CHAPTER 17

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17.1 INTRODUCTION AND DESIGN STANDARDS

Abutments for bridges have components of both foundation design and wall design. This chapter addresses the earth pressures acting on the abutments as well as retaining walls and reinforced slopes. 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.

Retaining walls and reinforced slopes have many benefits associated with their use. Unfortunately, there also 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 NYSDOT 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. All abutments, retaining walls, and reinforced slopes within NYSDOT Right-of-Way or whose construction is administered by NYSDOT shall be designed in accordance with the NYSDOT Geotechnical Design Manual (GDM) and the following documents:

- NYSDOT Bridge Manual
- NYSDOT Highway Design Manual
- AASHTO LRFD Bridge Design Specifications

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 supercede 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 NYSDOT GDM and the manuals and design specifications listed above:

17.2 OVERVIEW OF WALL CLASSIFICATIONS AND DESIGN PROCESS FOR WALLS

The various walls and wall systems can be categorized based on their intended functional life: permanent, temporary, and interim.

1. Permanent: A permanent system provides a structural support function for the life of the facility.
2. Temporary: A temporary system is designed to provide structural support during construction, and is removed when construction is complete.
3. Interim: An interim system is identical to a temporary system in function, except it remains in place (although it no longer provides a structural function) because its removal would be detrimental to the finished work.

The classification of retaining wall systems is based on the basic geotechnical mechanism used to resist lateral loads and the construction method used for the installation of the wall. The following are definitions used to classify retaining wall systems:

1. Externally Stabilized Structures: Externally stabilized structures rely on the integrity of wall elements (with or without braces, struts, walers and/or tiebacks or anchors) to both resist lateral loads and also prevent raveling or erosion of the retained soil.
2. Internally Stabilized Structures: Internally stabilized structures rely on friction developed between closely-spaced reinforcing elements and the backfill to resist lateral soil pressure. A separate, non-structural element (facing, erosion control mat and/or vegetation) is attached to prevent raveling or erosion of the retained soil.
3. Fill Type Retaining Walls: Retaining structures constructed from the base of the wall to the top (i.e. “bottom-up” construction).
4. Cut Type Retaining Walls: Retaining structures constructed from the top of the wall to the base (i.e. “top-down” construction).

An overview of the classification of retaining wall systems is provided in Table 17-1. The table provides a breakdown of available retaining wall systems, its associated method of construction, means of stability, design requirements and constraints (e.g. typical height range, maximum wall height).
<table>
<thead>
<tr>
<th>Wall Class</th>
<th>Wall Type</th>
<th>Construction Type</th>
<th>Wall Group</th>
<th>Design</th>
<th>Constraints</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchored Walls (Sheeting or</td>
<td>Anchored Walls (Sheeting or</td>
<td>Cut Wall</td>
<td>Deadman Anchors</td>
<td>Designed &amp; detailed in contract.</td>
<td>Typical Height Range: 15 ft. to 65 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Braced Walls</td>
<td>Designed &amp; detailed in contract.</td>
<td></td>
</tr>
<tr>
<td>Externally Stabilized</td>
<td>Cantilever Wall</td>
<td>Primarily Fill</td>
<td>Precast Cantilever Wall</td>
<td>Designed &amp; detailed in contract.</td>
<td>Typical Height Range: 6 ft. to 30 ft.</td>
</tr>
<tr>
<td>Fill Structures</td>
<td></td>
<td>Wall. May be</td>
<td>CIP Cantilever Wall</td>
<td>Designed &amp; detailed in contract.</td>
<td>Typical Height Range: 6 ft. to 30 ft. Maximum Wall Height = 30 ft.</td>
</tr>
<tr>
<td>Gravity Wall</td>
<td></td>
<td>Primarily Fill</td>
<td>Gabion</td>
<td>Designed &amp; detailed in contract.</td>
<td>Typical Height Range: 6 ft. to 20 ft. Maximum Wall Height = 20 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Wall. May be</td>
<td>CIP Mass Gravity</td>
<td>Designed &amp; detailed in contract.</td>
<td>Typical Height Range: 3 ft. to 10 ft. Maximum Wall Height = 23 ft.</td>
</tr>
<tr>
<td>Internally Stabilized</td>
<td>Fill Wall</td>
<td>Primarily Fill</td>
<td>Prefabricated Wall System (PWS)</td>
<td>Detailed in contract. Designed by Contractor’s Designer (Proprietary Wall).</td>
<td>Typical Height Range: 3 ft. to 50 ft.</td>
</tr>
<tr>
<td>Cut Structures</td>
<td></td>
<td>Wall. May be</td>
<td>Mechanically Stabilized Earth System (MSES)</td>
<td>Detailed in contract. Designed by Contractor’s Designer (Proprietary Wall).</td>
<td>Typical Height Range: 10 ft. to 65 ft.</td>
</tr>
<tr>
<td>Fill Type Retaining Wall</td>
<td>Fill Wall</td>
<td></td>
<td>Mechanically Stabilized Wall System (MSWS)</td>
<td>Detailed in contract. Designed by Contractor’s Designer (Proprietary Wall).</td>
<td>Typical Height Range: 6 ft. to 65 ft.</td>
</tr>
<tr>
<td>Geosynthetically</td>
<td>Geosynthetically Reinforced Soil</td>
<td>Fill Wall</td>
<td></td>
<td>Designed &amp; detailed in contract.</td>
<td>Typical Height Range: 6 ft. to 65 ft.</td>
</tr>
<tr>
<td>Reinforced Soil System (GRSS)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Internally Stabilized</td>
<td>Soil Nail Wall System</td>
<td>Cut Wall</td>
<td></td>
<td>Designed in contract. Designed by Contractor’s Design Consultant</td>
<td>Typical Height Range: 10 ft. to 65 ft.</td>
</tr>
<tr>
<td>Cut Structures</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 Cut wall construction refers to a wall system in which the wall is constructed from the top of the wall to the base (i.e., “top-down” construction). Fill wall construction refers to a wall system in which the wall is constructed from the base of the wall to the top (i.e., “bottom-up” construction).
A number of proprietary wall systems have been extensively reviewed by the Geotechnical Engineering Bureau, Materials Bureau and Office of Structures. This review has resulted in NYSDOT creating a Fill Type Retaining Wall Approved List. The design procedures and wall details for these preapproved wall systems shall be in accordance with this manual (NYSDOT GDM) and other manuals specifically referenced herein as applicable to the type of wall being designed, unless alternate design procedures have been agreed upon between NYSDOT and the proprietary wall manufacturer. In addition, Standard Sheet 554-01 Proprietary Fill Type Retaining Walls provides installation requirements. 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. Only the wall systems appearing on the Approved List are preapproved.

Standard cantilever wall designs are provided in the Highway Design Manual. The internal stability design and the external stability design for overturning and sliding stability have already been addressed in the wall design, and bearing resistance, settlement, and overall stability must be determined for each standard-design wall location by the Departmental Geotechnical Engineer.

All other walls are nonstandard, as standard designs have not been developed. 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 Departmental Geotechnical Engineer and, if structural design is required, by the Structural Designer. Geosynthetically Reinforced Soil System (GRSS) slopes are similar to nonstandard, nonproprietary walls in that the Departmental Geotechnical Engineer is responsible for the design, but the reinforcing may be a proprietary item.

17.3 REQUIRED INFORMATION

17.3.1 Site Data

NYSDOT Bridge Manual Section 3 discusses site data required for design.

17.3.2 Geotechnical Data Needed for Retaining Wall and Reinforced Slope Design

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

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

Table 17-2 provides a summary of information needs and testing considerations for retaining walls and reinforced slope design.
## CHAPTER 17
### Abutments, Retaining Walls, and Reinforced Slopes

<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/R33)  
• 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 |
| **Cut Walls** | • internal stability  
• external stability  
• excavation stability  
• global and compound stability  
• dewatering  
• chemical compatibility of wall/soil  
• lateral earth pressure  
• drop-drop on wall  
• pore pressures behind wall  
• obstructions in retained soil  
• liquefaction  
• see page  
• potential for subsidence (karst, mining, etc.)  
• constructability | • subsurface profile (soil, ground water, rock)  
• shear strength of soil  
• horizontal earth pressure coefficients  
• interface shear strength (soil and reinforcement)  
• hydraulic conductivity of soil  
• geologic mapping including orientation and characteristics of rock discontinuities  
• seismicity | • test cut to evaluate stand-up time  
• well pumping tests  
• piezometers  
• SPT  
• CPT  
• vane shear  
• dilatometer  
• pullout tests (anchors, nails)  
• geophysical testing | • triaxial tests  
• unconfined compression  
• direct shear  
• grain size distribution  
• Atterberg Limits  
• specific gravity  
• pH, resistivity tests  
• organic content  
• hydraulic conductivity  
• moisture content  
• unit weight |

### Table 17-2 Summary of Information Needs and Testing Considerations
NYSDOT GDM Chapter 6 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.

17.3.3 Site Reconnaissance

For each abutment, retaining wall, and reinforced slope, the Departmental Geotechnical Engineer should perform a site review and field reconnaissance. The Departmental Geotechnical Engineer 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 Project Designer. In addition, the Departmental Geotechnical Engineer should have access to detailed plan views showing existing site features, utilities, proposed construction, and right-of-way limits. With this information, the Departmental Geotechnical Engineer can review the wall/slope locations making sure that survey information agrees reasonably well with observed site topography. The Departmental Geotechnical Engineer should observe where utilities are located, as they will influence where field exploration can occur and they may affect design or constructability. The Departmental Geotechnical Engineer 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 Departmental Geotechnical Engineer 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 Departmental Geotechnical Engineer 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.

17.3.4 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 NYSDOT GEM Chapter 11.

Retaining walls and reinforced slopes that are equal to or less than 10 ft in exposed height as measured vertically from wall bottom to top or from slope toe to crest, as shown in Figure 17-1,
shall be investigated in accordance with this manual. For all retaining walls and reinforced slopes greater than 10 ft in exposed height, the field exploration shall be completed in accordance with the AASHTO LRFD Bridge Design Specifications and this manual.

Figure 17-1 Exposed Height (H) for a Retaining Wall or Slope

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

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

In areas underlain by heterogeneous soil deposits and/or rock formations, it will probably be necessary to perform more investigation to capture variations in soil and/or rock type and to assess consistency across the site area. In a laterally homogeneous area, drilling or advancing a large number of borings may be redundant, since each sample tested would exhibit similar engineering properties. In all cases, it is necessary to understand how the design and construction of the geotechnical feature will affect the soil and/or rock mass in order to optimize the exploration.

17.3.4.1 Exploration Type, Depth, and Spacing

Guidelines on the number and depth of borings are presented in NYSDOT Chapter 4 Table 4-3 and 4-4. While engineering judgment will need to be applied by the Departmental Geotechnical Engineer (or a licensed and experienced geotechnical professional) to adapt the exploration
program to the foundation types and depths needed and to the variability in the subsurface conditions observed, the intent of Table 4-3 and 4-4 regarding the minimum level of exploration needed should be carried out.

17.3.4.2 Walls and Slopes Requiring Additional Exploration

17.3.4.2.1 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 ft in length, exploration points should in general be spaced at 100 to 200 ft, but may be spaced at up to 500 ft in uniform, dense soil conditions. Even closer spacing than 100 to 200 ft 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 see page. 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 ft would have a test pit 8 ft deep and at least 12 ft in length at the bottom of the pit.

17.3.4.2.2 Walls With Ground Anchors or Deadmen 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 ft in length, exploration points should in general be spaced at 100 to 200 ft, but may be spaced at up to 500 ft in uniform, dense soil conditions. Even closer spacing than 100 to 200 ft 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 anchors or deadmen are likely to be installed, and deep enough to address overall stability issues.

17.3.4.2.3 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 ft 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 ft for hand holes and 20 ft for test pits.

17.3.5 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, prefabricated wall systems 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.

The collection of soil samples and field testing shall be in accordance with NYSDOT GDM Chapters 4, 5, and 6.

17.3.6 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 NYSDOT GDM Chapter 23, 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.

17.3.7 Wall Backfill

One of the primary components of all fill walls is the backfill soil. NYSDOT Standard Specifications provide requirements for Select Structural Fill. This granular material is used for backfill for most retaining wall systems as it has been developed for its strength and free-draining properties.

Prefabricated wall systems which utilize open top units incorporate select structural fill as unit infill and backfill material. Unit infill is the granular material placed within the bin (any volumetric space which is designated to be infilled and is encompassed within the dimensions of the open top unit), such as the open structure of an open top face unit or contiguous to the bevel
sides of a solid face unit. The backfill material is the granular material placed directly behind and/or above the bins in conjunction with the wall assembly.

Mechanically stabilized earth systems or mechanically stabilized wall systems incorporate a specialized backfill material to ensure long-term performance of the embedded reinforcing elements. See Section 17.5.3.2 for discussion regarding MSES and MSWS backfill material.

Other typical soil design properties for various types of backfill and native soil units are provided in NYSDOT GDM Chapter 6.

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 17-2 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.

17.4 GENERAL DESIGN REQUIREMENTS

17.4.1 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).

Acceptable design methodology and theory for the geotechnical design of flexible cantilevered or anchored retaining walls is provided in Geotechnical Design Procedure (GDP-11) Geotechnical Design Procedure for Flexible Wall Systems.

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

Specific design requirements for tiered walls, back-to-back walls, and MSES wall supported abutments are provided in the NYSDOT 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).

17.4.2 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. 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.

17.4.3 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 System (MSES) walls should be 1.1 times the average height of the MSES walls or greater. Back-to-back MSES walls with a width/height ratio of less than 1.1 shall not be used unless approved by the Geotechnical Engineering Bureau and Office of Structures. The maximum height for back-to-back MSES wall installations (i.e., average of the maximum heights of the two parallel walls) is 30 ft, again, unless a greater height is approved by the Geotechnical Engineering Bureau and the Office of Structures. Justification to be submitted to the Geotechnical Engineering Bureau and the Office of Structures 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 MSES walls may be connected to both faces, i.e., continuous from one wall to the other, provided the reinforcing is designed for at least double the
loading, if approved or required by the Departmental 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 ft are generally not desirable due to the increased cost of materials. 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.

17.4.4 Walls on Slopes

Standard Plan walls founded on slopes shall meet the requirements in the Highway Design Manual. All other walls shall have a near horizontal bench at the wall face at least 4 ft 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.

For soldier pile and lagging walls on slopes, see Section 17.5.2.5.

17.4.5 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 Departmental Geotechnical Engineer (see also NYSDOT GDM Section 17.4.4). 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 1V on 6H or flatter. Stepped foundations shall be 1V on 1.5H 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 MSES walls shall be sloped at 1V on 6H or flatter or stepped at 1V on 1.5H or flatter determined by a line through the corners of the steps. As MSES 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., MSES 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 ft, unless a greater depth is otherwise specified, shall be used.

### 17.4.6 Wall Height Limitations

Proprietary fill type retaining wall systems that are preapproved through the NYSDOT have limiting heights for unreinforced applications as noted on the Approved List. In addition, the Approved List identifies whether or not the particular proprietary fill type retaining wall system is acceptable in abutment load bearing applications.

Wall height limitations for other retaining walls are provided in Table 17-1.

### 17.4.7 Engineering Considerations

<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Relative Construction Time</th>
<th>Sensitivity to Differential Settlement</th>
<th>Approx. Base Width</th>
<th>Potential Settlement of Retained Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast-In-Place Concrete</td>
<td>Long</td>
<td>High</td>
<td>0.5 x wall height</td>
<td>Medium</td>
</tr>
<tr>
<td>Prefabricated Wall System</td>
<td>Short</td>
<td>Low</td>
<td>0.5 x wall height</td>
<td>Medium to High</td>
</tr>
<tr>
<td>Sheet Piling or Soldier Pile &amp; Lagging (cantilever)</td>
<td>Short</td>
<td>Low (High (1))</td>
<td>&lt; 3 ft.</td>
<td>High</td>
</tr>
<tr>
<td>Sheet Piling or Soldier Pile &amp; Lagging (tied-back)</td>
<td>Medium</td>
<td>High</td>
<td>&lt; 5 ft. (2)</td>
<td>Low</td>
</tr>
<tr>
<td>Gabion Wall</td>
<td>Medium</td>
<td>Low</td>
<td>0.5 x wall height</td>
<td>High</td>
</tr>
<tr>
<td>Geotextile Wall</td>
<td>Medium</td>
<td>Low (Medium)</td>
<td>0.5 x wall height</td>
<td>Medium to High</td>
</tr>
</tbody>
</table>

(1) with cast-in-place facing  
(2) easement needed for tiebacks if they extend outside of right-of-way

Table 17-3 Engineering Considerations

- **Relative Construction Time:**
  The present-day construction program consists almost entirely of the rehabilitation and/or upgrading of existing facilities. Construction time may be an important consideration in order to reduce the duration on inconvenience to the traveling public. Cast-In-Place concrete walls require the longest construction time since forms have to be erected and the concrete poured and permitted to gain strength before backfill loads can be applied to
the wall. In contrast, the concrete facing for soldier pile and lagging and sheet pile walls can by constructed while the walls support their design lateral loads.

- **Sensitivity to Differential Settlement:**
  If retaining walls sensitive to differential settlement are to be used in an area of compressible soils, special treatment, such as deep foundations, foundation re-loading or foundation densification, all of which involve an increase in expense and construction time, will be required.

- **Approximate Base Width:**
  An important consideration on urban projects is the width of the work during construction.

- **Potential Settlement of the Retained Mass:**
  In the case of cuts made adjacent to existing facilities, the potential settlement of the earth mass retaining by the wall and supporting these facilities has to be taken into account.

**17.4.8 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 17-4, 17-5 and 17-6) shall be used to establish acceptable settlement criteria:

<table>
<thead>
<tr>
<th>Total Settlement</th>
<th>Differential Settlement over 100 ft.</th>
<th>Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta H \leq 1$ in.</td>
<td>$\Delta H_{100} \leq 0.75$ in.</td>
<td>Design and Construct</td>
</tr>
<tr>
<td>$1$ in. $&lt; \Delta H \leq 2.5$ in.</td>
<td>$0.75$ in. $&lt; \Delta H_{100} \leq 2$ in.</td>
<td>Ensure structure can tolerate settlement</td>
</tr>
<tr>
<td>$\Delta H &gt; 2.5$ in.</td>
<td>$\Delta H_{100} &gt; 2$ in.</td>
<td>Obtain Approval(^1) prior to proceeding with design and construction.</td>
</tr>
</tbody>
</table>

\(^1\) Approval of Geotechnical Engineering Bureau and Office of Structures required.

**Table 17-4 Settlement Criteria for Reinforced Concrete Walls, Nongravity Cantilever Walls, Anchored/Braced Walls, and MSES Walls with Full height Precast Concrete Panels (Soil is Placed Directly Against Panel)**
### Table 17-5 Settlement Criteria for MSWS, Prefabricated Walls, and Rock Walls

<table>
<thead>
<tr>
<th>Total Settlement</th>
<th>Differential Settlement over 100 ft.</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 Geotechnical Engineering Bureau and Office of Structures required.

