All Other Corners Must be Converted to Radius Segments.

---

**TYPICAL DETAILS PC - LEDGER 100814.dwg**
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#### 45° Outside Corner Radial Solution (Redi-Rock PC Blocks)

**EXPOSED UNTEXTURED BLOCK SURFACE**

- **MIN RADIUS**
- **TOP BLOCKS TIGHT TOGETHER**

**CONVEX CURVES**

<table>
<thead>
<tr>
<th>NUMBER OF COURSES</th>
<th>HEIGHT OF BLOCKS (FACE OF BLOCK)</th>
<th>DISTANCE BETWEEN BLOCKS (IN CHART)</th>
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<td>1</td>
<td>1'-6&quot; 14'-6&quot; 0.13&quot;</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>3'-0&quot; 14'-8&quot; 0.21&quot;</td>
<td></td>
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<tr>
<td>3</td>
<td>4'-6&quot; 14'-10&quot; 0.28&quot;</td>
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<tr>
<td>4</td>
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<tr>
<td>14</td>
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</tr>
</tbody>
</table>

**CONCAVE CURVES**

- **OFFSET FROM THEORETICAL CORNER (SEE CHART)**

**FIRST ROW**

- PLACE BOTTOM ROW OF BLOCKS ACCORDING TO MINIMUM RADIUS REQUIREMENTS
- ROTATE BLOCKS AND MOVE FORWARD TO FULLY ENGAGE BOTH KNOBS BELOW (TYPICAL)

**SECOND ROW**

- PLACE STONE IN JOINT BETWEEN BLOCKS

---

**NOTES:**

- PLACE 18" HIGH PIECE OF NON-WOVEN GEOTEXTILE FABRIC (WSDOT 9-33.2(2) - TABLE 7) IN JOINT BETWEEN BLOCKS (TYP.)
- PLACE BOTTOM ROW OF BLOCKS ACCORDING TO MINIMUM RADIUS REQUIREMENTS
- ROTATE BLOCKS AND MOVE FORWARD TO FULLY ENGAGE BOTH KNOBS BELOW (TYPICAL)

---

**COMPLETED CORNER OUTFIT**

- **OFFSET FROM THEORETICAL CORNER (SEE CHART)**

**DISTANCE BETWEEN BLOCKS (IN CHART)**

- **TOP VIEW**

**ISOMETRIC VIEW**

- **TYPICAL DETAILS PC - LEDGER 100814.dwg**

---

**Appendix 15-P**

*Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls*
PIPES 18" DI. OR SMALLER INSTALLED AT A SKEWED ANGLE TO THE WALL

PROFILE VIEW

SECTION A-A

3D VIEW FROM BACK

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REINFORCEMENT PLACEMENT AROUND VERTICAL OBSTRUCTIONS

With the Redi-Rock PC Series

MAXIMUM DIAMETER = 32" SPACING = 46 1/8" ON-CENTER

GEOGRID STRIPS INSTALLED EVERY OTHER ROW OF BLOCKS

REDA-ROCK PC SERIES

05481 US 31 SOUTH   CHARLEVOIX, MI   49720
866-222-8400 ● 231-237-9521  Fax ●
www.redi-rock.com

SHEET NO.:

REVISED BY:
M. WALZ 05-16-11
C. HINES 05-07-15

LARGE OBSTRUCTION - CONCEPTUAL DETAIL

TOP VIEW

BACK VIEW

BLOCK DETAIL

NOTE:
(1) ALL STRUCTURAL STEEL ELEMENTS TO BE HOT-DIPPED GALVANIZED IN ACCORDANCE WITH ASTM A123 COATING THICKNESS GRADE 85.
(2) THE ABOVE DETAIL IS VALID ONLY FOR OBSTRUCTIONS 84" AND SMALLER AND/OR PC WALL SECTIONS THAT DO NOT REQUIRE GEOGRID REINFORCEMENT WITH TENSILE STRENGTH HIGHER THAN 8XT.

TYPICAL DETAILS PC - LEDGER100814.dwg
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 should be placed in an area where they are not in contact with the reinforced soil or wall. Pipes or other obstructions should be kept at least 3 inches away from the geo-grid to meet the minimum slope and clearance requirements. See contract documents for WSDOT approved backfill around pipe.

In the case of "dry" utilities (electric, gas, telecommunications), maintain a 3-inch minimum between the geo-grid and the pipe. Install geo-grid strips above and below the pipe and keep sufficient separation to meet the maximum geo-grid slope and clearance requirements. See WSDOT 9-33.2(2) - Table 7 for the minimum width centered on the joint (48 inches minimum center to center).

Concrete (Cast-In-Place Around Pipe) and 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 the wall (48-inch concrete pipe shown) should be properly leveled and backfilled to extend at least 12 inches behind the wall. Use a free-draining backfill material, such as crushed No. 57 per WSDOT 9-03.1(4)C.
STEP FOOTING DETAILS

PROFILE VIEW - CONCRETE FOOTING
(No Scale)

LEVELING PAD FOR POSITIVE CONNECTION (PC) BLOCKS

CONCRETE LEVELING PAD

Not shown for clarity

Note: Geogrid reinforcing

• 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 reviewed by the Professional Engineer who is responsible for the wall design.

CAST-IN-PLACE COPING DETAIL

NON-WOVEN GEOTEXTILE OR GEOMEMBRANE BARRIER BETWEEN CAST-IN-PLACE COPING AND TOP OF WALL (TYP.)

#6 STIRRUP @ 12" O.C.

#6 BARS

CAST IN PLACE COPING

HEIGHT VARIES ALONG WALL

1'-0" (MIN) TO 3'-2" (MAX)

GROUND

#6 BARS

SECTION A-A

JUST BEFORE STEP DOWN ON TOP OF WALL

SECTION B-B

JUST AFTER STEP DOWN ON TOP OF WALL

EXPANSION JOINT MATERIAL BETWEEN COPING SECTIONS, LOCATE AT ELEVATION CHANGES IN WALL (2'-0" FT. MAX.)

LENGTH OF COPING SECTIONS VARIES

ELEVATION VIEW

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)

CAST IN PLACE COPING

#6 STIRRUP @ 12" O.C.

EXPANSION JOINT MATERIAL BETWEEN COPING SECTIONS, LOCATE AT ELEVATION CHANGES IN WALL (20.0 FT. MAX.)

LENGTH OF COPING SECTIONS VARIES

ELEVATION VIEW

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)

CAST IN PLACE COPING

#6 STIRRUP @ 12" O.C.
FINISH GRADE TO EDGE OF BLOCKS
TOP OF WALL TREATMENT USING REDI-ROCK TOP AND GARDEN BLOCKS INSTEAD OF COPING

ALTERNATE GARDEN BLOCK PLACEMENT

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
CUSTOM WEEP HOLE PIPE CAST IN BLOCK

FIELD INSTALLED WEEP HOLE PIPE

1. **CURVED HOLE PIPE**
   - Pipe to extend 6" to 8" from back of block.
   - Connect to perforated wall drain.
   - Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.

2. **CURVED HOLE PIPE**
   - Notch 2.5" x 5" hole in side of Redi-Rock block.
   - Connect to perforated wall drain.

**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
- Diameter (Dia.) as specified on plans.
- Locate center of pipe 10" left and 6.75" down from top right corner of block.

**FIELD INSTALLED WEEP HOLE PIPE**

- Pipe to extend 6" to 8" from back of block.
- Connect to perforated wall drain.
- Notch 2.5" x 5" hole in side of Redi-Rock block.
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.

**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
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- Pipe to extend 6" to 8" from back of block.
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**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
- Diameter (Dia.) as specified on plans.
- Locate center of pipe 10" left and 6.75" down from top right corner of block.

**FIELD INSTALLED WEEP HOLE PIPE**

- Pipe to extend 6" to 8" from back of block.
- Connect to perforated wall drain.
- Notch 2.5" x 5" hole in side of Redi-Rock block.
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.

**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
- Diameter (Dia.) as specified on plans.
- Locate center of pipe 10" left and 6.75" down from top right corner of block.

**FIELD INSTALLED WEEP HOLE PIPE**

- Pipe to extend 6" to 8" from back of block.
- Connect to perforated wall drain.
- Notch 2.5" x 5" hole in side of Redi-Rock block.
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.

**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
- Diameter (Dia.) as specified on plans.
- Locate center of pipe 10" left and 6.75" down from top right corner of block.

**FIELD INSTALLED WEEP HOLE PIPE**

- Pipe to extend 6" to 8" from back of block.
- Connect to perforated wall drain.
- Notch 2.5" x 5" hole in side of Redi-Rock block.
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.

**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
- Diameter (Dia.) as specified on plans.
- Locate center of pipe 10" left and 6.75" down from top right corner of block.

**FIELD INSTALLED WEEP HOLE PIPE**

- Pipe to extend 6" to 8" from back of block.
- Connect to perforated wall drain.
- Notch 2.5" x 5" hole in side of Redi-Rock block.
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.

**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
- Diameter (Dia.) as specified on plans.
- Locate center of pipe 10" left and 6.75" down from top right corner of block.

**FIELD INSTALLED WEEP HOLE PIPE**

- Pipe to extend 6" to 8" from back of block.
- Connect to perforated wall drain.
- Notch 2.5" x 5" hole in side of Redi-Rock block.
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.

**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
- Diameter (Dia.) as specified on plans.
- Locate center of pipe 10" left and 6.75" down from top right corner of block.

**FIELD INSTALLED WEEP HOLE PIPE**

- Pipe to extend 6" to 8" from back of block.
- Connect to perforated wall drain.
- Notch 2.5" x 5" hole in side of Redi-Rock block.
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.

**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
- Diameter (Dia.) as specified on plans.
- Locate center of pipe 10" left and 6.75" down from top right corner of block.

**FIELD INSTALLED WEEP HOLE PIPE**

- Pipe to extend 6" to 8" from back of block.
- Connect to perforated wall drain.
- Notch 2.5" x 5" hole in side of Redi-Rock block.
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.

**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
- Diameter (Dia.) as specified on plans.
- Locate center of pipe 10" left and 6.75" down from top right corner of block.

**FIELD INSTALLED WEEP HOLE PIPE**

- Pipe to extend 6" to 8" from back of block.
- Connect to perforated wall drain.
- Notch 2.5" x 5" hole in side of Redi-Rock block.
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.

**CUSTOM WEEP HOLE PIPE CAST IN BLOCK**

- Solid PVC or HDPE pipe cast in block.
- Diameter (Dia.) as specified on plans.
- Locate center of pipe 10" left and 6.75" down from top right corner of block.