### Table 17-6 Settlement Criteria for MSES with Flexible Facings and Reinforced Slopes

<table>
<thead>
<tr>
<th>Total Settlement</th>
<th>Differential Settlement over 100 ft.</th>
<th>Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>ΔH ≤ 4 in.</td>
<td>ΔH_{100} ≤ 3 in.</td>
<td>Design and Construct</td>
</tr>
<tr>
<td>4 in. &lt; ΔH ≤ 12 in.</td>
<td>3 in. &lt; ΔH_{100} ≤ 9 in.</td>
<td>Ensure structure can tolerate settlement</td>
</tr>
<tr>
<td>ΔH &gt; 12 in.</td>
<td>ΔH_{100} &gt; 9 in.</td>
<td>Obtain Approval(^1) prior to proceeding with design and construction.</td>
</tr>
</tbody>
</table>

\(^1\) Approval of Geotechnical Engineering Bureau and Office of Structures required.

### 17.4.9 Active, Passive, At-Rest Earth Pressures

The Departmental Geotechnical Engineer 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.

The magnitude of earth pressure depends upon the physical properties of the soil, the interaction at the soil-structure interface and the magnitude and character of the deformations in the soil-structure system. Earth pressure is also influenced by the time-dependent nature of soil strength, which varies due to creep effects and chemical changes in the soil.

There are two well known classical earth pressure theories, the Rankine Theory and the Coulomb Theory. Each furnishes expressions for active pressures for a soil mass at the state of failure.
CHAPTER 17
Abutments, Retaining Walls, and Reinforced Slopes

Rankine Theory
The Rankine Theory is based on the assumptions that the wall introduces no changes in the shearing stresses at the surface of contact between the wall and the soil. It is also assumed that the ground surface is a straight line (horizontal or sloping surface) and that a plane failure surface develops.

Coulomb Theory
An inherent assumption of the Rankine Theory is that the presence of the wall does not affect the shearing stresses at the surface of wall contact. However, since the friction between the retaining wall and the soil has a significant effect on the vertical shear stresses in the soil, the lateral stresses on the wall are actually different than those assumed by the Rankine Theory. Most of this error can be avoided by using the Coulomb Theory, which considers the changes in tangential stress along the contact surface due to wall friction.

Logarithmic Spiral
A logarithmic spiral failure surface may be used to determine the active and passive pressures against retaining structures when interface friction acts along the back of the wall.

Rankine’s theory, Coulomb’s wedge theory, and the logarithmic spiral procedure result in similar values for active and passive thrust when the interface friction between the wall and the backfill is equal to zero. For interface friction angles greater than zero, the wedge method and the logarithmic procedure results in accurate values for passive thrust for all values of interface friction between the wall and the backfill. The accuracy of the passive thrust values computed using the wedge method diminishes with increasing values of interface friction because the boundary of the failure block becomes increasingly curved.

Accuracy of Coulomb’s Theory for Passive Earth Pressure Coefficients
Provides reasonable estimates for \( K_p \) and the orientation of the slop plane, \( \delta \), so as \( \delta \) is restricted to values which are less than \( \varphi/2 \). Coulomb’s relationship overestimates the value for \( K_p \) when \( \delta \) is greater than \( \varphi/2 \). The large shear component of \( P_p \) introduces significant curvature in the failure surface. The Coulomb procedure, however, restricts the theoretical slip surface to a plane. When \( \delta \) is greater than \( \varphi/2 \), the value for \( K_p \) must be computed using a method of analysis which use a curved failure surface to obtain valid values. Figures 17-3 and 17-4 shown the variation in the values for \( K_p \) with friction angles, computed using Coulomb’s equation for \( K_p \) based on a planer failure surface versus a log spiral failure surface analysis.
Figure 17-3 Active and Passive Earth Pressure Coefficients with Wall Friction – Sloping Backfill  
(NACFAC DM-7.2, 1982)
Figure 17-4 Active and Passive Earth Pressure Coefficients with Wall Friction – Sloping Wall
(NACFAC DM-7.2, 1982)
Earth pressures may be based on either Coulomb or Rankine theories. However, for construction submittals for alternate designs proposed by the Contractor, the analysis shall be in accordance with Geotechnical Design Procedure (GDP-11) Geotechnical Design Procedure for Flexible Wall Systems.

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

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

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

**Earth Pressures Computed Using the Trial Wedge Procedure**

The trial wedge procedure of analysis is used to calculate the earth pressure forces acting on walls when the backfill supports point loads or loads of finite width or when there is seepage within the backfill. The procedure involves the solution of the equations of equilibrium for a series of trial wedges within the backfill for the resulting earth pressure force on the back of the wall. When applying this procedure to active earth pressure problems, the shear strength along the trial slop plane is assumed to be fully mobilized. The active earth pressure force is equal to the largest value for the earth pressure force acting on the wall obtained from the series of trial wedge solutions.
To understand the trial wedge method, look at the free-body diagram of a typical wedge:

![Free-body diagram of a typical wedge](image)

**Figure 17-5 Trial Wedge Method: Free-Body Diagram**

On any wedge, the forces acting on it are:

- **W** = The weight of the wedge, which acts vertically downwards.
- **F** = The force of the wall against the wedge, this acts at an angle of δ to the normal.
- **R** = The force of the soil against the wedge, this acts at an angle of φ to the normal.

δ is the angle of soil-wall friction, and φ is the soil’s angle of internal friction.

When solving a problem, look at the equilibrium of several wedges behind the wall and then attempt to close the respective force triangles graphically. For example, assume a wall with a homogeneous backfill as shown in Figure 17-6. If a series of wedges are drawn, the force triangles for each wedge would look like Figure 17-7. If a plot of force, F, versus the angle of wedge makes with the horizontal, β, is done, the maximum value on the curve is the maximum force against the wall, shown in Figure 17-8.
Figure 17-6 Series of Wedges in Homogeneous Backfill

Figure 17-7 Force Triangles

Figure 17-8 Maximum Force Against The Wall
Finding the actual point of application of the resultant force is complicated. In theory, the sum of the moments acting on the critical wedge, and adjust the point of application of the wall force to achieve equilibrium. In practice, assume that the force acts at the point where the back of the wall intersects a line drawn parallel to the failure surface and through the center of gravity of the wedge.

![Diagram of retaining wall with forces and moments](image)

**Figure 17-9 Identifying Point of Application**

For Coulomb’s conditions, the force will act at the angle of soil-wall friction, $\delta$. For Rankine’s conditions, the force will act parallel to the ground’s surface. If the ground surface is uneven and Rankine’s Theory is used, the American Railway Engineering Association (AREA) says to draw a line from the back of the wall to a point on the ground surface that is twice the wall height behind the wall, and assume that the force acts parallel to this line (Huntington).

**Procedure**

Calculation of active pressure:

1. Draw wedges through the backfill.
2. Calculate the weight of each wedge. Include any surcharge on the wedge as part of the weight. If part of the surcharge on the backfill is a line load, draw the wedges such that the load is on the boundary between two wedges.
3. For each wedge, calculate $\beta$, the angle that the wedge makes with the horizontal and the value of $\beta-\phi$.
4. Draw the weight vectors co-linearly to a convenient scale.
5. For each wedge, sketch a line from the beginning of the weight vector at an angle corresponding to its value of $\beta - \phi$ to the vector.

6. At each weight value on the weight vector, sketch a line upwards at an angle of $90 - \theta - \delta$ to the vector. $\delta$ is the angle of wall friction and $\theta$ is the angle that the back of the wall makes with the vertical. Draw the line to intersect its corresponding line from Step 5.

7. Draw a smooth curve connecting the intersecting points of the lines in Steps 5 and 6. Line loads and strip loads will have discontinuities in the curve.

8. Draw a line parallel to the weight vector that is tangent to the highest point on the curve.

9. Draw a line at an angle of $90 - \theta - \delta$ connecting the tangent point and the weight vector. The length of this line, measure at the same scale as the weight vector, is the active force on the wall.

![Graphical Procedure](image)

**Figure 17-10 Graphical Procedure**

Finding the point of application:

A. Draw a line that connects the beginning of the weight vector with the intersection of the line from Step 9 above and the tangent to the highest point on the curve.

B. The angle that the line in Step A makes with the weight vector is the value of $\beta - \phi$ for the failure wedge.

C. Draw the failure surface at B to the horizontal, and calculate the center of gravity for the failure wedge.

D. Draw a line parallel to the failure surface through the center of gravity.

E. The active earth pressure acts at the point where the line from Step D intersects the back of the wall surface.
17.4.10 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 1V on 4H 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.

Abutments shall be detailed in accordance with the following Bridge Detail sheets:
- BD-EE-8ER1 Abutments with Spread Footings on Fill/ Surcharge Loading and Waiting Period.
- BD-EE-9ER1 Abutments with Spread Footings on Fill with Waiting Period.

17.4.11 Seismic Earth Pressures

For all walls and abutments, the Mononobe-Okabe method described in the AASHTO LRFD Bridge Design Specifications, Chapter 11 and Appendix A11.1.1.1, should be used. In addition, for this approach it is assumed that the wall backfill is completely drained and cohesionless (i.e., not susceptible to liquefaction).

Walls and abutments that are free to translate or move during a seismic event may use a reduced horizontal acceleration coefficient $k_h$ of approximately one-half effective peak ground acceleration coefficient $A_s$. Vertical acceleration coefficient, $k_v$, should be set equal to 0.

Walls and abutments that are not free to translate or move during a seismic event shall use a horizontal acceleration coefficient of 1.5 times effective peak ground acceleration coefficient, $A_s$. Vertical acceleration coefficient should be set equal to 0.

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 Article C11.6.5 of the AASHTO LRFD Bridge Design Specifications, or by using a Newmark time history analysis (see NYSDOT GDM Chapter 9) to calculate a yield acceleration that corresponds to the amount of horizontal wall displacement allowed. The reduced (yield) acceleration, as described above, should be calculated 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.
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.

The current AASHTO specifications are not consistent regarding the location of the resultant of the earth pressure when seismic loading occurs, nor are they consistent regarding the separation of the static earth pressure from the seismic earth pressure (i.e., the use of $\Delta K_{ae}$ to represent the seismic portion of the earth pressure versus the use of $K_{ae}$ to represent the total of the seismic and static earth pressure). Until this issue is resolved, the following policy shall be implemented regarding seismic earth pressure calculation:

- The seismic “component” of the Mononobe-Okabe earth pressure may be separated from the static earth pressure acting on the wall as shown in Article 11.10.7.1 in the AASHTO LRFD Bridge Design Specifications. If this is done, the seismic component, $\Delta K_{ae}$, shall be calculated as $K_{ae} - K_a$ for walls that are free to move and develop active earth pressure conditions, and as $K_{ae} - K_0$ for walls that are not free to move (i.e., at rest earth pressure conditions prevail, and $K_{ae}$ is calculated using a horizontal acceleration coefficient of 1.5 times the effective peak ground acceleration coefficient). Note that in this case, to complete the seismic design of the wall, the static earth pressure resulting from $K_{ae}$ or $K_0$ must be added to the seismic component of the earth pressure resulting from $\Delta K_{ae}$ to obtain the total earth pressure acting in the extreme event limit state. The load factor for EQ in Section 3 of the AASHTO LRFD Bridge Design Specifications (i.e., a load factor of 1.0) shall be applied to the static and seismic earth pressure loads, since in Mononobe-Okabe earth pressure analysis, a total static plus seismic earth pressure is calculated as one force initially, and then separated into the static and seismic components as a second step.

- The resultant force of the Mononobe-Okabe earth pressure distribution, as represented by $\Delta K_{ae}$ should be applied at 0.6H from the bottom of the pressure distribution. Note that the distribution is an inverted trapezoid if the resultant is applied at 0.6H, with the pressure at the top of the distribution equal to $0.8\Delta K_{ae}\gamma H$, and the pressure at the bottom equal to $0.2\Delta K_{ae}\gamma H$.

- If the seismic earth pressure force is calculated and distributed as a single force as specified in Appendix A11.1.1.1 of the AASHTO LRFD Bridge Design Specifications, the combined earth pressure force shall be applied at 0.5H from the bottom of the pressure distribution, resulting in a uniform pressure distribution in which the pressure is equal to $0.5K_{ae}\gamma H$. Note that since this uniform pressure distribution includes both the static and seismic component of lateral earth pressure, this uniform earth pressure must
not be added to the earth pressure resulting from $K_a$ or $K_0$. Note that this is the preferred approach to estimating earth pressures for the Extreme Event I (seismic) limit state.

- For all walls, the pressure distribution should be applied from the bottom of wall to the top of wall except cantilever walls, anchored walls, or braced walls. For these walls, the pressure should be applied from the top of wall to the elevation of finished ground line at the face of wall.

The Mononobe-Okabe seismic earth pressure theory was developed for a single layer cohesionless soil with no water present. For most gravity walls, this assumption is applicable in most cases. However, for cut walls such as anchored walls or non-gravity cantilever walls, it is possible and even likely that these assumptions may not be applicable. In such cases where these assumptions are not fully applicable, a weighted average (weighted based on the thickness of each layer) of the soil properties (e.g., effective stress $\phi$ and $\gamma$) should be used to calculate $K_{ae}$. Only the soil above the dredge line or finished grade in front of the wall should be included in the weighted average. If water behind the wall cannot be fully drained, the lateral pressure due to the difference in head must be added to the pressure resulting from $K_{ae}$ to obtain the total lateral force acting in the Extreme Event I limit state (note $K_{ae}$ includes the total of seismic and active earth pressure, as described previously).

As an alternative to the Mononobe-Okabe method, especially for those cases where the Mononobe-Okabe method is not applicable, limit equilibrium slope stability analysis may be used to estimate the total force (static plus seismic) behind the wall, using $k_h$ (the acceleration coefficient used to calculate $K_{ae}$) to include seismic force in the slope stability analysis (Chugh, 1995). Steps to accomplish this are as follows:

1. Set up slope/wall model geometry, soil properties, and ground water as would normally be done when conducting a slope stability analysis. The internal face of the wall should be modeled as a free boundary.
2. Select an appropriate slope stability analysis method. Spencer’s method is preferred because it satisfies equilibrium of forces and moments, but other analysis methods may be used, subject to approval by the NYSDOT Departmental Geotechnical Engineer.
3. Be sure that the failure surface search parameters are appropriate for the site and subsurface geometry so that the most critical surface is obtained.
4. Apply the earth pressure to be calculated as a boundary force on the face of the wall. In general, this force should be applied at a resultant location of 0.5 $H$ on the boundary, though the resultant location can be adjusted up or down to investigate the sensitivity of the location of the force, if desired. The angle of the applied force depends on the friction angle between the wall and the soil. An assumption of 0 to 0.67$\phi$ below the horizontal is typical, though a value up to $\phi$ may be used if the wall/ backfill soil interface is very rough.
5. Adjust the magnitude of the applied load until the calculated safety factor is 1.0. The force determined in this manner can be assumed to be equal to the total earth pressure acting on the wall during seismic loading.
If cohesive soils are present behind the wall, the residual drained friction angle rather than the peak friction angle (see NYSDOT GDM Chapter 6) should be used to determine the seismic lateral earth pressure. For anchored walls, since an empirically based Apparent Earth Pressure (AEP) based on the active, or in some cases at rest, earth pressure coefficient is used for static design, $K_{ae}$ should replace $K_a$ or $K_0$ in the AEP for seismic design. Note also that the slope of the active failure plane flattens as the earthquake acceleration increases.

For anchored walls, the anchors should 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.

Since the load factor used for the seismic lateral earth pressure for EQ is currently 1.0, to obtain the same level of safety for sliding and bearing obtained from the AASHTO Standard Specification design requirements, a resistance factor of slightly less than 1.0 is required. For bearing resistance during seismic loading, a resistance factor of 0.9 should be used.

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 should 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.

### 17.4.12 Liquefaction

Under extreme event loading, liquefaction and lateral spreading may occur. The Departmental Geotechnical Engineer 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 NYSDOT GDM Chapter 9.

### 17.4.13 Overall Stability

All retaining walls and reinforced slopes shall have a resistance factor for overall stability of 0.75 (i.e., a safety factor of 1.3 as calculated using a limit equilibrium slope stability method). This resistance factor is not to be applied directly to the soil properties used to assess this mode of failure. All abutments and those retaining walls and reinforced slopes deemed critical shall have a resistance factor of 0.65 (i.e., a safety factor of 1.5). Critical walls and slopes are those that support important structures like bridges and other retaining walls. Critical walls and slopes would also be those whose failure would result in a life threatening safety hazard for the public, or whose failure and subsequent replacement or repair would be an intolerable financial burden to the citizens of New York State. See NYSDOT GDM Chapter 11 for additional background and guidance regarding the assessment of overall stability.
It is important to check overall stability for surfaces that include the wall mass, as well as surfaces that check for stability of the soil below the wall, if the wall is located well above the toe of the slope. If the slope below the wall is determined to be potentially unstable, the wall stability should be evaluated assuming that the unstable slope material has moved away from the toe of the wall, if the slope below the wall is not stabilized. The slope above the wall, if one is present, should also be checked for overall stability.

Stability shall be assessed using limiting equilibrium methods in accordance with NYSDOT GDM Chapter 10.

**17.4.14 Wall Drainage**

Drainage should be provided for all walls. In instances where wall drainage cannot be provided, the hydrostatic pressure from the water shall be included in the design of the wall.

In general, wall drainage shall be in accordance with Standard Sheet 554-01 *Proprietary Fill Type Retaining Walls* and the Highway Design Manual.

Some examples of addressing drainage other than that shown on the Standard Sheets include:

- 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.

**17.4.15 Utilities**

Walls that have or may have future utilities in the backfill should minimize the use of soil reinforcement. MSEs, 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.

**17.4.16 Guardrail and Barrier**

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

For walls with a traffic barrier, the distribution of the applied impact load to the wall top shall be as described in the AASHTO LRFD Bridge Design Specifications Article 11.10.10.2 for LRFD designs unless otherwise specified in the NYSDOT Bridge 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 17-11 for an illustration of soil reinforcement load distributions. 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.
Figure 17-11 MSES Wall Soil Reinforcement Design for Traffic Barrier Impact for TL-3 and TL-4 loading (after Bligh, et al., 2010)
17.5 WALL TYPE SPECIFIC DESIGN REQUIREMENTS

17.5.1 Abutments

Abutment foundations shall be designed in accordance with NYSDOT GDM Chapter 11. 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 NYSDOT GDM Section 17.4.8).