**FIELD INSTALLED WEEP HOLE PIPE**

- Pipe to extend 6" to 8" from back of block.
- Connect to perforated wall drain.
- Notch 2.5" x 5" hole in side of Redi-Rock block.
- Place solid PVC or HDPE drain pipe through notched hole and grout pipe in place.
Appendix 15-P
Preapproved Wall Appendix: Specific Requirements and Details for Redi-Rock Positive Connection Walls

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

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### REDI-ROCK PC CAP & CORNER UNIT BLOCKS

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<thead>
<tr>
<th>Block Type</th>
<th>Volume</th>
<th>Weight</th>
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<td>4.68 ft³</td>
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<tr>
<td>Two-Sided Curve Cap</td>
<td>4.81 ft³</td>
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<td>Three-Sided Cap</td>
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</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 for the blocks as shown.
- Actual weights and volumes may vary.
- Weight shown is based on 143 pcf concrete.

**Dimensions Updated 11/20/13**
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 backfill in a windrow slightly greater than full lift against lock + load panel to hold geogrid in place.

   Roll grid into place. Stake or hold the geogrid taut and free of wrinkles while placing backfill. 12 inches at back of panel and 8 inches over the tail of the counterfort back.

3. Place the geogrid tail over the windrow, slope tail to top of rear of counterfort. Lock into place with backfill.

   Entire reinforced zone to be compacted to WSDOT standard specification for MSE reinforced zone.

LOCK+LOAD LIFT ASSEMBLY DETAIL
GRID DETAIL

TYPICAL SECTION AT WALL FACE

NOTE: REINFORCED ZONE TO BE COMPACTED TO WSDOT STANDARD SPECIFICATION FOR MSE REINFORCMENT ZONE.

ASYMPLY DETAIL

3/4" VERTICAL GAP
CONSIDERATION GAP

CUT ONLY PERPENDICULAR TO WALL PANEL

GEOTEXTILE GRID

LOCK-ON-GUARD PANEL

GEO GRID

CUTTED GEOTEXTILE ON BOTH SIDES OF COUNTERFORT TO RELIEVE TENSION.

FULL UNIFORMED COMPACTATION IS MANDATORY.

PRODUCT SPECIFIED REINFORCED BACKFILL ZONE.

# ALL REINFORCEMENT PRODUCTS FROM WSDOT QPL LIST.

ASSEMBLY DETAIL

GEOTEXTILE GRID WRAP
MINIMUM 4 FEET 

GAVEL BARROW, WSDOT SPEC
9-03-L14(4) MAX PARTIAL SIZE

GAVEL BARROW, WSDOT SPEC
9-03-L14(4) MAX PARTIAL SIZE

GEOTEXTILE COUNTERFORT

PANEL LOOP
Preapproved Wall Appendix: Specific Requirements and Details for Lock and Load Walls

Appendix 15-P

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October 2013

Front View  Top View  Side View

Half Panel Connecting Loop Detail

Note:
1) Minimum concrete compressive strength at 28 days is 5500 PSI
2) 6% Air Entrainment
3) 3 lbs structural fiber per cubic yard
4) Facing texture as specified in the contract

Back View  Side View

RIGHT ANGLE DISTANCE FROM COUNTERFORT BEARING AREA ON BACK OF PANEL TO INSIDE OF LOOP
6.25" TOLERANCE -0 +.25"

Connecting Loop

Note:
- Material is 1/4" DIA. T304 115 KSI Stainless Steel Wire ASTM A580
- All dimensions are outside to outside including radius
- Welding in accordance with AASHTO/AWS D1.5M/D1.5
NOTE:
1) MINIMUM CONCRETE COMpressive strength at 28 days 5500 PSI
2) 6% AIR ENTRAINMENT
3) 3 LBS STRUCTURAL FIBER PER CUBIC YARD

BAR BENDING DETAIL

NOTE:
1) MATERIAL IS 1/4" DIA. T304 115 KSI STAINLESS STEEL WIRE ASTM A580

CENTER LINE OF WIRE

LOCATION

1/4" DIA. T304 115 KSI STAINLESS STEEL WIRE ASTM A580

PICTORIAL VIEW
**Preapproved Wall Appendix: Specific Requirements and Details for Lock and Load Walls**

**Appendix 15-Q**

**Geotechnical Design Manual M 46-03.12 Page 15-Q-7**

**July 2019**

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

**WSDOT Geotechnical Design Manual M 46-03.08 Page 15-P-7**

**October 2013**

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**TOP VIEW**

**LEFT OUTSIDE CORNER**

- 6.75" -2.5" -1.25" -13"
- 5" -1.5"
- 2.7" -1.5"

**CONCRETE STIFFENER FOR 90 DEGREE CORNER**

**STANDARD LOOP HEIGHT TOLERANCE -0 +.25**

**CONNECTING LOOP .25 DIA. 6.75" 1.40" 0.7" 1.40"**

**SIDE VIEW**

**TOP VIEW**

**RIGHT OUTSIDE CORNER**

**TOP VIEW**

**LEFT OUTSIDE CORNER**

**NOTE:**

1. Minimum concrete compressive strength 5000 psi at 28 days
2. 6% air entrainment
3. 3 lbs structural fiber per cubic yard
4. Facing texture as specified in contract

---

**CORNER REINFORCEMENT DETAILS**

**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/AWS D1.1M/D1.5

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**REV. DESCRIPTION DATE**

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**LOCK+LOAD**

---

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

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

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NOTES:
1) MINIMUM CONCRETE COMPRESSIVE STRENGTH AT 28 DAYS IS 5500 psi
2) 6% AIR ENTRAINMENT
3) 3 LBS STRUCTURAL FIBER PER CUBIC YARD
4) COPING ONLY REQUIRED IF SPECIFIED

COPING DETAILS

COPING SIDE VIEW

CONSTRUCTION ADHESIVE IN BETWEEN COPING PIECES

COPING TOP VIEW

CONSTRUCTION ADHESIVE

ASSEMBLY SECTION

COUNTERFORT CONSTRUCTION ADHESIVE ALONG TOP OF PANEL

TOP VIEW OF PANEL

CONSTRUCTION ADHESIVE ALONG TOP OF PANEL

COUNTERFORT

COPING

CONSTRUCTION ADHESIVE

PANEL

PANEL
ALL GEORGRID MUST BE 100% OVER COUNTERFORT AND EXTEND TO BACK OF PANEL AND VERTICAL FOR 4" GEORGRID 100% COVERAGE UNLESS SPECIFIED OTHERWISE.

PLAN VIEW ACUTE OUTSIDE CORNER 71-89 DEGREES

PLAN VIEW ACUTE INSIDE CORNER 90-180 DEGREES

PLAN VIEW ACUTE INSIDE CORNER 90 DEGREES AND LESS (MINIMUM ANGLE 45°)

PLAN VIEW OBTUSE INSIDE CORNER 90-180 DEGREES

PLAN VIEW OBTUSE INSIDE CORNER 90 DEGREE

"3" OF BACK FILL BETWEEN LAPPED GEORGRID

PLAN VIEW ORTHOGONAL OUTSIDE CORNER 90 DEGREE

PLAN VIEW ORTHOGONAL INSIDE CORNER 90 DEGREE

"3" OF BACK FILL BETWEEN LAPPED GEORGRID
USE TRIM PANELS OR HALF WIDE PANELS TO PICK UP RUNNING BOND MINIMUM 3 PANELS OUT FROM PIPE

TP = TRIM PANEL  HP = HALF PANEL

MAX PIPE DIA 24"

ELEVATION DETAIL FOR PIPE DIAMETERS 24" OR LESS

NOTE:
1) TRIM PANELS TO FIT OUTSIDE DIAMETER OF PIPE
2) NO GEOSYNTHETIC SOIL REINFORCEMENT TO BE EXPOSED AROUND PIPE

6" THICK CEMENT PLACED CONCRETE HEAD WALL (9°F - 6000 PSI WITH MIN. #4 REBAR AT 12" O.C EACH WAY) BY OTHERS

PIPE GREATER THEN 24" DIA

ELEVATION DETAIL FOR PIPE DIAMETERS GREATER THAN 24"

NOTE:
SPECIAL HEADWALL DESIGN IS REQUIRED FOR PIPE DIAMETERS GREATER THAN 3 FEET

24" PIPE PENETRATION CROSS SECTION AT WALL FACE

PIPE PENETRATION THROUGH THE FACE OF WALL USING LOCK & LOAD

FOOTING PAD

DETAIL FOR PIPE DIAMETERS 24" AT UP TO 45"

NOTE:
1) TRIM PANELS TO FIT OUTSIDE DIAMETER OF PIPE
2) NO GEOSYNTHETIC SOIL REINFORCEMENT TO BE EXPOSED AROUND PIPE
Appendix 15-Q Preapproved Wall Appendix: Specific Requirements and Details for Lock and Load Walls

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

Appendix 15-P

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LOCK+LOAD

SPEC BACKFILL FOR GEOGRID REINFORCED MSEW ZONE

FACING GRAVEL, WSDOT SPEC 9-03.14(4) 3/4" MAX PARTICAL SIZE

2" STYROFOAM EXPANSION JOINT

3/4" x 6" SIDEWALK EXPANSION STRIP

WSDOT RAILING (AS REQUIRED)

42" MIN

SIDEWALK DETAIL
Typical Swale Section

Note:
Concrete footing minimum compressive strength after 28 days 3000 psi
Appendix 15-R  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 units (block width into the wall $W_u = 1$ ft). 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 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 table:

<table>
<thead>
<tr>
<th>Geogrid Product</th>
<th>Wall Height (H) (feet)</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>$N &lt; 1074$</td>
<td>$915 + N \tan 45^\circ$</td>
<td>3484</td>
</tr>
<tr>
<td></td>
<td>$9 &lt; H &lt; 18.9$</td>
<td>$1074 &lt; N &lt; 2268$</td>
<td>$1465 + N \tan 26^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$H &gt; 18.9$</td>
<td>$N &gt; 2268$</td>
<td>$2571$</td>
<td></td>
</tr>
<tr>
<td>Miragrid 5XT</td>
<td>$H &lt; 15.3$</td>
<td>$N &lt; 1837$</td>
<td>$1456 + N \tan 27^\circ$</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>$2101 + N \tan 9^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$H &gt; 28.5$</td>
<td>$N &gt; 3424$</td>
<td>$2643$</td>
<td></td>
</tr>
<tr>
<td>Miragrid 7XT</td>
<td>$H &lt; 11.9$</td>
<td>$N &lt; 1425$</td>
<td>$817 + N \tan 47.3^\circ$</td>
<td>6713</td>
</tr>
<tr>
<td></td>
<td>$11.9 &lt; H &lt; 28.5$</td>
<td>$1425 &lt; N &lt; 3417$</td>
<td>$1736 + N \tan 23.7^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$H &gt; 28.5$</td>
<td>$N &gt; 3417$</td>
<td>$3236$</td>
<td></td>
</tr>
<tr>
<td>Miragrid 8XT</td>
<td>$H &lt; 12.5$</td>
<td>$N &lt; 1500$</td>
<td>$1064 + N \tan 43^\circ$</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>$1365 + N \tan 36.2^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$H &gt; 28.2$</td>
<td>$N &gt; 3389$</td>
<td>$3845$</td>
<td></td>
</tr>
<tr>
<td>Miragrid 10XT</td>
<td>$H &lt; 6.3$</td>
<td>$N &lt; 750$</td>
<td>$1335 + N \tan 49.5^\circ$</td>
<td>10795</td>
</tr>
<tr>
<td></td>
<td>$6.3 &lt; H &lt; 28.2$</td>
<td>$750 &lt; N &lt; 2903$</td>
<td>$1753 + N \tan 31.5^\circ$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$H &gt; 28.2$</td>
<td>$N &gt; 2903$</td>
<td>$3532$</td>
<td></td>
</tr>
</tbody>
</table>