Abutments shall be detailed in accordance with the following Bridge Detail sheets:
- BD-AB-1ER1 Abutment Plan & Elevation For U-Wingwall Structure (For Skews Under 30 Degrees)
- BD-AB-2ER1 Abutment Plan & Elevation For U-Wingwall Structure (For Skews 30 Degrees And Over)
- BD-AB-3ER1 Abutment Plan & Elevation With Flared And In-line Wingwalls
- BD-AB-4ER1 Example Of Pile Layout And Footing Reinforcement Plan
- BD-AB-5ER1 Reinforcement Plans For Stem, Backwall, Header And Pedestal Details
- BD-AB-6ER1 Abutment Sections And Concrete Keyway Details (Pile Foundation Shown)
- BD-AB-7ER1 Wingwall Sections (Pile Foundation Shown)
- BD-AB-8ER1 Miscellaneous Abutment Details
- BD-AB-9ER1 Stone Veneer Panel Details

Integral abutment types shall be detailed in accordance with the following Bridge Detail sheets:
- BD-ID-1ER1 Integral Abutments Adjacent PC Beams Typical Sections & Details
- BD-ID-1AE Example of Integral Abutments Adjacent PC Beams - Plan, Elevation & Pile Layout
- BD-ID-1BE Example of Reinforcement Details for Integral Abutments Adjacent PC Beams
- BD-ID-2ER1 Integral Abutments Typical Sections & Details
- BD-ID-2AE Example of Integral Abutments with Steel Girders - Plan & Elevation
- BD-ID-2BE Example of Reinforcement Details for Integral Abutments with Steel Girders
- BD-ID-3ER1 Integral Abutments Miscellaneous Details
- BD-ID-4ER1 Integral Abutments Alternate Steel Typical Section & Details
- BD-ID-5ER1 Integral Abutments PCEF & AASHTO I-Beams Typical Section & Details


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- BD-ID-6ER1  Semi-Integral Abutments Typical Sections & Details
- BD-ID-7ER1  Semi-Integral Abutment Plan & Sections

Piers shall be detailed in accordance with the following Bridge Detail sheets:
- BD-PR-1ER1  Solid Pier
- BD BD-PR-2ER1  Solid Pier Pile Layout, Footing And Pedestal Reinforcement
- BD BD-PR-3ER1  Column Pier On Continuous Spread Footing (1 of 2)
- BD BD-PR-3AE  Column Pier On Continuous Spread Footing (2 of 2)
- BD BD-PR-4ER1  Hammerhead Pier (1 of 2)
- BD BD-PR-4AE  Hammerhead Pier (2 of 2)
- BD BD-PR-5ER1  Multi-Column Pier (Rectangular Columns On Individual Footings) (1 of 2)
- BD BD-PR-5AE  Multi-Column Pier (Rectangular Columns On Individual Footings) (2 of 2)
- BD BD-PR-6ER1  Multi-Column Pier (Round Columns) Plan, Elevation And Sections
- BD BD-PR-7ER1  Multi-Column Pier (Round Columns) Footing And Other Details

Excavation and embankments shall be detailed in accordance with the following Bridge Detail sheets:
- BD-EE-8ER1  Abutments with Spread Footings on Fill/ Surcharge Loading and Waiting Period.
- BD-EE-9ER1  Abutments with Spread Footings on Fill with Waiting Period.
- BD-EE-14E  Excavation and Embankment Sample Drawing
- BD-EE-15E  Excavation and Embankment Sample Drawing of Stage Construction (1 of 2)
- BD-EE-16E  Excavation and Embankment Sample Drawing of Stage Construction (2 of 2)

17.5.1.1 Integral Abutments

An integral abutment bridge is based on the theory that due to the flexibility of the piling, thermal stresses are transferred to the substructure by way of a rigid connection between the superstructure and substructure. The concrete abutment contains sufficient bulk to be considered a rigid mass. A positive connection with the ends of the beams or girders is provided by encasement in reinforced concrete. This provides for full transfer of temperature variation and live load rotational displacement to the abutment piling. See NYSDOT Bridge Manual Chapter 11.

The main consideration that determines the various criteria which should be applied is the overall span length. The controlling factors will be listed and expanded upon with qualifications being applied to each based upon the span length. A further list of controlling factors is presented in the Polity and Guidelines on Use of Integral Abutments.
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Length
By NYSDOT definition, the length of an integral abutment structure shall be equal to the abutment centerline of bearing to abutment centerline of bearing dimension.
- 100 ft. or less: No provision for expansion at the ends of approach slabs will be required unless the highway pavement is rigid concrete.
- 100 ft. up to 300 ft.: Provision shall be made for expansion at the ends of approach slabs. If at all possible, the span arrangement and interior bearing selections shall be such that approximately equal movement will occur at each abutment.
- Over 300 ft. up to 400 ft.: Lengths in this range shall be approved on an individual basis. Provision for expansion at the ends of approach slabs will be required.
- Over 400 ft.: Not recommended at this time.

Foundation Type
All integral abutments shall be supported on piles. Steel bearing H-piles or cast-in-place piles may be used for structure with span lengths of 165 ft. or less. Only steel bearing H-piles shall be used for structures with span lengths over 165 ft. Foundations shall be designed in accordance with NYSDOT GDM Chapter 11.

All piles shall be installed in one single line. When steel bearing H-piles are used the web of the piles shall be perpendicular to the centerline of the beams regardless of the skew, so that bending takes place about the weak axis of the pile. For total bridge length of 245 ft. or greater, the designer shall investigate orienting the piles for strong-axis bending when the total lateral displacement causes buckling of the pile flanges.

All piles should have sufficient depth of penetration, 20 ft. minimum, into acceptable soil layers. The purpose of this is to avoid a stilt-type effect, provide for scour protection and to provide sufficient lateral support to the pile, particularly since the top 8 ft. is augered and backfilled with sand.

The cast-in-place or steel bearing pile at each abutment shall be inserted in preaugered holes. These holes shall extend to a depth of 8 ft. below the bottom of the stem of each abutment and are a minimum of 20 in. in diameter allowing a clearance of 2 in. around the pile.
Design Procedure for Substructure

1. Horizontal reinforcement in the abutment stem shall be designed by considering the stem to be continuous between piles. Vertical steel in the stem is usually controlled by shear considerations.

2. Wingwalls integral with the main abutment stem shall be designed as cantilevers over the outer piles or facia girder. Wingwalls separate from the main abutment stem shall be designed separately.

3. Abutments shall be designed for passive pressure developed against the back of the stem and wingwalls. Passive pressure values will be provided in the Foundation Design Report.

4. Piles shall be designed for normal vertical loads ignoring any fixity developed between the superstructure and the pile top.
17.5.2 Nongravity Cantilever and Anchored Walls

Nongravity and Anchored walls shall be designed using the latest edition of the AASHTO LRFD Bridge Design Specifications. Key geotechnical design requirements for these types of walls are found in Sections 3 and 11 of the AASHTO LRFD specifications. Instead of the resistance factor for passive resistance of the vertical wall elements provided in the AASHTO LRFD specifications, a resistance factor for passive resistance of 0.75 shall be used.

17.5.2.1 Sheeting Walls

Sheet piles are structural units which when connected one to another will form a continuous wall. The wall continuity is obtained by interlocking devices formed as part of the manufactured product. In New York State, the majority of sheeting used is made of steel, with timber used less often.

General design considerations include:
- Evaluation of the forces and lateral pressures that act on the wall:
  - Surcharge Loads: Design of sheet pile walls must take into consideration the loads other than soil or water pressure to which the wall might be subjected. Equipment operating adjacent to the railroad or highway loads, piles of ore or stone, buildings and crane rails, are examples of surcharge loads.
  - Unbalanced Hydrostatic Head: A sometimes forgotten load condition in design is the effect of waves, tidal action, storm surges and heavy rainfall on the total loading condition behind a wall. Since the locks of sheet piling are more or less tight when installed and become more watertight as soil is drawn in, water can be trapped behind the wall causing a head imbalance and greatly increasing the total load, even though temporary.
- Determination of the required depth of penetration of the piling for stability.
- Computations of the maximum bending moments in the piling.
- Computations of stresses in the wall and selection of the appropriate section and steel grade.
- Design of waling and anchorage systems (deadman, tiebacks) if required.
### Table 17-7 General Types of Steel Sheet Piling

- The Z-Type configuration is the strongest and most efficient. They are ideal for high cantilever walls and are used primarily for bulkheads, headwalls, deeper braced cofferdams or similar wall requiring high bending strength.
- Arch Web sections are used generally in the same type of application as Z-sections but where loading requirements are less, shallower excavations and walls for short length.
- Flat sections are used in applications (circular cofferdams) where they are subjected to hoop tension form internal pressure exerted by the retained soil, rather than bending. As a result, the ability to transfer this stress across the interlocks is most important.

### Figure 17-14 Methods for Obtaining Higher Section Modulus

- HZ WALL SYSTEM
- MASTER PILE SYSTEM
- REINFORCED Z-TYPE PILING
Details for driving shoes for sheet piles are provided on Bridge Detail sheet BD-MS-5E Miscellaneous Pile Details.

Details for sheeting utilized in support of staged construction are provided on the following Bridge Detail sheets:

- BD-EE-15E Excavation and Embankment Sample Drawing of Stage Construction (1 of 2)
- BD-EE-16E Excavation and Embankment Sample Drawing of Stage Construction (2 of 2)

17.5.2.2 Soldier Pile and Lagging Walls

A soldier pile and lagging wall is a temporary or permanent non-gravity cantilevered wall which derives lateral resistance and moment capacity through embedment of vertical wall elements (soldier piles). The soil behind the wall is retained by lagging. The vertical elements may be drilled or driven steel or concrete piles. These vertical elements are spanned by lagging which may be wood, reinforced concrete, precast or CIP concrete panels, or reinforced shotcrete.

Details for soldier pile and lagging walls are provided on Bridge Detail sheet BD-EE-11E Excavation and Embankment Soldier Pile and Lagging Wall Sample Details. Details for splicing and/or driving shoes for soldier piles are provided on Bridge Detail sheet BD-MS-5E Miscellaneous Pile Details.
Figure 17-15 Cantilever Soldier Pile and Lagging Wall
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<table>
<thead>
<tr>
<th>Soil Competence</th>
<th>Soil Description</th>
<th>Unified Classification</th>
<th>Depth (ft.)</th>
<th>Recommended Thickness (in.) of Lagging (roughcut) for Clear Spans of:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5 ft.</td>
</tr>
<tr>
<td>Competent Soils</td>
<td>Silts or fine sand and silt above the water table. Sands and gravels (medium dense to dense). Clays (stiff to very stiff); non-fissured. Clays, medium consistency and $\gamma H/S_u &lt; 5$</td>
<td>ML, SM-ML GW, GP, GM GC, SW, SP, SM CL, CH</td>
<td>0 – 25</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CL, CH</td>
<td>25 – 60</td>
<td>3</td>
</tr>
<tr>
<td>Difficult Soils</td>
<td>Sandy and silty sands, (loose) Clayey sands (medium dense to dense) below water table. Clays, heavily overconsolidated, fissured. Cohesionless silt or fine sand and silt below water table.</td>
<td>SW, SP, SM SC CL, CH</td>
<td>0 – 25</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ML, SM-ML</td>
<td>25 – 60</td>
<td>3</td>
</tr>
<tr>
<td>Potentially Dangerous Soils</td>
<td>Soft clays $\gamma H/S_u &gt; 5$. Slightly plastic silts below water table. Clayey sands (loose), below water table</td>
<td>CL, CH</td>
<td>0 – 15</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ML</td>
<td>15 – 25</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC</td>
<td>25 – 35</td>
<td>4</td>
</tr>
</tbody>
</table>

Note: In the category of “Potentially Dangerous Soils, use of lagging is questionable.

Table 17-8 Recommended Thickness of Wood Lagging (FHWA-RD-75-128)
17.5.2.3 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 ft 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.

Walls designed as cantilevers usually undergo large lateral deflections and are readily affected by scour and erosion in front of the wall. Since the lateral support for a cantilevered wall comes from passive pressure exerted on the embedded portion, penetration depths can be quite high, resulting in excessive stressed and severe yield. Earth pressures against a cantilevered wall is illustrated in Figure 17-16.

![Image of Earth Pressure on Cantilever Sheet Piling](after Teng) (USS Steel Sheet Piling Manual)

The distribution of earth pressure is different for sheet piling in granular soils and sheet piling in cohesive soils. Also, the pressure distribution in clays is likely to change with time.

In general, the drilled hole for the soldier piles for nongravity cantilever walls will be filled with concrete or grout, as described in the BD Sheets (BD-EE11E).

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
arching may not occur, and a smaller number of pile diameters (e.g., 1 to 2 diameters) should be
asumed for this passive resistance arching effect. For soldier piles placed in very dense soils,
such as glacially consolidated till, when grout is used, the strength of the grout may be similar
eough 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.

17.5.2.4 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.

Anchor walls derive their support by two means: passive pressure on the front of the embedded
portion of the wall and anchor tie rods near the top of the piling. This method is suitable for
heights up to about 40 ft., depending on the soil conditions. For higher walls the use of high-
strength steel piling, reinforced sheet piling, relieving platforms or additional tiers of tie rods may
be necessary. The overall stability of anchored sheet pile walls and the stresses in the members
depends on the interaction of a number of factors, such as the relative stiffness of the piling, the
depth of piling penetration, the relative compressibility and strength of the soil, the amount of
anchor yield, etc. In general, the greater the depth of penetration the lower the resultant flexural
stresses.

Figure 17-17 shows the general relationship between the depth of penetration, lateral pressure
distribution and elastic line or deflection shape.

![Figure 17-17 Affect of Depth of Penetration on Pressure Distribution and Deflected Shape (USS Steel Sheet Piling Manual)]
In general, the drilled hole for the soldier piles for anchored/braced walls will be filled with concrete or grout, as described in the BD Sheets (BD-EE11E). 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 grout 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 Departmental Geotechnical Engineer 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.

A braced excavation is a retaining structure, usually temporary in nature, which is used to support the sides of deep excavations. Such structures generally consist of vertical steel sheet piling braced by a system of wales and struts. They are used primarily for the excavation of trenches in construction situations where adjacent ground must be supported against settlement or slides. Usually in urban areas, the need to prevent settlement of the adjacent ground is a matter of prime importance, as such settlements can have disastrous effects on the structural integrity of adjacent buildings.

In general, the method of construction incorporates the following basic steps: (a) steel sheet piles are driven into the ground to a predetermined depth; (b) during excavation the sheeting is braced by horizontal wales supported by a system of struts or prestressed tiebacks; the support system for each wale system must be in place and tightened or prestressed against the sheeting before further excavation can proceed in order to prevent lateral deflection.

![Figure 17-18 Braced Cut](image-url)
Of significant importance are the benefits of driving the steel sheet piling to a greater depth than the design of excavation. In soft clays this usually results in resisting the heave of the bottom of the excavation. Greater wall depths may also be advantageous in excavations in granular soil below the water table thereby serving as a cutoff wall and reducing the danger of piping and the formation of boils. In addition, the continuity of sheet pile walls helps prevent excessive material loss from behind the wall.

![Pressure Diagrams for Braced Excavations](image)

**Figure 17-19 Pressure Diagrams for Braced Excavations**

In addition to the depiction identified in Figure 17-19,

- If groundwater is present, add its pressure to the trapezoidal soil pressure (see (b) of Figure 17-20):
- If surcharge load is present, add the lateral component of the surcharge load to the soil pressure distribution (see (c) of Figure 17-20):
Details for ground anchors are provided on Bridge Detail sheet BD-EE-12E *Excavation and Embankment Tieback Wall Details*.

Details for braced excavations are provided on Bridge Detail sheet BD-EE-10E *Excavation and Embankment Braced Excavation Details*.
CHAPTER 17
Abutments, Retaining Walls, and Reinforced Slopes

CUT IN COMHENSIONLESS SOIL

STABILITY IS INDEPENDENT OF H AND B, BUT VARIES WITH $\gamma^1$, $\phi^1$ AND SEEPAGE CONDITION.
SAFETY FACTOR, $F_s = 2N\gamma_2 (\frac{\gamma^1}{\gamma}) K_a \tan \phi$

$N_\gamma = BEARING\ CAPACITY\ FACTOR, FIGURE\ 1, CHAPTER\ 4$
IF GROUNDWATER IS AT A DEPTH OF (B) OR MORE BELOW BASE OF CUT:
$\gamma^1$ AND $\gamma_2$ ARE TAKEN AS MOIST UNIT WEIGHT
IF GROUNDWATER IS STATIC AT BASE OF CUT:
$\gamma^1 = MOIST\ WEIGHT, \gamma_2 = SUBMERGED\ WEIGHT.$
IF SEEPAGE IS MOVING UPWARD TO BASE OF CUT:
$\gamma_2 = \{SATURATED\ UNIT\ WEIGHT\} - \{UPLIFT\ PRESSURE\}$

CUT IN CLAY, DEPTH OF CLAY UNLIMITED ($T > 0.7B$)

L = LENGTH OF CUT

IF SHEETING TERMINATES AT BASE OF CUT:
SAFETY FACTOR, $F_s = \frac{N_c C}{\gamma T H + q}$

$N_c = BEARING\ CAPACITY\ FACTOR, FIGURE\ 2, CHAPTER\ 5$
WHICH DEPENDS ON DIMENSIONS OF THE EXCAVATION: $B, L$ AND $H$ (USE $H = z$).
C = UNDRAINED SHEAR STRENGTH OF CLAY IN FAILURE ZONE BENEATH AND SURROUNDING BASE OF CUT.
q = SURFACE SURCHARGE.
IFSAFETY FACTOR IS LESS THAN 1.5, SHEETING MUST BE CARRIED BELOW BASE OF CUT TO INSURE STABILITY.
FORCE ON BURIED LENGTH:
IF $H_1 > \frac{2}{3} \frac{B}{\sqrt{2}}$, $P_H = 0.7 (\gamma_T H - 1.4 C H - \pi C B)$
IF $H_1 < \frac{2}{3} \frac{B}{\sqrt{2}}$, $P_H = 1.5H_1 (\gamma_T H - 1.4 C H - \pi C)$

CUT IN CLAY, DEPTH OF CLAY LIMITED BY HARD STRATUM ($T=0.7B$)

FAILURE SURFACE

SHEETING TERMINATES AT BASE OF CUT. SAFETY FACTOR:
CONTINUOUS EXCAVATION: $F_s = N_{CD} \frac{C_1}{\gamma_T H + q}$
RECTANGULAR EXCAVATION: $F_s = N_{CR} \frac{C_1}{\gamma_T H + q}$

$N_{CD}$ AND $N_{CR} = BEARING\ CAPACITY\ FACTORS.$
FIGURE 5, CHAPTER 4, WHICH DEPEND ON DIMENSIONS OF THE EXCAVATION: $B, L$ AND $H, (USE H = z)$

NOTE: IN EACH CASE FRICTION AND ADHESION ON BACK OF SHEETING IS DISREGARDED.
CLAY IS ASSUMED TO HAVE A UNIFORM SHEAR STRENGTH = C THROUGHOUT FAILURE ZONE.

Figure 17-21 Stability of Base of Braced Cut (NAVFAC DM-7.2, 1982)
Figure 17-22 Chart for Obtaining the Depth of Sheet Piling to Prevent Piping in a Braced Cofferdam
(after NAVFAC DM-7.1, 1982)
Coarse sand underlying fine sand

Presence of coarse layer makes flow in fine material more nearly vertical and generally increases seepage gradients in the fine layer compared to the homogeneous cross-section of Fig. 62 (a).

If top of coarse layer is at a depth below sheeting tips greater than width of excavation, safety factors of Fig. 62 (a) for infinite depth apply.

If top of coarse layer is at a depth below sheeting tips less than width of excavation, the uplift pressures are greater than for the homogeneous cross-section. If permeability of coarse layer is more than ten times that of fine layer, failure head \(H_0\) = thickness of fine layer \(H_f\).