$N =$ 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:

$$T_{ac} = \frac{T_{MARV} \cdot CR_u}{RF_{CR} \cdot RF_D}$$  \hspace{1cm} (15-R-1)

where,

- $T_{MARV} = \text{the minimum average roll value for the ultimate geosynthetic strength } T_{ult}$,
- $CR_u = \frac{T_{ultconn}}{T_{lot}}$, 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),
- $RF_{CR} = \text{creep reduction factor for the geosynthetic}$, and
- $RF_D = \text{the durability reduction factor for the geosynthetic}$.

$RF_{CR}$ and $RF_D$ shall be as provided in the WSDOT QPL, Appendix D.

Approved details for the 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 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.
The 3 Plane Compac II Unit is not approved for use on WSDOT projects. See WSDOT Standard Specification 6-13.34.
Note:
Place two keystone fiberglass pins in each unit.

Pin Connection - Near Vertical Setback Section

Geogrid & Pin Connection - Near Vertical Setback Section

Compac II Block to Block
Pin Connections Isometrics
Appendix 15-R  Preapproved Wall Appendix: Specific Requirements and Details for KeyGrid Walls

Notes:
Keystone Compact II units shown.

Due to corner perpendicular wall setback per course, to maintain running bond course alignment, cut the adjoining unit to the perpendicular wall face labeled "Compact II Cut Unit" as needed in both directions for proper wall joint alignment.

Gauged reinforcement not shown for detail clarity, design as per engineer.

For angles between 130° and 170°, it is recommended to install a radius curve.

Odd Numbered Courses Iso View

Even Numbered Courses Iso View

Unreinforced Concrete Leveling Pad

Unreinforced Concrete Leveling Pad

Compact II Cut Unit

Compact II Cut Unit

Typical Base and/or Odd Numbered Courses

Typical Second and/or Even Numbered Courses

Unit Drainage Fill Limits (WISDOT 9-03.93)

Center of Unit Face with Adjoining Wall Face (Base Course Only)
Appendix 15-R  Preapproved Wall Appendix: Specific Requirements and Details for KeyGrid Walls

Notes:

Keystone Compact II units shown.

Compact Ii corner piece units to be used for each odd or even course vertically up the wall corner.

Due to corner perpendicular wall setback per course, to maintain running bond course alignment, cut the next unit from each corner unit labeled "Compact II Cut Unit". By cutting the adjoining edge of the Compact II unit on either side of the corner unit enables the wall to maintain proper wall alignment.

Place additional unit drainage fill at inside wall corners to extend back from wall face each way a distance equal to the wall height 1/2 (0.125).

For angles between 135° and 170°, it is recommended that a radius curve be installed.

Geogrid not shown for drawing clarity. Geogrid reinforcement design as per engineer.

Typical Base and / or Odd Numbered Courses

Odd Numbered Courses Iso View

Even Numbered Courses Iso View

Unit Drainage Fill Limits (WSDOT 9-03.3(3))

Additional Wall Corner Unit Drainage Fill Limits (See Notes)

Keystone Compact II Unit

Unreinforced Concrete Leveling Pad

Compact II Cut Unit (After Base Course)

Keystone Compact II Unit (Typ)

Keystone Compact II Corner Units

Unreinforced Concrete Leveling Pad

Install 2' Long #4 Rebar and Grout (non-shrink) after completion of 3 courses

Typical Second and / or Even Numbered Courses

Typical Oustide Corner Details
Notes:

Keystone Compac II units shown.

Full corner piece units to be used for each odd or even course vertically up the wall corner.

Due to corner perpendicular wall setback per course, to maintain running bond course alignment, cut the next unit back from each corner unit labeled “Compac II Cut Unit.” By cutting the adjoining edge of the Compac II unit on either side of the corner unit enables the wall to maintain proper wall alignment.

Place additional unit drainage fill at outside wall corners to extend back from wall face each way a distance equal to the wall height / 2 (H/2).

Geogrid not shown for clarity. Geogrid reinforcement design step by step by engineer.

Typical Base and/or Odd Numbered Courses

Typical Second and/or Even Numbered Courses
Installation Notes:
- Gapping between the units will occur as the units batter, move or setback away from the point of radius. The rate of gapping is controlled by the severity of the batter (i.e., a '5' on setback will gap more quickly than a 90° vertical setback).
- The distance between the pin holes on adjacent first course units should not exceed 1/4" from center. For best visual appearance, a maximum 1/2" gap is recommended.
- Drainage zone and backfill materials should be placed compacted and cup to the geogrid elevation and Keystone unit pins should be in place prior to geogrid installation.
- Measure, cut, and orient the geogrid, as per the engineered design and the geogrid manufacturer specifications on correct strength direction.
- Place the geogrid over the Keystone unit pins and tension the geogrid by pulling it back away from the wall. Place a stake through the geogrid at the back to tension the geogrid in place.
- Proceed with placement of additional Keystone units then unit drainage zone and backfill materials. Start by the wall and moving back away from the wall place the geogrid over the geogrid to hold the geogrid in place under tension. After the backfilling process the tension slacks may be removed for reuse.
- Compacting the backfill materials up to the next wall elevation where a geogrid is to be placed.

Limitations:
- 5' minimum radius for 3 Pano and Straight-Face Units.

Depending on wall height, radius and setback selection some gapping between units may occur. If gaps exceed acceptable limits, re-drill new pin holes as needed using a 5/8" minimum bit and reinstall units to close gaps.
**Installation Notes:**

- **Place base course units with a small gap between adjacent units.** This gap will close with the placement of each additional course of Keystone units as the units settle, move or settle back toward the point of radius. The rate of closure is controlled by the severity of the setback. The distance between the pin holes on adjacent first course units should not exceed 12" on center.

- **Additional crushed rock or stone drainage fill at outside wall curves to extend back from wall face each way at wall height/2 (H/2).**

- **Drainage zone and backfill materials should be placed compacted and up to the geogrid elevation and Keystone unit pins should be in place prior to geogrid installation.**

- **Measure, cut and orient the geogrid, as per the engineer’s design and the geogrid manufacturer’s specifications, on correct strength direction.**

- **Place the geogrid over the Keystone unit pins and tension the geogrid by pulling it back away from the wall. Place a stake through the geogrid at the back to tension the geogrid in place.**

- **Proceed with placement of additional Keystone units then unit drainage zone and backfill material. Start all the way down and moving back away from the wall plane the unit drainage zone and backfill materials over the geogrid to hold the geogrid in place under tension. After the backfilling process the tension stakes may be removed for reuse.**

- **Compacted the backfill materials up to the next wall elevation where a geogrid is to be placed.**

**Limitations:**

- 3" minimum radius for 3-Place or Straight foot units.

Depending on wall height, radius and setback selection some bridging between units may occur. If the previously proposed placement of additional units, try one of the following:

- Trim unit corners using a masonry cold chisel or concrete power saw.
- Push units back and realign. If gaps exceed acceptable limits, re-drill new pin holes as needed.

**Minimum gapping will be required with a near vertical setback.**
Appendix 15-R  Preapproved Wall Appendix: Specific Requirements and Details for KeyGrid Walls

Wall Section with 24" Pipe Parallel to the Wall Face

- Compac II Unit - Near Vertical Setback Shown
- Note is the same for 1" Setback

Wall Section with 48" Max. Pipe Perpendicular to the Wall Face

- Compac II Unit - Near Vertical Setback Shown
- Note is the same for 1" Setback
Appendix 15-R  Preapproved Wall Appendix: Specific Requirements and Details for KeyGrid Walls

Base Leveling Pad Notes:
1. Construct leveling pad 2500 psi unreinforced concrete.
2. The leveling pad foundation is to be approved by the site geotechnical engineer prior to leveling pad placement.

Leveling Pad and Wall Step Detail

Inside Corner Leveling Pad Detail

Outside Corner Leveling Pad Detail
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*Wu), 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:

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

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

(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.