---

Fine sand underlying coarse sand

Presence of fine layer constricts flow beneath sheeting and generally decreases seepage gradients in the coarse layer.

If top of fine layer lies below sheeting tips, safety factors are intermediate between those for an impermeable boundary at top or bottom of the fine layer in Fig. 62 (a).

If top of the fine layer lies above sheeting tips the safety factors of Fig. 62 (a) are somewhat conservative for penetration required.

---

Fine layer in homogeneous sand stratum

If the top of fine layer is at a depth greater than width of excavation below sheeting tips, safety factors of Fig. 62 (a) apply, assuming impervious base at top of fine layer.

If top of fine layer is at a depth less than width of excavation below sheeting tips, pressure relief is required so that unbalanced head below fine layer does not exceed height of soil above base of layer.

If fine layer lies above subgrade of excavation, final condition is safer than homogeneous case, but dangerous condition may arise during excavation above the fine layer and pressure relief is required as in the preceding case.

Figure 17-23 Depth of Sheet Piling in Stratified Sand to Prevent Piping in a Braced Cofferdam (after NAVFAC DM-7.1, 1982)
17.5.2.5 Permanent Ground Anchors

A ground anchor consists of a prestressing steel element, called a tendon (bar or strand), which is inserted below ground into a preformed hole. The tendon is anchored to the ground by friction over the lower portion of the hole with cement grout. The remaining tendon length is typically enclosed in a sheath which permits free movement of the tendon.

In order to develop a tensile force in the tendon, a jack is attached to the free end of the tendon at the ground surface and the tendon is incrementally loaded until the desired load is achieved. As this is done, the ground surrounding the lower part of the grout encased tendon resists the tensile force applied by the jack. Before the jack load is released, the tendon is locked off against the structure thereby transmitting the tensile force developed in the tendon.

17.5.2.5.1 Applications

Retaining wall structures in cut situations are the most common ground anchor applications, particularly where high retaining walls are required or where limited right-of-way prevents the use of safe unsupported excavation slopes during construction.

Other common applications of ground anchors include:
- Repair or reconstruction of existing structures.
- Slope stabilization
- Bridge abutment construction in a cut situation.
- Tiedowns for structures concrete dams subjected to uplift or overturning forces.

17.5.2.5.2 Types of Anchors

Ground anchors are installed using a variety of drilling and grouting methods; the combination of which defines the specific anchor type.

Figure 17-24 Ground Anchors Used to Construct or as Part of Retaining Walls
The installation method selected by the Contractor will depend on soil type, specified capacity, structural requirements such as hole size, total anchor length, tendon diameter, groundwater conditions, site restrictions, and equipment availability.

Details for ground anchors are provided on Bridge Detail sheet BD-EE-12E Excavation and Embankment Tieback Wall Details.

17.5.2.5.3 Terminology

- **Tieback:** a system used to transfer tensile loads from a structure to soil or rock. A tieback includes all prestressing steel (tendon), the anchorage, grout, coatings, sheathing, couplers and encapsulation if used.
- **Tendon:** the steel used to transfer load from the anchorage to soil or rock.
  1. “Uncoated Seven-Wire Stress Relieved Strand for Prestressed Concrete” – ASTM A416, or “Uncoated Seven-Wire Compacted Stress Relieved Strand for Prestressed Concrete” – ASTM A779.
  2. Continuously threaded “Uncoated High-Strength Steel Bar for Prestressing Concrete” – ASTM A722.
- **Anchorage:** that portion of the tieback, including bearing plate nuts and wedges, that is used to transfer load from the structure to a tendon.
- **Bond Length:** that portion of the tieback which is bonded to the soil or rock and transfers the tensile force from the tendon to the soil or rock.
- **Tendon Bond Length:** the length of the tendon which is bonded to the group. This is usually, but not necessarily, the same as the Bond Length.
- **Stressing Length:** that portion of the tendon which is not bonded to grout.
- **Sheath:** that portion of the tieback which encases the tendon in the stressing length only.
- **Encapsulation:** that portion of the tieback which encases or encapsulates the entire length of the tieback, including the sheath, to provide an additional barrier to corrosion.
- **Total Movement:** the total elongation of the tieback under load measured at the anchor head.
- **Residual Movement:** that permanent set of the tieback resulting from stressing and releasing the tieback.
- **Trumpet:** A steel pipe or tube, integrally attached to the bearing plate that surrounds the tendon in the vicinity of the structure.
- **Creep Rate:** the magnitude of total movement measured during a load hold per log cycle of time.
- **Centralizer:** a device used to center the bond length of the tieback in the hole to assure minimum grout cover over the tieback.
- **Spacer:** a device used in strand tendons to separate each strand in the bond length to permit the grout to bond with each strand.
- **GUTS:** The guaranteed ultimate tensile strength of the tendon.
Figure 17-25 Strand Tendon and Bar Tendon Terminology
The Departmental Geotechnical Engineer 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 ft 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 ft minimum, and the grout in the unbonded zone is not placed by pressure grouting methods.

The Departmental Geotechnical Engineer 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 NYSDOT.

The Departmental Geotechnical Engineer shall estimate the nominal anchor bond stress ($\tau_n$) for the soil conditions and common anchor grouting methods. AASHTO LRFD Bridge Design Specifications and the FHWA publications listed at the beginning of this chapter provide guidance on acceptable values to use for various types of soil and rock. The Departmental Geotechnical Engineer 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 ft should be assumed when estimating the feasible anchor resistance. FHWA research has indicated that anchor bond lengths greater than 40 ft are not fully effective. Anchor bond lengths greater than 50 ft shall be approved by the Geotechnical Engineering Bureau.

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 will be tested in accordance with the NYSDOT Standard Specifications.

17.5.2.5.4 Installation

Geotechnical Engineering Manual GEM-17 Ground Anchor Inspector’s Manual provides a quick and easy-to-use set of inspection guidelines for the various aspects of tieback construction, including pre-installation inspection, installation, testing and trouble-shooting load cell problems. The manual provides checklists that are intended to serve as reminders for Inspectors already familiar with tieback installation.
17.5.2.6 Deadmen

The Departmental Geotechnical Engineer 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 17-26).
Figure 17-26 Deadman Anchor Design (after NAVFAC, 1982)
17.5.2.7 Soldier Pile & Lagging Walls on Slopes

- Design Considerations

1. Most important consideration – upper limit of water table. Water will influence the design and affect the design parameters more dramatically than any other consideration.
2. If the slope is steeper than $\phi$: Determine elevation where a safe slope will intercept the proposed wall. Use $\phi$ of soil and draw intercept from midpoint of unretained soil slope (See Figure 17-27).

![Figure 17-27 Safe Slope Intercept](image)

3. Only rely on passive resistance from soil located below the intercept elevation as determined in 2.
4. Minimum pile spacing is 3 times pile width for driven piles or 3 times hole diameter for augered piles. Augered hole diameter $\geq$ pile diagonal + 6 inches.
5. If an anchor is required:
   A. Determine if equipment access is sufficient for drilled and grouted anchors. A min. 8 ft. wide platform located 2 ft. below the anchor elevation is required. (See Figure 17-28). Equipment can be suspended over the side of the wall at a greater cost. Remember: The anchor cannot be located any lower than 2 ft. above the bottom of the maximum exposed face for a cantilever design (temp.).
B. Anchors relying on deadmen for resistance should be located near the top of the wall to minimize excavation requirements for rod placement and bracing (wales behind deadmen). Remember: Check the deadman location for resistance (see USS Steel Sheetpiling Design Manual – p. 45, Fig. 42).

6. Design Checks:
   A. Specify H-piles if soldier piles are to be driven. Tip treatment is required and the max. H-pile section modulus will govern design.
   B. Augering of holes for soldier piles is more costly than driving of piles.
   C. Soldier piles must be able to support the vertical force component of an inclined anchor load.

7. Design “Quickies”:
   A. As pile spacing increases, pile section modulus & embedment increase.
   B. As anchor location drops, pile section modulus & embedment decrease and the anchor load increases. However, the cantilevered section modulus and depth requirement for excavation prior to anchor installation may govern final design parameters.
   C. Augered passive resistance is larger than driven (pile width vs. augered hole diameter).

8. Other Considerations:
   A. Is number of borings sufficient for job?
   B. Pile alignment is critical if using concrete lagging.
   C. Vibrations if piles are to be driven.
   D. Is corrosion protection of piles necessary for a permanent wall installation?
   E. If anchors extend outside of right-of-way, easements will be required.
   F. H-Piles can be ordered in 50 ksi steel in lieu of standard 36 ksi steel.
   G. A minim distance (deflection) has to be provided between the guiderail and top of wall (on face of wall). (see Table 17-9).
<table>
<thead>
<tr>
<th>Barrier Type</th>
<th>Post Type (Deflection Category)</th>
<th>Post Spacing (ft.)</th>
<th>Standard Deflection (ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable</td>
<td>Weak Post (Flexible)</td>
<td>16</td>
<td>11</td>
</tr>
<tr>
<td>Cable &amp; Cable Median</td>
<td></td>
<td>12</td>
<td>9’-6”</td>
</tr>
<tr>
<td>Barrier</td>
<td></td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>7</td>
</tr>
<tr>
<td>Corrugated W-Beam Guiderail</td>
<td>Weak Post (Flexible)</td>
<td>12’-6”</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6’-3”</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4’-2”</td>
<td>5</td>
</tr>
<tr>
<td>Box Beam Guide Rail</td>
<td>Weak Post (Semi-Rigid)</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>

**Table 17-9 Guiderail Deflection Distance**  
(NYSDOT HDM Chapter 10)

**Figure 17-29 Example Soldier Pile & Lagging Wall on Slope**
17.5.3 Mechanically Stabilized Earth System (MSES) Walls

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

According to NYSDOT Standard Specifications, an internally stabilized earth system is a series of tensile reinforcing elements which, when placed in multiple layers within the backfill volume, improves the strength such that the vertical face of the stabilized earth volume is essentially self supporting. Therefore, a Mechanically Stabilized Earth System (MSES) is an internally stabilized fill structure faced with precast panel units.

A Mechanically Stabilized Earth System (MSES) is comprised of an unreinforced concrete leveling, footing, facing units, earth backfill, and a material used to stabilize the backfill. These proprietary systems (Figure 17-30) employ either strip, grid, or sheet type metallic inextensible, or grid type geosynthetic extensible, tensile reinforcements in the backfill to be stabilized. Inextensible reinforcements are defined as reinforcements where deformation at failure is much less than the deformability of the soil. Extensible reinforcements are defined as reinforcements where deformation at failure is comparable to, or even greater than, the deformability of the soil.

All MSES’s are designed by the Wall System Designer. Approved MSES’s appear on the NYSDOT Approved List.

Figure 17-30 Schematic Illustration of Mechanically Stabilized Earth Wall (Reinforced Earth Wall)
CHAPTER 17
Abutments, Retaining Walls, and Reinforced Slopes

Geotechnical Engineering Manual (GEM-16) Mechanically Stabilized Earth System Inspection Manual provides NYSDOT Construction Inspectors with a quick and easy-to-use set of inspection guidelines for the installation of MSES walls. The manual provides a series of checklists intended to serve as reminders to Inspectors of the important aspects for the successful construction of the wall.

• Review Guidelines

MSES walls must be designed for both external and internal stability. To be internally stable the MSES structure must be coherent and self-supporting under the action of its own weight and any externally applied forces. This is accomplished through stress transfer from the soil to the reinforcement. The reinforcements must be sized and spaced to preclude rupture under the stresses they are required to carry and to prevent pull-out from the soil mass.

MSES walls are subject to the same external stability design criteria as conventional retaining walls, independent of the type of reinforcing system utilized. The structure must be stable against sliding due to the lateral pressure of the soil retained by the wall (including landslide forces) resist overturning about its toe, and be safe against foundation failure and overall slope failure.

• Structure Dimensions

Dimension the structure to ensure that the following minimum Factors of Safety are satisfied:

<table>
<thead>
<tr>
<th>Factors of Safety</th>
<th>Failure Mode</th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding</td>
<td>≥ 1.5</td>
<td>≥ 1.1</td>
<td></td>
</tr>
<tr>
<td>Overturning</td>
<td>≥ 2.0</td>
<td>≥ 1.5</td>
<td></td>
</tr>
<tr>
<td>Bearing Capacity</td>
<td>≥ 2.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pullout Resistance</td>
<td>≥ 1.5</td>
<td>≥ 1.35</td>
<td></td>
</tr>
<tr>
<td>Overall Stability</td>
<td>≥ 1.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 17-10 Minimum Factors of Safety**

Minimum reinforcement length (L) will be approximately 70 percent of the mechanical wall height (Figure 17-31) H1, as measured from the top of the leveling footing. The reinforcement length should be uniform throughout the entire height of the wall and satisfy all external and internal stability considerations. External loads such as abutment footings or surcharges will increase the minimum reinforcement length.
Equation 17-1

\[ H_1 = H + \left( \frac{\tan \beta \times 0.3H}{1 - 0.3 \tan \beta} \right) \]

- Wall Embedment Depth

To prevent local bearing capacity failures and damage from frost heave, structures should be designed for the following minimum embedment unless constructed on rock foundations:

<table>
<thead>
<tr>
<th>Slope in Front of Structure</th>
<th>Minimum Embedment Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal – for Walls</td>
<td>H/20</td>
</tr>
<tr>
<td>Horizontal – for Abutments</td>
<td>H/10</td>
</tr>
<tr>
<td>1V on 3H</td>
<td>H/10</td>
</tr>
<tr>
<td>1V on 2H</td>
<td>H/7</td>
</tr>
<tr>
<td>2V on 3H</td>
<td>H/5</td>
</tr>
</tbody>
</table>

**Table 17-11 Minimum Embedment Depths**

The minimum embedment depth to the top of the leveling pad should be 2 ft. or below the prevailing depth of frost penetration, whichever is greater.
• Allowable Settlements

Allowable longitudinal differential settlements for mechanically stabilized earth structures are largely a function of panel size and joint design. It has been established that for discrete modular panels, less than 30 ft.² in area with a minimum design joint of ¾ in., differential settlements of 1% are allowable in the longitudinal direction. For panels with smaller design joints, or of larger size, this allowance should be proportionally reduced.

Allowable transverse differential settlements are a function of reinforcement length and size and ability of the panel connection to rotate. In general, differential vertical movements between the panel connection and the reinforcement must be controlled or limited to insure that reinforcement yield strains are not reached.

• Drainage

In cut areas and/or hillsides with established piezometric levels, construct the mechanically stabilized earth structures with drainage blankets in back of and possibly beneath the stabilized zone. Drainage measures should be considered for all structures to prevent saturation of the stabilized backfill or to intercept any surface flows containing aggressive elements such as de-icing chemicals.

• Special Loading Conditions

Incorporate concentrated line loads and strip loads into the internal design by using a simplified uniform vertical distribution of 2V on 1H to determine the vertical component of stress with depth within the stabilized earth mass.

Traffic surcharge loads in accordance with criteria outlined in Section 3.20.3 of AASHTO Standard Specifications for Highway Bridges.

• Pile Supported Abutments

The horizontal forces transmitted to the piles may be resisted by their own lateral capacity acquired in soils below the mechanically stabilized earth structure or by additional reinforcement in the upper portion of the mechanically stabilized earth structure. A minimum clear distance of 1.5 ft. shall be provided between the facing and the piles. Alternately, lateral capacity must still be obtained within MSES or below MSES, the piles can be sleeved through the fill, the annulus filled with compressible material to absorb the deflections.

17.5.3.1 Live Load Considerations for MSE Walls

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

Therefore, include live load in calculation of $T_{\text{max}}$, 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 ft shoulder and a minimum vehicle width of 8 ft, the active zone for steel reinforced walls would be wide enough for this to happen only if the wall is over 30 ft high, and for geosynthetic walls over 22 ft high. For walls of greater height, live load would need to be considered for pullout for the typical traffic loading situation.

17.5.3.2 Backfill Considerations for MSES Walls

One of the primary components of all fill walls is the backfill soil. Mechanically stabilized earth systems or mechanically stabilized wall systems are fill walls which incorporate a specialized backfill material to ensure long-term performance of the embedded reinforcing elements. Stability of these systems is achieved by the weight of the reinforced soil mass resisting the overturning and sliding forces generated by the lateral stresses from the retained soil behind the reinforced mass.

NYSDOT Standard Specifications provide requirements for Mechanically Stabilized Earth System Backfill Material. This granular material is used for backfill for both MSES and MSWS as these walls require a high quality backfill for durability, good drainage, constructability, and good soil reinforcement interaction which can be obtained from this well-graded, granular material. The high quality backfill requirements provided in the MSES specification has established minimum or maximum electrochemical index properties to coincide with design procedures for the maximum corrosion rates associated with those properties. Backfills incorporated with buried steel elements of these wall systems which fall outside these ranges provide the potential for a reduced lifespan of the structure along with increased maintenance costs.

The NYSDOT will test for the corrosion potential of any system with exposed metal in the backfill. Stockpiled materials will be tested for resistivity and pH, and may be tested for sulfides at the Department's discretion. Material failing to meet the resistivity criterion may be tested for sulfate and chlorides. Material meeting the criteria for both sulfates and chlorides and having a resistivity greater than 10 ohm-m will be acceptable.

Geotechnical Control Procedure (GCP-20) Procedure for Taking Random Samples of Backfill Material for Mechanically Stabilized Earth Systems provides a uniform and non-biased method by which the NYSDOT will select locations to take samples of backfill from behind a MSES or
MSWS structure during construction. This sampling is just one aspect of the overall QA process for MSES or MSWS walls.

### 17.5.3.3 Compound Stability Assessment for MSES Walls

If the MSES wall is located over a soft foundation soil or on a relatively steep slope, compound stability of the wall and slope combination should be evaluated as a service limit state in accordance with the AASHTO LRFD Specifications. It is recommended that this stability evaluation only be used to evaluate surfaces that intersect within the bottom 20 to 30 percent of the reinforcement layers. As discussed by Allen and Bathurst (2002) and Allen and Bathurst (2003), available limit equilibrium approaches such as the ones typically used to evaluate slope stability do not work well for internal stability of reinforced soil structures, resulting in excessively conservative designs, at least for geosynthetic or otherwise extensible reinforced systems.

The results of the compound stability analysis, if it controls the reinforcement needs near the base of the wall, should be expressed as minimum total reinforcement strength and total reinforcement pullout resistance for all layers within a “box” at the base of the wall to meet compound stability requirements. The location of the critical compound stability failure surface in the bottom portion of the wall should also be provided so that the resistant zone boundary location is identified.

Regarding pullout, the length of reinforcement needed behind the critical compound stability failure surface may vary significantly depending on the reinforcement coverage ratio anticipated and the frictional characteristics of the soil reinforcement. Therefore, several scenarios for these two 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 17-32. In this figure “H” is the total wall height, “T” is the load required in each reinforcement layer that must be resisted to achieve the desired level of safety in the wall for compound stability, and T_total is the total force increase needed in the compound stability analysis to achieve the desired level of safety with regard to compound stability. This total force should be less than or equal to the total long-term tensile strength, T_{al}, of the reinforcement layers within the defined “box” and the total pullout resistance available for the reinforcement contained within the box, considering factored loads and resistance values. The engineer needs to select the value of “B” that meets this pullout length requirement. However, the value of “B” selected should be minimized to keep the wall base width required to a minimum, to keep excavation needs as small as possible.

From the wall supplier’s view, the contract would specify a specific value of “B” that is long enough such that the desired minimum pullout resistance can be obtained but that provides a consistent basis for bidding purposes with regard to the amount of excavation and shoring needed to build the wall.
Note that for taller walls, it may be desirable to define more than one box at the wall base to improve the accuracy of the pullout length for the intersected reinforcement layers. If the wall is tiered, a box may need to be provided at the base of each tier, depending on the horizontal separation between tiers.

17.5.3.4 Design of MSE Walls Placed in Front of Existing Permanent Walls or Rock

Widening existing facilities sometimes requires MSE walls to be built in front of those existing facilities with inadequate room to obtain the minimum 0.7H wall base width. To reduce excavation costs and shoring costs in side hill situations, the “existing facility” could in fact be a shoring wall or even a near vertical rock slope face. See Figure 17-32 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 ft, whichever is greater, where H is the total height of the new wall. Note that for soil reinforcement lengths that are less than 8 ft, 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 ft 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 ft 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., $T_{\text{max}}$).

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

- For new walls with a height over 30 ft, a lateral deformation analysis should be conducted (e.g., using a properly calibrated numerical model). Approval from the Geotechnical Engineering Bureau and Office of Structures 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.

![Diagram of MSE wall](image)

**Figure 17-33 Example of Steep Shored MSEs Wall**

### 17.5.3.5 MSEs Wall Supported Abutments

MSEs walls directly supporting spread footing bridge abutments shall be 30 ft or less in total height (i.e., height of exposed wall plus embedment depth of wall). If for any specific wall system the height limit specified for that wall system is less than the 30 ft maximum limit, the height limit for the specific wall system shall not be exceeded. Abutment spread footings 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 footing bearing service loads greater than 2.0 tsf, a vertical settlement monitoring program with regard to footing 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, and settlement monitoring points directly below the footing 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 the footing supports, corrective action shall be taken.

The front edge of the abutment footing shall be 2 ft or more from the back of the MSEs facing units. There shall be at least 5 ft vertical clearance between the MSEs facing units and the

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**Note:** This text is a draft and may be subject to changes before final publication.
bottom of the superstructure, and 5 ft horizontal clearance between the back of the MSES facing units and face of the abutment wall to provide access for bridge inspection. Fall protection shall be installed as necessary.

For spans up to 30 ft, the front edge of the bearings, placed on top of a load distribution slab located at the wall top, need only be at least 1 ft behind the back of the MSES wall facing units, with at least 1 ft of vertical clearance between the MSES facing units and the bottom of the superstructure used to span between the two MSES walls.

The bearing resistance for the footing supported by the MSES 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 NYSDOT Bridge Manual for additional guidance on the application of load groups for design of MSES 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 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.

Details for MSES walls directly supporting spread footing bridge abutments are provided on Bridge Detail sheet BD-EE-13E Excavation and Embankment Sample M.S.E.S. Details.

17.5.3.6 Full Height Propped Precast Concrete Panel MSES 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.

NYSDOT has incurred some problems in construction of these types of systems including movement in tipping and minor rotational movement. For systems of great 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 Departmental Geotechnical Engineer.

17.5.3.7 Flexible Faced MSE Walls With Vegetation

If a vegetated face is to be used with an MSE wall, the exposed (i.e., above ground wall height shall be limited to 20 ft or less, and the wall face batter shall be no steeper than 6V on 1H, unless the facing is battered at 2V on 1H or flatter, in which case the maximum height could be extended to 30 ft). A flatter facing batter may be needed depending on the wall system. 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.
17.5.3.8 Mechanically Stabilized Wall System (MSWS) Walls

According to NYSDOT Standard Specifications, a prefabricated wall system is either
• a series of prefabricated wall system open top face units assembled to form bins which are connected in unbroken sequence, or
• a combination of specific prefabricated wall system solid face units with a characteristic alignment and connection method, which utilize the weight of the wall system elements and the weight of the infill to resist lateral soil pressure.

An internally stabilized wall system is a wall system which, when constructed beyond wall heights exceeding the maximum allowable unreinforced height per the Approved List, relies on reinforcing elements within the backfill to provide stability. Therefore, a Mechanically Stabilized Wall System (MSWS) is an internally stabilized fill structure consisting of prefabricated wall system units which has introduced reinforcements within the backfill.

All MSWS's are designed by the Wall System Designer. Approved MSWS’s appear on the NYSDOT Approved List.

For extremely high dry cast block faced walls, block cracking in near vertical walls below a depth of 25 ft from the wall top in some block faced walls is possible. Key contributing factors include tolerances in the vertical dimension of the blocks that are too great (maximum vertical dimension tolerance should be maintained at +1/16 in or less, even though the current ASTM requirements for these types of blocks have been relaxed to +1/8 in), poor block placement technique, soil reinforcement placed between the blocks that creates too much unevenness between the block surfaces, some forms of shimming to make facing batter adjustments, and inconsistencies in the block concrete properties. See Figure 17-34 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 face possibly 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 ft of walls that have a hinge height of approximately 25 to 30 ft (total height of 35 ft or more) and may have been caused by excessive downdrag forces due to backfill soil compressibility immediately behind the facing.
Considering these potential problems, for dry cast concrete block faced walls, the wall height should be limited to 30 ft if near vertical, or to a hinge height of 30 ft if battered. Block wall heights greater than this may be considered on a project specific basis, subject to the approval of the Geotechnical Engineering Bureau and Office of Structures, if the requirements identified below are met:

- Total settlement is limited to 2 in and differential settlement is limited to 1.5 in as identified in Table 17-5. Since this is specified in Table 17-5, 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 multifilament 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 ft or more below the wall top, but doing this for block lifts at depths of less than 25 ft 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 ft 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.

**17.5.3.9 Geosynthetically Reinforced Soil System (GRSS) Walls**

According to NYSDOT Standard Specifications, an internally stabilized earth system is a series of tensile reinforcing elements which, when placed in multiple layers within the backfill volume, improves the strength such that the vertical face of the stabilized earth volume is essentially self supporting. Therefore, a Geosynthetically Reinforced Soil System (GRSS) is an internally stabilized fill structure faced with either welded wire forms (typically used for temporary walls), geocells, or timbers.

A GRSS is comprised of earth backfill, extensible (geosynthetic) reinforcing elements used for internal stabilization and surface protection to resist erosion. For wall applications, the surface protection is the permanent facing elements (excluding precast units) or a geotextile face wrap which typically includes welded wire forms remaining from the installation operation. Extensible reinforcements are defined as reinforcements where deformation at failure is comparable to, or even greater than, the deformability of the soil. Standard Sheet 554-02 *Geosynthetically Reinforced Soil Systems* provides installation requirements.

![Figure 17-35 Geosynthetically Reinforced Soil System Wall Typical Section](image-url)
Selection of the allowable tensile force per unit width of reinforcement, $T_a$, is more complex for geosynthetics than for steel. The tensile properties of geosynthetics are affected by creep, construction damage, aging, temperature, and confining stress. Furthermore, characteristics of geosynthetic products manufactured with the same base polymer vary widely, and the details of polymer behavior are generally unfamiliar to Civil Engineers. See Section 17.5.3.10.5 for design.

17.5.3.10 Internal Stability Using K-Stiffness Method

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

17.5.3.10.1 K-Stiffness Method Loads and Load Factors

The methods used in historical design practice for calculating the load in the reinforcement to accomplish internal stability design include the Simplified Method, the Coherent Gravity Method, and the FHWA Structure Stiffness Method. All of these methods are empirically derived, relying on limit equilibrium concepts for their formulation, whereas, the K-Stiffness Method, also empirically derived, relies on the difference in stiffness of the various wall components to distribute a total lateral earth pressure derived from limit equilibrium concepts to
the wall reinforcement layers and the facing. Though all of these methods can be used to evaluate the potential for reinforcement rupture and pullout for the Strength and Extreme Event limit states, only the K-Stiffness Method can be used to directly evaluate the potential for soil backfill failure and to design the wall internally for the service limit state. These other methods used in historical practice indirectly account for soil failure and service limit state conditions based on the successful construction of thousands of structures (i.e., if the other limit states are met, soil failure will be prevented, and the wall will meet serviceability requirements for internal stability).

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

The design procedures provided herein assume that the wall facing combined with the reinforced backfill acts as a coherent unit to form a gravity retaining structure. The effect of relatively large vertical spacing of reinforcement on this assumption is not well known and a vertical spacing greater than 2.7 ft should not be used without full scale wall data (e.g., reinforcement loads and strains, and overall deflections) which supports the acceptability of larger vertical spacings. Allen and Bathurst (2003) do report that based on data from a number of wall case histories, the correlation between vertical spacing and reinforcement load appears to remain linear for vertical spacings ranging from 1 to 5 ft, though the data at vertical spacings greater than 2.7 ft are very limited. However, larger vertical spacings can result in excessive facing deflection, both localized and global, which could in turn cause localized elevated stresses in the facing and its connection to the soil reinforcement.

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

**Equation 17-2**

$$\sigma_v = \gamma_p \gamma_r H + \gamma_p \gamma_f S + \gamma_{LL} q$$

where:

$\sigma_v$ = the factored pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present (KSF)

$\gamma_p$ = the load factor for vertical earth pressure EV in Table 17-12

$\gamma_{LL}$ = the load factor for live load surcharge per the AASHTO LRFD Specifications

$q$ = live load surcharge (KSF)

$H$ = the total vertical wall height at the wall face (FT)

$S$ = average soil surcharge depth above wall top (FT)

$\gamma_r$ = the unit weight of the reinforced soil backfill (KCF)

$\gamma_f$ = the unit weight of the soil backfill behind and above the reinforced soil zone (KCF)

Note that sloping soil surcharges are taken into account through an equivalent uniform surcharge and assuming a level backslope condition. For these calculations, the wall height “$H$” is referenced from the top of the wall at the wall face to the top of the bearing pad, excluding any copings and appurtenances.
Methods used in historical practice (e.g., the Simplified Method) calculate the vertical stress resulting from gravity forces within the reinforced backfill at each level, resulting in a linearly increasing gravity force with depth and a triangular lateral stress distribution. The K-Stiffness Method instead calculates the maximum gravity force resulting from the gravity forces within the reinforced soil backfill to determine the maximum reinforcement load within the entire wall reinforced backfill, $T_{\text{max}}$, and then adjusts that maximum reinforcement load with depth for each of the layers using a load distribution factor, $D_{\text{max}}$, to determine $T_{\text{max}}$. This load distribution factor was derived empirically based on a number of full scale wall cases and verified through many numerical analyses (see Allen and Bathurst, 2003).

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

\[
T_{\text{max}} = 0.5S_v K \sigma_v D_{\text{max}} \Phi_g \Phi_{\text{local}} \Phi_{fb} \Phi_{fs} \Phi_c
\]

where:
- $S_v$ = tributary area (assumed equivalent to the average vertical spacing of the reinforcement at each layer location when analyses are carried out per unit length of wall), in ft.
- $K$ = is an index lateral earth pressure coefficient for the reinforced backfill, and shall be set equal to $K_o$ as calculated per Article 3.11.5.2 of the AASHTO LRFD Specifications. $K$ shall be no less than 0.3 for steel reinforced systems.
- $\sigma_v$ = the factored pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present, as calculated in Equation 17-2 (ksf)
- $D_{\text{max}}$ = distribution factor to estimate $T_{\text{max}}$ for each layer as a function of its depth below the wall top relative to $T_{\text{max}}$ (the maximum value of $T_{\text{max}}$ within the wall)
- $S_{\text{global}}$ = global reinforcement stiffness (ksf)
- $\Phi_g$ = global stiffness factor
- $\Phi_{\text{local}}$ = local stiffness factor
- $\Phi_{fb}$ = facing batter factor
- $\Phi_{fs}$ = facing stiffness factor
- $\Phi_c$ = soil backfill cohesion factor
- $D_{\text{max}}$ shall be determined from Figure 17-37.

The global stiffness, $S_{\text{global}}$, considers the stiffness of the entire wall section, and it shall be calculated as follows:
Equation 17-4

\[ S_{global} = \frac{J_{ave}}{H} = \frac{\sum_{i=1}^{n} J_i}{n} \]

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

Equation 17-5

\[ \Phi_s = 0.25 \left( \frac{S_{global}}{P_a} \right)^{0.25} \]

where:
\( p_a \) = atmospheric pressure (a constant equal to 2.11 ksf), and the other variables are as defined previously.

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

Equation 17-6

\[ S_{local} = \frac{J}{S_v} \]

where:
\( J \) is the stiffness of an individual reinforcement layer (kips/ft.), and
\( S_v \) is the vertical spacing of the reinforcement layers near a specific layer (ft.).

The local stiffness factor, \( \Phi_{local} \), is then defined as follows:

Equation 17-7

\[ \Phi_{local} = \left( \frac{S_{local}}{S_{global}} \right)^a \]

where
\( a = \) a coefficient which is also a function of stiffness. Based on observations from the available data, set \( a = 1.0 \) for geosynthetic walls and \( = 0.0 \) for steel reinforced soil walls.
The wall face batter factor, \( \Phi_{fb} \), which accounts for the influence of the reduced soil weight on reinforcement loads, is determined as follows:

**Equation 17-8**

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

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

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

**Equation 17-9**

\[
K_{ab} = \frac{\cos^2 (\phi + \omega)}{\cos^3 \omega [1 + \frac{\sin \phi}{\cos \omega}]} \cos (\omega)
\]

where:
- \( \phi = \) peak soil friction angle (\( \phi_{peak} \)), and
- \( \omega = \) wall/slope face inclination (positive in a clockwise direction from the vertical). The wall face batter \( \omega \) is set equal to 0 to determine \( K_{av} \) using Equation 17-9. The horizontal component of the active earth pressure coefficient, assuming no wall/soil interface friction, is determined as follows:

**Equation 17-10**

\[
K_{abh} = K_{ab} \cos (\omega)
\]

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

The facing stiffness factor, \( \Phi_{fs} \), was empirically derived to account for the significantly reduced reinforcement stresses observed for geosynthetic walls with segmental concrete block and propped panel wall facings. It is not yet known whether this facing stiffness correction is fully applicable to steel reinforced wall systems. On the basis of data available at the time of this report, Allen and Bathurst (2003) recommend that this facing stiffness factor be determined as a function of a non-dimensional facing column stiffness parameter \( F_f \):
Equation 17-11

\[
F_f = \frac{1.5H^3p_a}{EB_w^3(\frac{h_{eff}}{H})}
\]

and

Equation 17-12

\[
\Phi_{fs} = \eta(F_f)^\kappa
\]

where:

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

Equation 17-11 was developed by treating the facing column as an equivalent uniformly loaded cantilever beam. It is recognized that Equation 17-11 represents a rather crude model of the stiffness of a retaining wall facing column, considering that the wall toe may not be completely fixed, the facing column often contains joints (i.e., the beam is not continuous), and the beam is attached to the reinforcement at various points. Since this analysis is being used to isolate the contribution of the facing to the load carrying capacity of the wall system, a simplified model that treats the facing as an isolated beam can be used. Once significant deflection occurs in the facing column, the reinforcement is then forced to carry a greater percentage of the load in the wall system. The full-scale wall data was used by Allen and Bathurst (2003) to empirically determine the percentage of load carried by these two wall components. Due to these complexities, these equations have been used in this analysis only to set up the form of a parameter that can be used to represent the approximate stiffness of the facing column.

For modular block faced wall systems, due to their great width, \(h_{eff}\) can be considered approximately equal to the average height of the facing column between reinforcement layers, and that the blocks between the reinforcement layers behave as if continuous. The blocks are in compression, partially due to self weight and partially due to downdrag forces on the back of the facing (Bathurst, et al. 2000), and can effectively transmit moment throughout the height of the column between the reinforcement layers that are placed between the blocks where the reinforcement is connected to the facing. The compressibility of the reinforcement layer placed between the blocks, however, can interfere with the moment transmission between the blocks above and below the reinforcement layer, effectively reducing the stiffness of the facing column.
Therefore, $h_{eff}$ should be set equal to the average vertical reinforcement spacing for this type of facing. Incremental panel faced systems are generally thinner (a thickness of approximately 4 to 5.5 in) and the panel joints tend to behave as a pinned connection. Therefore, $h_{eff}$ should be set equal to the panel height for this type of facing. The stiffness of flexible wall facings is not as straight-forward to estimate. Until more is known, a facing stiffness factor $\Phi_{fs}$ of 1.0 should be used for all flexible faced walls (e.g., welded wire facing, geosynthetic wrapped facings, including such walls where a precast or cast-in-place concrete facing is placed on the wall after the wall is built).

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

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

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

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

**Equation 17-13**

$$ \Phi_c = 1 - \frac{\lambda}{\gamma H} $$

where:

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

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

$D_{\text{max}}$ shall be determined as shown in Figure 17-37. Allen and Bathurst (2003) found that as the reinforcement stiffness increases, the load distribution as a function of depth below the wall top becomes more triangular in shape. $D_{\text{max}}$ is the ratio of $T_{\text{max}}$ in a reinforcement layer to the maximum reinforcement load in the wall, $T_{\text{max}}$. Note that the empirical distributions provided in Figure 17-37 apply to walls constructed on a firm soil foundation. The distributions that would result for a rock or soft soil foundation may be different from those shown in this figure, and in general will tend to be more triangular in shape as the foundation soils become more compressible.

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

![Figure 17-37 $D_{\text{max}}$ as a Function of Normalized Depth Below Wall Top Plus Average Surcharge Depth: (A) Generally Applies to Geosynthetic Walls, (B) Generally Applies to Polymer Strap Walls and Extensible or Very Lightly Reinforced Steel Reinforced Systems, and (C) Generally Applies to Steel Reinforced Systems](image)

Triaxial or direct shear soil friction angles should be used with the Simplified Method provided in the AASHTO LRFD Specifications, to be consistent with the current specifications and empirical derivation for the Simplified Method, whereas plane strain soil friction angles should be used with the K-Stiffness Method, to be consistent with the empirical derivation and

calibration for that method. The following equations maybe used to make an approximate estimate of the plane strain soil friction angle based on triaxial or direct shear test results.

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

**Equation 17-14**

\[ \phi_{ps} = 1.5\phi_{tx} - 17 \]

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

**Equation 17-15**

\[ \phi_{ps} = \tan^{-1}(1.2\tan\phi_{ds}) \]

All soil friction angles are in degrees for both equations. Direct shear or triaxial soil friction angles may be used for design using the K-Stiffness Method, if desired, but it should be recognized that doing so could add some conservatism to the resulting load prediction. Note that if presumptive design parameters are based on experience from triaxial or direct shear testing of the backfill, a slight increase in the presumptive soil friction angle based on Equations 17-14 or 15-15 is appropriate to apply.