$RF_{CR}$ and $RF_D$ shall be as provided in the WSDOT QPL, Appendix D.
### Table 15-S-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>$T_{ultconn}$ (lbs/ft)</th>
<th>$T_{lot}$ (lbs/ft)</th>
</tr>
</thead>
</table>
| GEOWALL Pro $W_u = 12$ in. | StrataGrid | SG200 | $\sigma_N < 1427$  
$\sigma_N > 1427$ | $\sigma_N \tan(38^\circ) + 756$  
$1892$ | 3724 |
| GEOWALL Pro $W_u = 12$ in. | StrataGrid | SG350 | $\sigma_N < 2967$  
$\sigma_N > 2967$ | $\sigma_N \tan(23^\circ) + 1077.5$  
$2353$ | 5211 |
| GEOWALL Pro $W_u = 12$ in. | StrataGrid | SG500 | $\sigma_N < 2983$  
$\sigma_N > 2983$ | $\sigma_N \tan(30^\circ) + 1060$  
$2814$ | 6751 |
| GEOWALL Pro $W_u = 12$ in. | StrataGrid | SG550 | $\sigma_N < 3100$  
$\sigma_N > 3100$ | $\sigma_N \tan(36^\circ) + 1076$  
$3328$ | 8247 |
| GEOWALL Pro $W_u = 12$ in. | StrataGrid | SG600 | $\sigma_N < 3000$  
$\sigma_N > 3000$ | $\sigma_N \tan(33^\circ) + 1252$  
$3200$ | 9553 |
| GEOWALL Pro $W_u = 12$ in. | Mirafi | 3XT | $\sigma_N < 1975$  
$\sigma_N > 1975$ | $\sigma_N \tan(29^\circ) + 1339$  
$3994$ | 5334 |
| GEOWALL Pro $W_u = 12$ in. | Mirafi | 5XT | $\sigma_N < 3062$  
$\sigma_N > 3062$ | $\sigma_N \tan(29^\circ) + 1339$  
$3994$ | 5334 |
| GEOWALL Pro $W_u = 12$ in. | Mirafi | 7XT | $\sigma_N < 2776$  
$\sigma_N > 2776$ | $\sigma_N \tan(35^\circ) + 1087$  
$3102$ | 6442 |
| GEOWALL Pro $W_u = 12$ in. | Mirafi | 8XT | $\sigma_N < 3100$  
$\sigma_N > 3100$ | $\sigma_N \tan(38^\circ) + 1178$  
$3600$ | 7898 |
| GEOWALL Pro $W_u = 12$ in. | Mirafi | 10XT | $\sigma_N < 3003$  
$\sigma_N > 3003$ | $\sigma_N \tan(36^\circ) + 1130$  
$3312$ | 10973 |
| GEOWALL Max $W_u = 21$ in | StrataGrid | SG200 | $1.75\sigma_N < 1643$  
$(1.75\sigma_N) > 1643$ | $(1.75\sigma_N) \tan(37^\circ) + 1246$  
$(1.75\sigma_N) + 2135$ | 3724 |
| GEOWALL Max $W_u = 21$ in | StrataGrid | SG350 | $1.75\sigma_N < 2777$  
$(1.75\sigma_N) > 2777$ | $(1.75\sigma_N) \tan(31^\circ) + 1471$  
$(1.75\sigma_N) + 2650$ | 5211 |
| GEOWALL Max $W_u = 21$ in | StrataGrid | SG500 | $1.75\sigma_N < 2674$  
$(1.75\sigma_N) > 2674$ | $(1.75\sigma_N) \tan(33^\circ) + 1605$  
$(1.75\sigma_N) + 2622$ | 6751 |
| GEOWALL Max $W_u = 21$ in | StrataGrid | SG550 | $1.75\sigma_N < 2796$  
$(1.75\sigma_N) > 2796$ | $(1.75\sigma_N) \tan(41^\circ) + 1580$  
$(1.75\sigma_N) + 2881$ | 8427 |
| GEOWALL Max $W_u = 21$ in | StrataGrid | SG600 | $1.75\sigma_N < 2799$  
$(1.75\sigma_N) > 2799$ | $(1.75\sigma_N) \tan(44^\circ) + 1768$  
$(1.75\sigma_N) + 3773$ | 9553 |
| GEOWALL Max $W_u = 21$ in | Mirafi | 3XT | $1.75\sigma_N < 1651$  
$(1.75\sigma_N) > 1651$ | $(1.75\sigma_N) \tan(45^\circ) + 1314$  
$(1.75\sigma_N) + 2821$ | 3994 |
| GEOWALL Max $W_u = 21$ in | Mirafi | 5XT | $1.75\sigma_N < 1941$  
$(1.75\sigma_N) > 1941$ | $(1.75\sigma_N) \tan(54^\circ) + 23$  
$(1.75\sigma_N) + 3921$ | 5334 |
| GEOWALL Max $W_u = 21$ in | Mirafi | 7XT | $1.75\sigma_N < 2700$  
$(1.75\sigma_N) > 2700$ | $(1.75\sigma_N) \tan(44^\circ) + 1611$  
$(1.75\sigma_N) + 3791$ | 6442 |
| GEOWALL Max $W_u = 21$ in | Mirafi | 8XT | $1.75\sigma_N < 2763$  
$(1.75\sigma_N) > 2763$ | $(1.75\sigma_N) \tan(52^\circ) + 1294$  
$(1.75\sigma_N) + 4193$ | 7898 |
| GEOWALL Max $W_u = 21$ in | Mirafi | 10XT | $1.75\sigma_N < 2226$  
$(1.75\sigma_N) > 2226$ | $(1.75\sigma_N) \tan(53^\circ) + 1240$  
$(1.75\sigma_N) + 3108$ | 10973 |
| GEOWALL Max II, $W_u = 18$ in | StrataGrid | SG200 | $1.5\sigma_N < 2700$  
$(1.5\sigma_N) > 2700$ | $(1.5\sigma_N) \tan(15^\circ) + 1540$  
$2260$ | 3724 |
Table 15-S-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>$T_{ultconn}$ (lbs/ft)</th>
<th>$T_{lot}$ (lbs/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GEOWALL Max II, $W_u = 18$ in</td>
<td>StrataGrid</td>
<td>SG350</td>
<td>$(1.5\sigma_N) &lt; 3600$</td>
<td>$(1.5\sigma_N) + 1650$</td>
<td>2680</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$(1.5\sigma_N) &gt; 3600$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GEOWALL Max II, $W_u = 18$ in</td>
<td>StrataGrid</td>
<td>SG500</td>
<td>$(1.5\sigma_N) &lt; 4500$</td>
<td>$(1.5\sigma_N) + 1570$</td>
<td>3570</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$(1.5\sigma_N) &gt; 4500$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GEOWALL Max II, $W_u = 18$ in</td>
<td>StrataGrid</td>
<td>SG600</td>
<td>$(1.5\sigma_N) &lt; 6300$</td>
<td>$(1.5\sigma_N) + 2125$</td>
<td>5200</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$(1.5\sigma_N) &gt; 6300$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. MSEW's input is in lb/ft² 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 $T_{ultconn}$ 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 $\frac{1}{8}$ 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.
- 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.
CONSTRUCTION DRAWINGS
Prepared for

STATE OF WASHINGTON
DEPARTMENT OF TRANSPORTATION

GEOWALL™ System

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.