### 17.5.3.10.2 K-Stiffness Method Load Factors

In addition to the load factors provided in Section 3.4.1 of the AASHTO LRFD specifications, the load factors provided in Table 17-12 shall be used as minimum values for the K-Stiffness Method. The load factor \( \gamma_p \) to be applied to maximum load carried by the reinforcement \( T_{max} \) due to the weight of the backfill for reinforcement strength, connection strength, and pullout calculations shall be EV, for vertical earth pressure. The load factors presented in Table 17-12 were developed using the soil reinforcement load data presented by Allen and Bathurst (2003), Allen at al. (2003, 2004), and Bathurst et al. (2008b), and the load factor calibration methodology as described in Allen, et al. (2005) and Bathurst, et al. (2008a).

Loads carried by the soil reinforcement in mechanically stabilized earth walls are the result of vertical and lateral earth pressures which exist within the reinforced soil mass, reinforcement extensibility, facing stiffness, wall toe restraint, and the stiffness and strength of the soil backfill within the reinforced soil mass. The calculation method for \( T_{max} \) is empirically derived, based on reinforcement strain measurements, converted to load based on the reinforcement stiffness, from full scale walls at working stress conditions (see Allen and Bathurst, 2003; and Bathurst, et al., 2008). Research by Allen and Bathurst (2003) indicates that the working loads measured in MSES wall reinforcement remain relatively constant throughout the wall life, provided the wall is designed for a stable condition, and that the load statistics remain constant up to the point that the wall begins to fail. Therefore, the load factors for MSES wall reinforcement loads provided in Table 17-12 can be considered valid for strength limit states.
Another strength limit state that needs to be considered for these walls is the prevention of soil failure. Soil failure is defined as contiguous or near contiguous zones of soil with shear strains in excess of the strain at peak strength. Contiguous shear zones have been observed in test walls taken to collapse under uniform surcharge loading (Bathurst 1990, Bathurst et al. 1993b, Allen and Bathurst 2002b). Allen and Bathurst (2002b) found that once a wall goes beyond working stress conditions, the load levels in the reinforcement begin to increase as internal soil shear surfaces continue to develop and the soil approaches a residual strength. Once the soil has exceeded its peak shear strain and begins to approach its residual shear strength, for all practical purposes the wall has failed and an internal strength limit state for the soil achieved.

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

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

<table>
<thead>
<tr>
<th>Type of Load</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>EV: Vertical Earth Pressure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MSE S Wall soil reinforcement loads (K-stiffness Method, steel strips and grids)</td>
<td>1.55</td>
<td>NA</td>
</tr>
<tr>
<td>MSE S Wall soil reinforcement/facing connection loads (K-stiffness Method, steel grids attached to rigid facings)</td>
<td>1.80</td>
<td>NA</td>
</tr>
<tr>
<td>MSE S Wall soil reinforcement loads (K-stiffness Method, geosynthetics, reinforcement rupture)</td>
<td>1.55</td>
<td>NA</td>
</tr>
<tr>
<td>MSE S Wall soil reinforcement loads (K-stiffness Method, geosynthetics, soil failure)</td>
<td>1.40</td>
<td>NA</td>
</tr>
<tr>
<td>MSE S Wall soil reinforcement/facing connection loads (K-stiffness Method, geosynthetics)</td>
<td>1.80</td>
<td>NA</td>
</tr>
</tbody>
</table>

#### Table 17-12 Load Factors for Permanent Loads for Internal Stability of MSE S Walls Designed Using the K-Stiffness Method, γp, for the Strength Limit State

The load factors provided in Table 17-12 were determined assuming that the appropriate mean soil friction angle is used for design. In practice, since the specific source of material for wall backfill is typically not available at the time of design, presumptive design parameters based on previous experience with the material that is typically supplied to meet the backfill material specification. It is likely that these presumptive design parameters are lower bound conservative values for the backfill material specification selected.

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

### 17.5.3.10.3 K-Stiffness Method Resistance Factors

For the service limit state, a resistance factor of 1.0 should be used, except for the evaluation of overall slope stability as prescribed by the AASHTO LRFD specifications (see also Section 17.4.12). For the strength and extreme event limit states for internal stability using the K-Stiffness Method, the resistance factors provided in Table 17-13 shall be used as maximum values. These resistance factors were derived using the data provided in Allen and Bathurst (2003). Reliability theory, using the Monte Carlo Method as described in Allen, et al. (2005) was applied to statistically characterize the data and to estimate resistance factors. The load factors provided in Table 17-12 were used for this analysis.

The resistance factors, specified in Table 17-13 are consistent with the use of select granular backfill in the reinforced zone, homogeneously placed and carefully controlled in the field for conformance with the NYSDOT Standard Specifications. The resistance factors provided in Table 17-13 have been developed with consideration to the redundancy inherent in MSE walls.
due to the multiple reinforcement layers and the ability of those layers to share load one with another. This is accomplished by using a target reliability index, $\beta$, of 2.3 (approximate probability of failure, $P_f$, of 1 in 100 for static conditions) and a $\beta$ of 1.65 (Approximate $P_f$ of 1 in 20) for seismic conditions. A $\beta$ of 3.5 (approximate $P_f$ of 1 in 5,000) is typically used for structural design when redundancy is not considered or not present; see Allen et al. (2005) for additional discussion on this issue. Because redundancy is already taken into account through the target value of $\beta$ selected, the factor $\eta$ for redundancy prescribed in the AASHTO LRFD specifications should be set equal to 1.0. The target value of $\beta$ used herein for seismic loading is consistent with the overstress allowed in previous practice as described in the AASHTO Standard Specifications for Highway Bridges (AASHTO 2002).

17.5.3.10.4 Safety Against Structural Failure (Internal Stability)

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

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

The load in the reinforcement shall be determined at two critical locations: the zone of maximum stress and the connection with the wall face. Potential for reinforcement rupture and pullout are evaluated at the zone of maximum stress, which is assumed to be located at the boundary between the active zone and the resistant zone in Figure 11.10.2-1 of the AASHTO LRFD Bridge Design Specifications. Potential for reinforcement rupture and pullout are also evaluated at the connection of the reinforcement to the wall facing. The reinforcement shall also be designed to prevent the backfill soil from reaching a failure condition.
<table>
<thead>
<tr>
<th>Limit State and Reinforcement Type</th>
<th>Internal Stability of MSE Walls, K-Stiffness Method</th>
<th>Resistance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi_{rr}$ Reinforcement Rupture</td>
<td>Metallic</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>Geosynthetic</td>
<td>0.85 (3)</td>
</tr>
<tr>
<td>$\varphi_{sf}$ Soil Failure</td>
<td>Metallic</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>Geosynthetic</td>
<td>1.00 (1)</td>
</tr>
<tr>
<td>$\varphi_{cr}$ Connection Rupture</td>
<td>Metallic</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>Geosynthetic</td>
<td>0.80 (3)</td>
</tr>
<tr>
<td>$\varphi_{po}$ Pullout (2)</td>
<td>Steel ribbed strips (at z&lt;6.5 ft.)</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>Steel ribbed strips (at z&gt;6.5 ft.)</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Steel smooth strips</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Steel grids</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>Geosynthetic</td>
<td>0.80</td>
</tr>
<tr>
<td>$\varphi_{EQr}$ Combined static/earthquake loading (reinforcement and connector rupture)</td>
<td>Metallic</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Geosynthetic</td>
<td>0.85 (3)</td>
</tr>
<tr>
<td>$\varphi_{EQp}$ Combined static/earthquake loading (pullout) (2)</td>
<td>Steel ribbed strips (at z&lt;6.5 ft.)</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>Steel ribbed strips (at z&gt;6.5 ft.)</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>Steel smooth strips</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>Steel grids</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>Geosynthetic</td>
<td>0.80</td>
</tr>
</tbody>
</table>

(1) If default value for the critical reinforcement strain of 3.0% or less is used for flexible wall facings, and 2.0% or less for stiff wall facings (for a facing stiffness factor of less than 0.9).
(2) Resistance factor values in table for pullout assume that the default values for $F^*$ and $\alpha$ provided in Article 11.10.6.3.2 of the AASHTO LRFD Specifications are used and are applicable.
(3) This resistance factor applies if installation damage is not severe (i.e. $RF_{ID}<1.7$). Severe installation damage is likely if very light weight reinforcement is used. Note that when installation damage is severe, the resistance factor needed for this limit state can drop to approximately 0.15 or less due to greatly increased variability in the reinforcement strength, which is not practical for design.

Table 17-13 Resistance Factors for the Strength and Extreme Event Limit States for MSES Walls Designed Using the K-Stiffness Method

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

17.5.3.10.5 Strength Limit State Design for Internal Stability Using the K-Stiffness Method – Geosynthetically Reinforced Walls (MSES, MSWS, or GRSS)

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

1. Select a trial reinforcement spacing, \( S_v \), and stiffness, \( J_{EOC} \), based on the time required to reach the end of construction (EOC). If the estimated time required to construct the wall is unknown, an assumed construction time of 1,000 hours should be adequate. Note that at this point in the design, it does not matter how one obtains the stiffness. It is simply a value that one must recognize is an EOC stiffness determined through isochronous stiffness curves at a given strain and temperature, and that it represents the stiffness of a continuous reinforcement layer on a per ft of wall width basis. Use the selected stiffness to calculate the trial global stiffness of the wall, \( S_{global} \), using Equation 17-4, with \( J_{EOC} \) equal to \( J_i \) for each layer. Also select a soil friction angle for design (see NYSDOT GDM 15.5.3.9.1). Once the design soil friction angle has been obtained, the lateral earth pressure coefficients needed for determination of \( T_{max} \) (Step 4) can be determined (see NYSDOT GDM Section 17.5.3.9.1). Note that if the reinforcement layer is intended to have a coverage ratio, \( R_c \), of less than 1.0 (i.e., the reinforcement it to be discontinuous), the actual product selected based on the K-Stiffness design must have a stiffness of \( J_{EOC}(1/R_c) \).

2. Begin by checking the strength limit state for the backfill soil. The goal is to select a stiffness that is large enough to prevent the soil from reaching a failure condition.

3. Select a target reinforcement strain, \( \varepsilon_{targ} \), to prevent the soil from reaching its peak shear strain. The worst condition in this regard is a very strong, high peak friction angle soil, as the peak shear strain for this type of soil will be lower than the peak shear strain obtained from most backfill soils. The results of full-scale wall laboratory testing showed that the reinforcement strain at which the soil begins to exhibit signs of failure is on the order of 3 to 4 percent for high shear strength sands (Allen and Bathurst, 2003). This empirical evidence reflects very high shear strength soils and is probably a worst case for design purposes, in that most soils will have larger peak shear strain values than the soils tested in the fullscale walls. A default value for \( \varepsilon_{targ} \) adequate for granular soils is 3 percent for flexible faced walls, and 2 percent for stiff faced walls if a \( \Phi_{fs} \) of less than 0.9 is used for design. Lower target strains could also be used, if desired.

4. Calculate the factored load \( T_{max} \) for each reinforcement layer (Equation 17-3). To determine \( T_{max} \), the facing type, dimensions, and properties must be selected to determine \( \Phi_{fs} \). The local stiffness factor \( \Phi_{local} \) for each layer can be set to 1.0, unless the
reinforcement spacing or stiffness within the design wall section is specifically planned to be varied. The global wall stiffness, \( S_{\text{global}} \), and global stiffness factor, \( \Phi_g \), must be estimated from \( J_{\text{EOC}} \) determined in Step 1.

5. Estimate the factored strain in the reinforcement at the end of the wall design life, \( \varepsilon_{\text{rein}} \), using the K-Stiffness Method as follows:

\[
\varepsilon_{\text{rein}} = \left( \frac{T_{\text{max}}}{J_{\text{DL}} \phi_{sf}} \right)
\]

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

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

7. Next, check the strength limit state for reinforcement rupture in the backfill. The focus of this limit state is to ensure that the long-term factored rupture strength of the
reinforcement is greater than the factored load calculated from the K-Stiffness Method. $T_{\text{max}}$ calculated from Step 4 is a good starting point for evaluating this limit state. Note that the global wall stiffness for this calculation is based on the EOC stiffness of the reinforcement, as the reinforcement loads should still be based on EOC conditions, even though the focus of this calculation is at the end of the service life for the wall.

8. Calculate the strength reduction factors $RF_{\text{ID}}$, $RF_{\text{CR}}$, and $RF_{\text{D}}$ for the reinforcement type selected using the approach prescribed in AASHTO PP66-10. Because the focus of this calculation is to prevent rupture, these factors must be based on reinforcement rupture. Applying a resistance factor to address uncertainty in the reinforcement strength, determine $T_{\text{ult}}$, the ultimate tensile strength of the reinforcement as follows:

**Equation 17-17**

$$T_{\text{max}} \leq \frac{T_{\text{ult}} \phi_{rr} R_c}{RF_{\text{ID}} RF_{\text{CR}} RF_{\text{D}}}$$

Where,

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

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

**Equation 17-18**

$$T_{\text{aldesign}} = \frac{T_{\text{max}}}{\phi_{rr} R_c}$$

where:

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

10. If the geosynthetic reinforcement is connected directly to the wall facing (this does not include facings that are formed by simply extending the reinforcement mat), the reinforcement strength needed to provide the required long-term connection strength must be determined. Determine the long-term connection strength ratio $CR_{cr}$ at each reinforcement level, taking into account the available normal force between the facing blocks, if the connection strength is a function of normal force. $CR_{cr}$ is calculated or measured directly per the AASHTO LRFD Specifications.
11. Using the unfactored reinforcement load from Step 6 and an appropriate load factor for the connection load to determine $T_{\text{max}}$ (factored) at the connection, determine the adequacy of the long-term reinforcement strength at the connection. Compare the factored connection load at each reinforcement level to the available factored long-term connection strength as follows:

Equation 17-19

$$T_{\text{max}} \leq \phi_{cr} T_{ac} R_c = \frac{\phi_{cr} T_{\text{ult}} CR_{cr} R_c}{RF_D}$$

where:

$T_{\text{max}}$ is the factored reinforcement load. Note that for modular block faced walls, the connection test data produced and used for design typically already has been converted to a load per unit width of wall facing – hence, $R_c = 1$. For other types of facing (e.g., precast concrete panels, if discontinuous reinforcement is used (e.g., polymer straps), it is likely that $R_c < 1$ will need to be used in Equation 17-19. If the reinforcement strength available is inadequate to provide the needed connection strength as calculated from Equation 17-19, decrease the spacing of the reinforcement or increase the reinforcement strength. Then recalculate the global wall stiffness and re-evaluate all previous steps to ensure that the other strength limit states are met. If the strength limit state for reinforcement or connection rupture is controlling the design, increase the reinforcement stiffness and check the adequacy of the design, increasing $T_{\text{al}}$ or $T_{\text{ult}}$ if necessary.

12. It must be recognized that the strength ($T_{\text{ult}}$ and $T_{\text{al}}$) and stiffness ($J_{EOC}$) determined from the K-Stiffness Method could result in the use of very light weight geosynthetics. In no case shall geosynthetic reinforcement be used that has an RFID applicable to the anticipated soil backfill gradation and installation conditions anticipated of greater than 1.7, as determined per AASHTO PP66-10. Furthermore, reinforcement coverage ratios, $R_c$, of less than 1.0 may be used provided that it can be demonstrated the facing system is fully capable of transmitting forces from un-reinforced segments laterally to adjacent reinforced sections through the moment capacity of the facing elements. For walls with modular concrete block facings, the gap between soil reinforcement sections or strips at a horizontal level shall be limited to a maximum of one block width in accordance with the AASHTO LRFD Specifications, to limit bulging of the facing between reinforcement levels or build up of unacceptable stresses that could result in performance problems. Also, vertical spacing limitations in the AASHTO LRFD Specifications for MSE walls apply to walls designed using the K-Stiffness method.

13. Determine the length of the reinforcement required in the resisting zone by comparing the factored $T_{\text{max}}$ value to the factored pullout resistance available as calculated per the AASHTO LRFD Specifications. If the length of the reinforcement required is greater than desired (typically, the top of the wall is most critical), decrease the spacing of the reinforcement, recalculate the global wall stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.
17.5.3.10.6 Strength Limit State Design for Internal Stability Using the K-Stiffness Method – Steel Reinforced Walls

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

1. Select a trial reinforcement spacing and steel area that is based on end-of-construction (EOC) conditions (i.e., no corrosion). Once the trial spacing and steel area have been selected, the reinforcement layer stiffness on a per ft of wall width basis, $J_{EOC}$, and wall global stiffness, $S_{global}$, can be calculated (Equation 17-4). Note that at this point in the design, it does not matter how one obtains the reinforcement spacing and area. They are simply starting points for the calculation. Also select a design soil friction angle to calculate $K$ (see Section 17.5.3.9.1). Note that for steel reinforced wall systems, the reinforcement loads are not as strongly correlated to the peak plane strain soil friction angle as are the reinforcement loads in geosynthetic walls (Allen and Bathurst, 2003). This is likely due to the fact that the steel reinforcement is so much stiffer than the soil. The K-Stiffness Method was calibrated to a mean value of $K_o$ of 0.3 (this results from a plane strain soil friction angle of 44°, or from triaxial or direct shear testing a soil friction angle of approximately 40°). Therefore, soil friction angles higher than 44° shall not be used. Lower design soil friction angles should be used for weaker granular backfill materials.

2. Begin by checking the strength limit state for backfill soil failure. The goal is to select a reinforcement density (spacing, steel area) that is great enough to keep the steel reinforcement load below yield ($A_s F_y R_c / b$, which is equal to $A_s F_y S_h$). $F_y$ is the yield stress for the steel, $A_s$ is the area of steel before corrosion (EOC conditions), and $S_h$ is the horizontal spacing of the reinforcement (use $S_h = 1.0$ for continuous reinforcement). Depending on the ductility of the steel, once the yield stress has been exceeded, the steel can deform significantly without much increase in load and can even exceed the strain necessary to cause the soil to reach a failure condition. For this reason, it is prudent to limit the steel stress to $F_y$ for this limit state. Tensile tests on corroded steel indicate that the steel does not have the ability to yield to large strains upon exceeding $F_y$, as it does in an uncorroded state, but instead fails in a brittle manner (Terre Armee, 1979). Therefore, this limit state only needs to be evaluated for the steel without corrosion effects.

3. Using the trial steel area and global wall stiffness from Step 1, calculate the factored $T_{max}$ for each reinforcement layer using Equations 17-2 and 17-3.

4. Apply an appropriate resistance factor to $A_s F_y S_h$ to obtain the factored yield strength for the steel reinforcement. Then compare the factored load to the factored resistance, as shown in Equation 17-20 below. If the factored load is greater than the factored yield strength, then increase $A_s$ and recalculate the global wall stiffness and $T_{max}$. Make sure that the factored yield strength is greater than the factored load before going to the next limit state calculation. In general, this limit state will not control the design. If the yield strength available is well in excess of the factored load, it may be best to wait until the...
strength required for the other limit states has been determined before reducing the amount of reinforcement in the wall. Check to see that the factored reinforcement load \( T_{\text{max}} \) is greater than or equal to the factored yield resistance as follows:

**Equation 17-20**

\[
T_{\text{max}} \leq \frac{A_s F_y}{b} R_c \phi_{sf} = \frac{A_s F_y}{S_h} \phi_{sf}
\]

where:
- \( \phi_{sf} \) is the resistance factor for steel reinforcement resistance at yield, and \( S_h \) is the horizontal spacing of the reinforcement. For wire mesh, and possibly some welded wire mats with large longitudinal wire spacing, the stiffness of the reinforcement macrostructure could cause the overall stiffness of the reinforcement to be significantly less than the stiffness of the steel itself. In-soil pullout test data may be used in that case to evaluate the soil failure limit state, and applied to the approach provided for soil failure for geosynthetic walls (see Equation 17-16 in Step 5 for geosynthetic wall design).