GEOWALL Pro

GEOWALL Max II

GEOWALL Max

www.basalite.com
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

Foundation Soil
Concrete Leveling Pad

Retained Zone
Approximate Limits of Excavation

8" Min. Low Permeable Soil
Cap Unit

2" Unit fill as measured from the face of units

A back drain is suggested in a cut situation or where groundwater is present. For fill walls, the source of groundwater may not exit.
A back drain could be a geocomposite, 25 the slope height, 1/3 coverage on the slope.

8" GEOWALL™ Unit

8" step

Leveling Pad Detail

No.  Date  Revision  By

Scale:  As Noted

Title: GEOWALL Pro Details

Sheet No.  2 of 12

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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.
Geogrid is to be placed on level backfill over the fiberglass pins. Place next unit. Pull grid taut and backfill. Stake as required.
Geogrid is to be placed on level backfill over the fiberglass pins. Place next unit. Pull grid taut and backfill. Stake as required.

8" Min. Low Permeable Soil
Retained Zone
Approximate Limits of Excavation
Reinforced Soil
Foundation Soil
Concrete Leveling Pad

Typical Reinforced Wall Section
Near Vertical Setback

8" GEOWALL™ Unit
8" step

Leveling Pad Detail

Section A-A

UNIT FILL TO BE WSDOT STANDARD SPECIFICATION SECTION 8.03.92. CRUSHED SURFACING TOP COURSE
12" unit fill
Cap Unit
GEOWALL™ Max Unit

A back drain is suggested in a cut section or where groundwater is present. For fill areas, the source of ground water may not exist.
A back drain could be a geocomposite, 2/3 the slope height, 1/3 coverage on the sides.

0°-1/8°
8°
1/4" = 1'

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.
NOTE:
1. FOR PIPES LARGER THAN 24", A CONCRETE COLLAR MAY BE CAST AROUND PIPE FOR EASE OF CONSTRUCTION AND APPEARANCE.
   SAW CUT UNITS TO FIT WITHIN 1/8" OF PIPE.
   CONCRETE COLLAR (IF APPLICABLE)

SCOUR PROTECTION AS REQUIRED, USE RIP RAP OR CONCRETE SLAB IN OUTLET AREA.

Typical Pipe Outlet Detail
SCALE: N.T.S.

Angled Pipe Outlet Detail
SCALE: N.T.S.

For culverts oriented up to a 45 degree skew angle as measured from perpendicular to the wall face.

Pipe Penetration

CONCRETE COLLAR (IF APPLICABLE)

SCOUR PROTECTION AS REQUIRED, USE RIP RAP OR CONCRETE SLAB IN OUTLET AREA.
**Appendix 15-S Preapproved Wall Appendix: Specific Requirements and Details for Basalite GEOWALL**

**SECTION B-B**

- **0.6" x 0.6" CHAMFER (TYP.)**
- **GROUT TOP TWO ROWS OF GEOWALL UNITS**
  - **NOTE:** WHERE NEEDED, PROTECT UPPER GEOGRID LAYER FROM DIRECT CONTACT WITH POURRED CONCRETE. PLACE 4" OF UNIT FILL AGGREGATE IN THE BLOCK CORES AND THEN PLACE A GROUT STOP ON TOP OF THE AGGREGATE, THEN ADD GROUT.
  - **4 @ DOWEL LOCATIONS**
  - **8 @ BARS**
  - **SLIP RING LOCATIONS**
  - **3.64 @ 3.6 O.C.**

**SECTION A-A**

- **9.8'(TYP)**
- **EXP JOINT (TYP)**
- **0.8" EXP JOINT (TYP)**

**C.I.P. SIDE COPING ELEVATION**

**C.I.P. TOP COPING ELEVATION**

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Visit www.basalite.com for more information.
Geogrid Installation on Curves and Obtuse Corners

Minimum Radius: 4 ft (PRO); 6 ft (MAX, MAX II)

Place Additional Pieces of Geogrid When Angle Exceeds 20 degs.

Additional Drainage Fill Extend Wall Height 2

Obtuse Inside Corner

Drainage Fill

Obtuse Outside Corner

Geogrid Installation at 90 deg and Acute Corners

Maximum Outside Angle: 90° (zero ft)

3” of Soil Fill is Required Between Overlapping Geogrid for Proper Anchorage, (Typ.)

Additional Drainage Fill Extend Wall Height 2

90 deg or Acute Inside Corner

Overlap units in corner (see Details p 9 of 11)

90 deg Outside Corner

GEOWALL Corner Unit Alternate Corner Block Each Layer.
(see Details p. 9 of 10)

These details apply to the GEOWALL, GEOWALL MAX AND GEOWALL MAX II Units

GEОGRID 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.
Typical Post Detail

SCALE: N.T.S.
GEOTECHNICAL DESIGN MANUAL M:46.03.12
July 2019

Preapproved Wall Appendix: Specific Requirements and Details for Basalite GEOWALL Appendix 15-S

GEOWALL BARRIER 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.

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