5. Next, check the strength limit state for reinforcement rupture in the backfill. The focus of this limit state is to ensure that the long-term rupture strength of the reinforcement is greater than the load calculated from the K-Stiffness Method. Even though the focus of this calculation is at the end of the service life for the wall, the global stiffness for the wall should be based on the stiffness at the end of wall construction, as reinforcement loads do not decrease because of lost cross-sectional area resulting from reinforcement corrosion. \( T_{\text{max}} \) obtained from Step 5 should be an adequate starting point for this limit state calculation.

6. Calculate the strength of the steel reinforcement at the end of its service life, using the ultimate strength of the steel, \( F_u \), and reducing the steel cross-sectional area, \( A_s \), determined in Step 5, to \( A_c \) to account for potential corrosion losses. Then use the resistance factor \( \phi_{rr} \), as defined previously, to obtain the factored long-term reinforcement tensile strength such that \( T_{\text{al}} \) is greater than or equal to \( T_{\text{max}} \), as shown below:

**Equation 17-21**

\[
T_{\text{al}} = \frac{F_u A_c}{S_h} \phi_{rr}
\]

and

**Equation 17-22**

\[
T_{\text{max}} \leq \frac{F_u A_c}{b} R_c \phi_{rr} = \frac{F_u A_c}{S_h} \phi_{rr}
\]

where:
- \( F_u \) is the ultimate tensile strength of the steel, and \( A_c \) is the steel cross-sectional area per ft. of wall length reduced to account for corrosion loss. The resistance factor is dependent
on the variability in \( F_u \), \( A_s \), and the amount of effective steel cross-sectional area lost as a result of corrosion. As mentioned previously, minimum specification values are typically used for design with regard to \( F_u \) and \( A_s \). Furthermore, the corrosion rates provided in the AASHTO LRFD Specifications are also maximum rates based on the available data (Terre Armee, 1991). Recent post-mortem evaluations of galvanized steel in reinforced soil walls also show that AASHTO design specification loss rates are quite conservative (Anderson and Sankey, 2001). Furthermore, these corrosion loss rates have been correlated to tensile strength loss, so that strength loss due to uneven corrosion and pitting is fully taken into account. Therefore, the resistance factor provided in Table 17-13, which is based on the variability of the un-aged steel, is reasonable to use in this case, assuming that non-aggressive backfill conditions exist.

If \( T_{al} \) is not equal to or greater than \( T_{max} \), increase the steel area, recalculate the global wall stiffness on the basis of the new value of \( A_s \), reduce \( A_s \) for corrosion to obtain \( A_c \), and recalculate \( T_{max} \) until \( T_{al} \) based on Equation 17-22 is adequate to resist \( T_{max} \).

7. If the steel reinforcement is connected directly to the wall facing (this does not include facings that are formed by simply extending the reinforcement mat), the reinforcement strength needed to provide the required long-term connection strength must be determined. This connection capacity, reduced by the appropriate resistance factor, must be greater than or equal to the factored reinforcement load at the connection. If not, increase the amount of reinforcing steel in the wall, recalculate the global stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.

8. Determine the length of reinforcement required in the resisting zone by comparing the factored \( T_{max} \) value to the factored pullout resistance available as calculated per Section 11 of the AASHTO LRFD specifications. If the length of reinforcement required is greater than desired (typically, the top of the wall is most critical), decrease the spacing of the reinforcement, recalculate the global wall stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.

**17.5.3.10.7 Combining Other Loads With the K-Stiffness Method Estimate of \( T_{max} \) for Internal Stability Design**

**Seismic Loads** – Seismic design of MSEs walls when the K-Stiffness Method is used for internal stability design shall be conducted in accordance with Articles 11.10.7.2 and 11.10.7.3 of the AASHTO LRFD Specifications, except that the static portion of the reinforcement load is calculated using the K-Stiffness Method. The seismic load resulting from the inertial force of the wall active zone within the reinforced soil mass (\( T_{md} \) in AASHTO LRFD Article 11.10.7.3) is added to \( T_{max} \) calculated using the K-Stiffness Method by superposition. A load factor of 1.0 for the load combination (static plus seismic), and the resistance factors for combined seismic and static loading provided in Table 17-13 shall be used for this Extreme Event Limit State.

**Concentrated Surcharges and Traffic Barrier Impact Loads** – The load increase at each reinforcement layer resulting from the concentrated surcharge and traffic barrier impact loads calculated as specified in the AASHTO LRFD Design Specifications, Articles 3.11.6.3 and 11.10.10 and NYSDOT GDM Sections 17.5.3.4 and 17.4.15, shall be added to the K-Stiffness calculation of \( T_{max} \) by superposition at each affected reinforcement level, considering the
tributary area of the reinforcement. The load factor used for each load due to the surcharge or traffic impact load shall be as specified in the AASHTO LRFD Bridge Design Specifications.

17.5.3.10.8 Design Sequence Considerations for the K-Stiffness Method

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

For example, for geosynthetic reinforced wrap-faced walls, with or without a concrete facia placed after wall construction, the reinforcement needed to prevent soil failure will typically control the global reinforcement stiffness needed, while pullout capacity is generally not a factor, and connection strength is not applicable. For modular concrete block-faced or precast panel-faced geosynthetic walls, the connection strength needed is likely to control the global reinforcement stiffness. However, it is also possible that reinforcement rupture or soil failure could control instead, depending on the magnitude of the stiffness of a given reinforcement product relative to the long-term tensile strength needed. The key here is that the combination of the required stiffness and tensile strength be realistic for the products available.

Generally, pullout will not control the design unless reinforcement coverage ratios are low. If reinforcement coverage ratios are low, it may be desirable to evaluate pullout early in the design process. For steel strip, bar mat, wire ladder, and polymer strap reinforced systems, pullout often controls the reinforcement needed because of the low reinforcement coverage ratios used, especially near the top of the wall. However, connection strength can also be the controlling factor. For welded wire wall systems, the tensile strength of the reinforcement usually controls the global wall reinforcement stiffness needed, though if the reinforcement must be connected to the facing (i.e., the facing and the reinforcement are not continuous), connection strength may control instead. Usually, coverage ratios are large enough for welded wire systems (with the exception of ladder strip reinforcement) that pullout is not a controlling factor in the determination of the amount of reinforcement needed. For all steel reinforced systems, with the possible exception of steel mesh reinforcement, the soil failure limit state does not control the reinforcement design because of the very low strain that typically occurs in steel reinforced systems.
17.5.4 Geosynthetically Reinforced Soil System (GRSS) Slopes

Geosynthetically reinforced systems consist of geogrid or geotextile materials arranged in horizontal planes in the backfill to resist outward movement of the reinforced soil mass.

![Typical Geosynthetically Reinforced Soil System Slope](image)

**Figure 17-38 Typical Geosynthetically Reinforced Soil System Slope**

Reinforced slopes do not have a height limit but they do have a face slope steepness limit. Reinforced slopes steeper than 1V on 0.5H shall be considered to be a wall and designed as such. Reinforced slopes with a face slope steeper than 1V on 1H shall have a wrapped face or a welded wire slope face, but should be designed as a reinforced slope.

17.5.4.1 Geosynthetically Reinforced Soil System Slope Components

System components include:

- **Foundation**: a stable soil or bedrock upon which the slope is constructed. Stability in the foundation is assumed.
- **Retained Soil**: The soil which remains in place beyond the limits of the excavation.
- **Subsurface Drainage**: Drainage medium (granular material or prefabricated geocomposite) installed at the limits of the reinforced soil zone to control and collect ground water seepage.
- **Reinforced Soil**: The soil which is placed in lifts adjacent to the retained soil and incorporates horizontal layers of reinforcing to create the sloped structure.
- **Reinforcement**: Geosynthetic (geogrid or geotextile) with sufficient strength and soil
compatible modulus, placed horizontally within the slope to provide tensile forces to resist instability.

- Surface Protection: The erosion resistant covering of the finished slope surface.

### 17.5.4.2 Criteria for Geosynthetic Reinforcement Design

**Figure 17-39 Criteria for Geosynthetic Reinforcement Design**

**17.5.4.3 Safety Factors**

Recommended minimum stability factors of safety are:

- Greater than or equal to 1.5 against horizontal sliding of the reinforced mass along its base;
- Greater than or equal to 1.3 against external, deep-seated failures;
- Greater than or equal to 1.3 against compound failure surfaces; and
- Greater than or equal to 1.3 against internal failure.

### 17.5.4.4 Design of Reinforced Slopes

The design of reinforced slopes is contained in the FHWA Geotextile Engineering Manual (Christopher and Holts, 1985). Two recommended methods from that manual are detailed in the

The backfill requirements are provided in the NYSDOT Standard Specifications.

The final recommended design is to be based on a layout that is a compromise between the requirement to minimize the number of layers and the desire to keep the layout as simple as possible to ease construction.

17.5.4.4.1 Primary Geosynthetic Reinforcement

The geosynthetic reinforcement (geogrids or geotextiles) used in slopes must satisfy both strength and soil interaction requirements. The strength requirements focus on the tensile modulus and the long-term strength of the reinforcement. The long-term design strength is also known as the Long-Term Design Tensile Strength (T_D).

The T_D Load is determined by applying partial factors of safety of the ultimate tensile strength of the reinforcement to account for creep, chemical and biological resistance and installation damage:

\[
T_D = \frac{T_{ULT}}{FS_{CR} \times FS_{CD} \times FS_{DU}}
\]

where:
- \(T_D\) = Long-Term Design Tensile Strength
- \(T_{ULT}\) = Ultimate Tensile Strength – determined in the primary strength direction in accordance with ASTM D4595
- \(FS_{CR}\) = Factor of Safety for Creep Deformation for 100 Year Design Life – calculated in accordance with Geosynthetic Research Institute Standard Practice GRI-GG4 using ASTM Standard Test Methods D5262 to determine long-term strength, \(T_{LT}\) and D4595 to determine short-term strength, \(T_{ST}\).
- \(FS_{CD}\) = Factor of Safety for Construction Damage – calculated in accordance with Geosynthetic Research Institute Standard Practice GRI-GG4.
- \(FS_{DU}\) = Factor of Safety for Durability – determined in accordance with EPA9090 and ASTM D4595. The minimum tested \(FS_{DU}\) value permitted is 1.10.

In lieu of testing, the following default factors of safety apply:

- \(FS_{CR}\) = 5.0 (HDPE); 5.0 (PP); 2.5 (PETP)
- \(FS_{CD}\) = 2.0
- \(FS_{DU}\) = 2.0
17.5.4.4.2 Secondary Geosynthetic Reinforcement

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength (In direction perpendicular</td>
<td>ASTM D4595</td>
<td></td>
</tr>
<tr>
<td>to slope face)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. 5% strain</td>
<td></td>
<td>500 lbs/ft.</td>
</tr>
<tr>
<td>b. Ultimate T_{ULT}</td>
<td></td>
<td>1000 lbs/ft.</td>
</tr>
</tbody>
</table>

Table 17-14 Secondary Geosynthetic Reinforcement Properties

The secondary reinforcement shall extend 3 ft. to 5 ft. back into the fill, from the face.

17.5.4.5 Reinforced Slope Facings

Reinforced slopes should be vegetated after construction to prevent or minimize erosion due to rainfall and runoff on the face. Geosynthetic reinforced slopes inherently can be difficult sites to establish and maintain vegetative cover due to the steep grades that can be achieved. The steepness of the grade limits the amount of water absorbed by the soil before runoff occurs. Once vegetation is established on the face, it must be protected to insure long-term survival.

Slopes steeper than 1V on 1H typically required facing support during construction. Removable facing supports or left-in-place welded wire mesh forms are typically used. A permanent facing such as gunite or emulsified asphalt may be applied to the slope face to provide long-term ultra violet protection, if the geosynthetic UV resistance is not adequate for the lift of the structure.

A synthetic (permanent) erosion control mat that is stabilized against ultra-violet light and is inert to naturally occurring soil-born chemicals and bacteria may be required. The erosion control mat serves three functions: 1) protects the base soil face against erosion until vegetation is established, 2) reduces runoff velocity for increased water absorption by the soil thus promoting long-term survival of the vegetative cover, and (3) reinforces the root system of the vegetative cover. Maintenance of vegetation may be required.

A permanent synthetic mat may not be required in applications characterized by flatter slopes (less than 1V on 1H), low height slopes, and/or moderate runoff.

17.5.4.6 Reinforced Slope Drainage

Uncontrolled subsurface water seepage will decrease stability of slopes. Special emphasis on the design and construction of subsurface drainage features is essential where drainage is critical for maintaining slope stability. Redundancy in the drainage system may be required.
### Geosynthetically Reinforced Soil System Slope and Wall Preliminary Design Guidelines

<table>
<thead>
<tr>
<th>GRSS Height</th>
<th>GRSS Sloped Face</th>
<th>1V on 1.5 H</th>
<th>1V on 1H</th>
</tr>
</thead>
<tbody>
<tr>
<td>H = 5 ft. (1.5 m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>T</td>
<td>L</td>
<td>T</td>
</tr>
<tr>
<td>1 ft. (0.3 m)</td>
<td>66 lbs/ft. (1 kN/m)</td>
<td>5.6 ft. (1.7 m)</td>
<td>122 lbs/ft. (1.8 kN/m)</td>
</tr>
<tr>
<td>2 ft. (0.6 m)</td>
<td>121 lbs/ft. (1.8 kN/m)</td>
<td>218 lbs/ft. (3.2 kN/m)</td>
<td></td>
</tr>
<tr>
<td>3 ft. (1.0 m)</td>
<td>166 lbs/ft. (2.4 kN/m)</td>
<td>290 lbs/ft. (4.2 kN/m)</td>
<td></td>
</tr>
<tr>
<td>H = 10 ft. (3.0 m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>T</td>
<td>L</td>
<td>T</td>
</tr>
<tr>
<td>1 ft. (0.3 m)</td>
<td>111 lbs/ft. (1.6 kN/m)</td>
<td>9.3 ft. (2.8 m)</td>
<td>212 lbs/ft. (3.1 kN/m)</td>
</tr>
<tr>
<td>2 ft. (0.6 m)</td>
<td>211 lbs/ft. (3.1 kN/m)</td>
<td>400 lbs/ft. (5.8 kN/m)</td>
<td></td>
</tr>
<tr>
<td>3 ft. (1.0 m)</td>
<td>299 lbs/ft. (4.4 kN/m)</td>
<td>561 lbs/ft. (8.2 kN/m)</td>
<td></td>
</tr>
<tr>
<td>H = 15 ft. (4.5 m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>T</td>
<td>L</td>
<td>T</td>
</tr>
<tr>
<td>1 ft. (0.3 m)</td>
<td>158 lbs/ft. (2.3 kN/m)</td>
<td>13.0 ft. (4.0 m)</td>
<td>304 lbs/ft. (4.4 kN/m)</td>
</tr>
<tr>
<td>2 ft. (0.6 m)</td>
<td>305 lbs/ft. (4.5 kN/m)</td>
<td>583 lbs/ft. (8.5 kN/m)</td>
<td></td>
</tr>
<tr>
<td>3 ft. (1.0 m)</td>
<td>440 lbs/ft. (6.4 kN/m)</td>
<td>837 lbs/ft. (12.2 kN/m)</td>
<td></td>
</tr>
<tr>
<td>H = 20 ft. (6.0 m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>T</td>
<td>L</td>
<td>T</td>
</tr>
<tr>
<td>1 ft. (0.3 m)</td>
<td>206 lbs/ft. (3.0 kN/m)</td>
<td>16.8 ft. (5.1 m)</td>
<td>397 lbs/ft. (5.8 kN/m)</td>
</tr>
<tr>
<td>2 ft. (0.6 m)</td>
<td>400 lbs/ft. (5.8 kN/m)</td>
<td>768 lbs/ft. (11.2 kN/m)</td>
<td></td>
</tr>
<tr>
<td>3 ft. (1.0 m)</td>
<td>581 lbs/ft. (8.5 kN/m)</td>
<td>1113 lbs/ft. (16.2 kN/m)</td>
<td></td>
</tr>
<tr>
<td>H = 25 ft. (7.5 m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>T</td>
<td>L</td>
<td>T</td>
</tr>
<tr>
<td>1 ft. (0.3 m)</td>
<td>253 lbs/ft. (3.7 kN/m)</td>
<td>20.6 ft. (6.3 m)</td>
<td>489 lbs/ft. (7.1 kN/m)</td>
</tr>
<tr>
<td>2 ft. (0.6 m)</td>
<td>495 lbs/ft. (7.2 kN/m)</td>
<td>953 lbs/ft. (13.9 kN/m)</td>
<td></td>
</tr>
<tr>
<td>3 ft. (1.0 m)</td>
<td>724 lbs/ft. (10.6 kN/m)</td>
<td>1390 lbs/ft. (20.3 kN/m)</td>
<td></td>
</tr>
<tr>
<td>H = 30 ft. (9.0 m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>T</td>
<td>L</td>
<td>T</td>
</tr>
<tr>
<td>1 ft. (0.3 m)</td>
<td>301 lbs/ft. (4.4 kN/m)</td>
<td>24.5 ft. (7.5 m)</td>
<td>582 lbs/ft. (8.5 kN/m)</td>
</tr>
<tr>
<td>2 ft. (0.6 m)</td>
<td>591 lbs/ft. (8.6 kN/m)</td>
<td>1138 lbs/ft. (16.6 kN/m)</td>
<td></td>
</tr>
<tr>
<td>3 ft. (1.0 m)</td>
<td>868 lbs/ft. (12.7 kN/m)</td>
<td>1668 lbs/ft. (24.3 kN/m)</td>
<td></td>
</tr>
</tbody>
</table>

**Table 17-15 Geosynthetically Reinforced Soil System Slope Preliminary Design Guidelines**

S = Approximate spacing between the layers
T = Design Tensile Strength
L = Length

*Note: Secondary reinforcement is not shown and may be required.*
### GRSS Height

<table>
<thead>
<tr>
<th>H Height (ft. (m))</th>
<th>Vertical Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 ft. (1.5 m)</td>
<td>1.5 ft. (450 mm)</td>
</tr>
<tr>
<td>10 ft. (3.0 m)</td>
<td>1.5 ft. (450 mm)</td>
</tr>
<tr>
<td>15 ft. (4.5 m)</td>
<td>1.5 ft. (450 mm)</td>
</tr>
<tr>
<td>20 ft. (6.0 m)</td>
<td>1.5 ft. (450 mm)</td>
</tr>
<tr>
<td>25 ft. (7.5 m)</td>
<td>1.5 ft. (450 mm)</td>
</tr>
<tr>
<td>30 ft. (9.0 m)</td>
<td>1.5 ft. (450 mm)</td>
</tr>
</tbody>
</table>

#### Design Assumptions:
1. Stable Foundation
2. Pore pressures do not exist within stabilized mass.
3. F.S. = 1.3
4. Surcharge = 250 psf (12.0 kPa)
5. Backfill properties:
   - Unit Weight = 125pcf (20 kN/m³)
   - Internal Angle of Friction = 30°
   - Cohesion = 0 psf (0 kPa)
6. Soil/Geogrid Interaction Coefficient = 0.8
7. Direct Sliding Coefficient = 1.0
   
   **Ref:** GEB memorandum dated December 4, 1995

#### Table 17-16 Geosynthetically Reinforced Soil System Wall

**Preliminary Design Guidelines**

#### 17.5.4.7 Reinforced Slope Construction

Standard Sheet 554-02 *Geosynthetically Reinforced Soil Systems* provides installation requirements.

#### 17.5.5 Prefabricated Wall System

Prefabricated Wall Systems shall be designed in accordance with gravity wall methodology and the wall-slope combination shall be stable regarding overall stability as determined per NYSDOT GDM Chapter 10.

According to NYSDOT Standard Specifications and stated in Section 17.5.3.8, a Prefabricated Wall System (PWS) is either

- a series of prefabricated wall system open top face units assembled to form bins which are connected in unbroken sequence, or
- a combination of specific prefabricated wall system solid face units with a characteristic alignment and connection method, which utilize the weight of the wall system elements and the weight of the infill to resist lateral soil pressure.

An open top unit is a face unit which has an open structure to allow placement of infill material, utilizing the weight of the wall system elements and the weight of the infill to resist lateral soil...
pressure. Open top units are typically manufactured via the wet casting (precast) process.

A solid unit is a face unit which has a solid mass, utilizing the weight of the wall system elements to resist lateral soil pressure. A solid unit may require some infill material depending on the geometric bevel of the units. Solid face units are typically manufactured via the dry casting process.

A PWS is an externally stabilized fill structure comprised of prefabricated face units & coping units, including leveling pads, unit infill, earth backfill, joint filler material and geotextile, and a subsurface drainage system to reduce hydrostatic pressure on the wall system.

All PWS's are designed by the Wall System Designer. Approved PWS’s appear on the NYSDOT Approved List. When reinforcement is introduced to a PWS, they shall be reclassified as Mechanically Stabilized Wall Systems and the pertinent sections of the specification shall apply.

17.5.6 Geocell Walls

Geocell walls shall be designed in accordance with gravity wall methodology and the wall-slope combination shall be stable regarding overall stability as determined per NYSDOT GDM Chapter 10.

A geocell is a three-dimensional, permeable polymeric honeycomb or web structure of expandable panels used to confine fill materials to create structural stability.

Figure 17-41 Geocell Expanded Honeycomb Features
Geocells are manufactured from High Density Polyethylene (HDPE). When expanded during installation, the interconnected strips form the walls of a flexible, three-dimensional cellular structure into which specified infill materials are placed and compacted. Typical applications utilize infill material conforming to §733-14 Select Structural Fill, with the added stipulation that the maximum particle size is 2 in. The outermost cells are identified to be filled with topsoil meeting the material requirements of §713-01 Topsoil.

The geocell arrangement creates a free-draining system that holds infill materials in place and prevents mass movements by providing confinement through tensile reinforcement. Cellular confinement systems improve the structural and functional behavior of soils and aggregate infill materials. The geocell mass may be extended to become part of a larger wall system of several geocell interconnected strips tied together to form a structure.

For ease of construction, the Geotechnical Engineering Bureau has developed a detail sheet for a geocell stretching frame (see Figure 17-44).
CHAPTER 17
Abutments, Retaining Walls, and Reinforced Slopes

Figure 17-44 Geocell Stretching Frame
17.5.7 Gabion Walls

Gabion walls shall be designed in accordance with gravity wall methodology and the wall-slope combination shall be stable regarding overall stability as determined per NYSDOT GDM Chapter 10.

A gabion (Figure 17-45) is a heavy duty, galvanized steel wire mesh basket, in the shape of a box, that is divided by diaphragms into cells. Filled with heavy materials (typically rocks or broken concrete) that cannot escape through the mesh openings, it generally is used as a construction block, becoming part of a larger unit of several gabions tied together to form a structure.

![Figure 17-45 Typical Gabion Baskets](image)

The stone sizes for the baskets are identified in the NYSDOT Standard Specifications.

Advantages:
- Blend very well with natural surroundings.
- Preformed baskets are light and easy to work with during construction.
- Can sustain differential settlements without serious distress.

Disadvantages:
- If quality backfill material is not available within a reasonable distance, may not be economically feasible.
• Overlying facilities may distort.

17.5.7.1 Applications

Principal uses include:
• Channel linings and bank protection.
• Retaining Walls.
• Bridge abutment protection.
• Culverts (headwalls and outlet protection).

Gabions form free draining vertical, stepped or battered retaining structures that often do not require a special foundation. The flexibility of gabion walls allows them to withstand foundation movements and differential settlements.

These features are especially advantageous in constructing a range of retaining walls for general earth retention, bridge abutments (for short or light duty bridges as well as wingwalls and end protection for culvert structures.

Wall design typically follows gravity retaining methods or reinforced soil structure methods. For the latter (Figure 17-46 (c)) layers of the same mesh used for gabion fabrication provide the reinforcement.

![Figure 17-46 Typical Cross Sections for a) Gravity Gabion Wall (Stepped); b) Semi-Gravity Gabion Wall; and c) Reinforced Soil Gabion Wall](image)

Experience indicates that rupture of the wire baskets may be a problem for walls in the 23 ft. to 30 ft. height range. This problem may be the result of localized over-stressing due to excessive differential settlement, or it may be due to wire damage during construction caused by dropping rock fragments. Post-construction damage resulting from stream load abrasion or vandalism may also be a problem. Therefore, gabion walls in the 23 ft. to 30 ft. range should be closely monitored during an after construction.
17.5.8 Rock Walls

Rock walls shall be designed in accordance with the gravity wall methodology and the wall-slope combination shall be stable regarding overall stability as determined per NYSDOT GDM Chapter 10.

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 (ft)</th>
<th>Minimum Rock Size</th>
<th>Typical Rock Weight (lbs)</th>
<th>Average Dimension (in.)</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>

Table 17-17 Minimum Rock Sizes for Rock Walls

Rock walls shall be 12 ft or less in total height. Rock walls used to retain fill shall be 6 ft or less in total height if the rocks are placed concurrent with backfilling. Rock walls up to 12 ft in height may be constructed in fill if the fill is overbuilt and then cut back to construct the wall. Fills constructed for this purpose shall be compacted to 95 percent maximum density, per NYSDOT Standard Specifications.

17.5.9 Soil Nail Walls

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


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

Geotechnical Engineering Manual (GEM-21) *Design and Construction Guidelines for a Soil Nail Wall Systems* provides further information on the design and construction of a soil nail wall.

For external stability and compound stability analysis, as described in NYSDOT GDM Section 17.5.3.3 and the AASHTO LRFD Bridge Design Specifications, limit equilibrium slope stability programs as described in NYSDOT GDM Chapter 10 should be used. The program S-Nail also has the ability to conduct compound stability analyses and may be used for this type of analysis as well.

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

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

The Departmental Geotechnical Engineer 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.

### 17.5.9.1 Soil Nail Wall Concepts and Components

Soil nailing is a technique used to reinforce and strengthen existing ground. The fundamental concept of soil nailing is that soil can be effectively reinforced by installing closely spaced grouted steel bars, called "nails," into a slope or excavation as construction proceeds from the top down. Nails are commonly referred to as "passive" inclusions. The term "passive" means that the nails are not pre-tensioned (as are tiebacks) when they are installed. The nail bars are forced into tension as the ground deforms laterally in response to the loss of support caused by continued excavation. The grouted nails increase the shear strength of the overall soil mass and limit displacement during and after excavation. They act similarly to reinforcing strips in a Reinforced Earth® structure. A structural facing connected to the nails is used when the slope angle exceeds a predetermined critical value or when environmental conditions would cause deterioration of the exposed soil face over its design life.
Soil nailing is used to retain vertical, battered, or stepped excavations. Typical details of this wall type are shown in Figures 17-47 and 17-48. Soil nails are bonded along their full length and are not constructed with a permanent unbonded length as are tieback anchors.

Soil nail wall construction is sensitive to ground conditions, construction methods, equipment and excavation sequencing. Soil nailing cannot be used in all types of ground. For soil nail walls to be most economical, they should be constructed in ground that can stand unsupported on a vertical or steeply sloped cut of 3 ft. to 6 ft. for at least one to two days, and that can maintain an open drill hole for at least several hours.

Figure 17-47 Soil Nail Wall Typical Section

A typical soil nail consists of a deformed steel reinforcing bar (generally Grade 60), also called a tendon, which is inserted into a pre-drilled, straight shafted, drill hole generally ranging from 4 in. to 12 in. in diameter. After the nail tendon is inserted, the drill hole is completely filled with structural grout pumped under low-pressure via a tremie pipe ("open hole" installation). The grout "bonds" the nail tendon to the surrounding ground.

Soil nails are typically installed by "open hole" methods using a variety of drilling equipment, usually augers. The open-hole method is used to install 80 to 90 percent of drilled and grouted soil nails. “Cased” or “auger cast” installation methods can be used when caving soil conditions are encountered.

The nail tendon is available in various diameters. Nail tendons should be installed as single units
without couplers (unless couplers are allowed by the project specifications). Couplers are typically allowed for temporary extension of the nail tendon during testing. Welding is not permissible. Permanent soil nail systems require corrosion protection of the tendon.

The nail spacing should be no less than 3 ft. vertical and 3 ft. horizontal. In very dense glacially over consolidated soils, horizontal nail spacing should be no greater than 8 ft. and vertical nail spacing should be no greater than 6 ft. In all other soils, horizontal and vertical nail spacing should be 6 ft. 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 ft. 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. All other nails shall be epoxy coated unless the wall is temporary.

For nail testing, a minimum bond length and a minimum unbonded length of 5 ft. is required. Nail testing shall be in accordance with the NYSDOT Standard Specifications.
Drainage is considered a critical element, and is incorporated into all permanent walls and many temporary walls.

Face drainage is the type most commonly used and usually consists of prefabricated vertical geocomposite drainage strips installed for the top to the bottom as the excavation proceeds downward. The drainage strips are typically 12 in. to 18 in. wide, and are centered between the vertical nail columns. The strips are connected to weep hole outlet pipes and to a footing drain at the wall base.

A surface water collector ditch is usually placed behind the top of the wall to prevent surface runoff from either recharging the ground behind the wall or flowing over the top of wall.

Typically, temporary soil nail facing consists of 3 in. to 4 in. of shotcrete reinforced with a single layer of welded wire mesh. The temporary shotcrete facing is placed concurrently with the excavation each lift. For permanent wall facings, a soil nail wall may consist of full-thickness shotcrete, CIP concrete over temporary shotcrete, or precast concrete panels over shotcrete. Permanent facings consisting of CIP concrete are placed over the shotcrete following completion of the excavation to full height. The soil nail head/bearing plate must be structurally connected to the permanent wall, and the temporary shotcrete wall is sometimes considered sacrificial in the structural facing design.
Permanent shotcrete walls are constructed with either (1) the full-thickness shotcrete placed concurrently with each excavation lift or, (2) a second full-height shotcrete layer placed over the initial shotcrete layer following the excavation to full depth. This type of wall generally has a total thickness of 6 in. to 12 in., and is reinforced with reinforcing bars or welded wire mesh.

Advantages:

- Reported lower cost, due to relatively rapid installation of the unstressed inclusions (nails) which are considerable shorter than earth anchors and a relatively thin shotcrete or concrete facing.
- Light construction equipment is usually required to install nails. Grouting of the boreholes is generally accomplished by gravity. This feature may be of particular importance for sites with difficult access.
- Since there are a large number of nails, failure of any one may not detrimentally affect the stability of the system, as would be the case for a conventional tieback system.
- In heterogeneous soils with cobbles, boulders and weathered zones or hard rock zones, it offers the advantage of small diameter shorter drill holes for nail installation and eliminates the need for soldier pile installation which is costly under these conditions.
- Soil nailed structures are more flexible than conventional rigid structures. Consequently these structures can conform to the surrounding ground and withstand greater total and differential ground movements in all directions.
- Surface deflections can be controlled by the installation of additional nails or stressing in the upper level of nails to a small percentage of their working loads.

Disadvantages:

- Permanent underground easements may be required.
- Groundwater drainage systems may be difficult to construct and their long-term effectiveness is difficult to ensure.
- In urban areas, the closely spaced array of reinforcements may interfere with nearby utilities. In addition, horizontal displacement may be somewhat greater than with prestressed tiebacks, which may cause distortions to immediately adjoining structures.
- Nail capacity may not be economically developed in cohesive soils subject to creep, even at relatively low load levels.
- The long-term performance of shotcrete facings has not been fully demonstrated, particularly in areas subject to freeze-thaw cycles.

17.6 STANDARD CANTILEVER WALLS

Standard cantilever walls designs are provided in the Highway Design Manual. The internal stability design and the external stability design for overturning and sliding stability have already been completed, and the minimum required soil bearing pressure below the wall is identified. The Departmental Geotechnical Engineer shall assess whether or not a standard cantilever wall is geotechnically applicable and stable given the specific site conditions and constraints.
17.7 TEMPORARY CUT SLOPES AND SHORING

17.7.1 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 NYSDOT 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, NYSDOT 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 1V on 1H 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 1V on 1H 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 1V on 1H 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.

17.7.2 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. NYSDOT GDM Chapter 4 provides requirements for field exploration and testing for cut slope design, and NYSDOT GDM Section 17.3 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 NYSDOT GDM shall also be applied to temporary shoring and excavation design.

17.7.3 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.

17.7.3.1 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 NYSDOT 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 NYSDOT GDM. The design methodology, input parameters, and assumptions used must be clearly stated on the required submittals (see NYSDOT GDM Section 17.7.2).

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 MSES 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 NYSDOT GDM Chapters 10 and 13 shall be used and met, in addition to meeting the requirements of 29 CFR 1926 Subpart P Excavations.

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 affect 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.

17.7.3.2 Safety Factors/Resistance Factors

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

For design of cut slopes that are part of a temporary excavation, assuming that the cut slopes not supporting a structure, a factor of safety of 1.25 or more as specified in NYSDOT GDM Chapters 10 and 13, 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.

17.7.3.3 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.

17.7.3.4 Design Property Selection

The procedures provided in NYSDOT GDM Chapter 6 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 NYSDOT GDM Chapter 6).

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.

### 17.7.4 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 NYSDOT GDM. Federal regulations regarding temporary cut slopes are presented in Code of Federal Regulations (CFR) Part 29, Sections 1926 Subpart P. Key aspects of Subpart P with regard to temporary slopes are summarized below for convenience. To assure obtaining the most up to date requirements regarding temporary slopes, the CFR should be reviewed.

29 CFR 1925 Subpart P presents maximum allowable temporary cut slope inclinations based on soil or rock type, as shown in Table 17-18. The allowable slopes presented are applicable to cuts 20 ft or less in height. Occupational Safety & Health Administration (OSHA) requires that slope inclinations steeper than those specified by Subpart P or greater than or equal to 20 ft in height must be designed by a registered Professional Engineer.

<table>
<thead>
<tr>
<th>Soil or Rock Type</th>
<th>Maximum Allowable Temporary Cut Slopes (Less than 20 ft. Maximum Height)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stable Rock</td>
<td>Vertical</td>
</tr>
<tr>
<td>Type A Soil</td>
<td>1V on ¾ H</td>
</tr>
<tr>
<td>Type B Soil</td>
<td>1V on 1H</td>
</tr>
<tr>
<td>Type C Soil</td>
<td>1V on 1 ½ H</td>
</tr>
</tbody>
</table>

**Table 17-18 29 CFR 1926 Subpart P Allowable Temporary Cut Slopes**

**Type A Soil** – "Type A" means cohesive soils with an unconfined, compressive strength of 1.5 ton per square foot (tsf) (144 kPa) or greater. Examples of cohesive soils are: clay, silty clay, sandy clay, clay loam and, in some cases, silty clay loam and sandy clay loam. Cemented soils such as caliche and hardpan are also considered Type A. However, no soil is Type A if:
• The soil is fissured; or
• The soil is subject to vibration from heavy traffic, pile driving, or similar effects; or
• The soil has been previously disturbed; or
• The soil is part of a sloped, layered system where the layers dip into the excavation on a slope of four horizontal to one vertical (1V on 4H) or greater; or
• The material is subject to other factors that would require it to be classified as a less stable material.

**Type B Soil** – "Type B" means:
• Cohesive soil with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa); or
• Granular cohesionless soils including: angular gravel (similar to crushed rock), silt, silt loam, sandy loam and, in some cases, silty clay loam and sandy clay loam.
• Previously disturbed soils except those which would otherwise be classed as Type C soil.
• Soil that meets the unconfined compressive strength or cementation requirements for Type A, but is fissured or subject to vibration; or
• Dry rock that is not stable; or
• Material that is part of a sloped, layered system where the layers dip into the excavation on a slope less steep than four horizontal to one vertical (1V on 4H), but only if the material would otherwise be classified as Type B.

**Type C Soil** – "Type C" means:
• Cohesive soil with an unconfined compressive strength of 0.5 tsf (48 kPa) or less; or
• Granular soils including gravel, sand, and loamy sand; or
• Submerged soil or soil from which water is freely seeping; or
• Submerged rock that is not stable, or
• Material in a sloped, layered system where the layers dip into the excavation or a slope of four horizontal to one vertical (1V on 4H) or steeper.

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 supporting a structure or wall, regardless of height, shall also be designed by a registered Professional Engineer in accordance with the NYSDOT 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 ft. 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 Departmental Geotechnical Engineer or his representative to confirm adequate stability.
In general, the longer an excavation remains open, the more conservative one should be. This is the case for cohesive soils where their shear strength is time dependent. Excavated slope in cohesionless soils are subject to erosion from rainfall runoff. These slope could be protected by use of an appropriate geosynthetic (geotextile or geomembrane).

**17.7.5 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 in. 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 NYSDOT GDM Chapter 11 and Section 17.4.7.

In the case of cantilever walls, the resistance factor of 0.75 applied to the passive resistance accounts for variability in properties and other sources of variability, as well as the prevention of excess deformation to fully mobilize the passive resistance. The amount of deformation required to mobilize the full passive resistance typically varies from 2 to 6 percent of the exposed wall height, depending on soil type in the passive zone (AASHTO 2010).

**17.7.5.1 Performance Requirements for Temporary Shoring and Cut Slopes to Support Railroad Tracks**

The following items are to be included in the design and construction procedures for all permanent and temporary facilities adjacent to railroad tracks:

1. Footings for all piers, columns, walls or other facilities shall be located and designed so that nay temporary sheeting and shoring for support of adjacent track or tracks during construction will not be closer than toe of ballast slope (7’-5” is dimension from gage or rail to toe of ballast for tangent track; see dimensions on Standard Plan No. 700003B for dimensions on curved track).

2. When support of track or tracks is necessary during construction of above-mentioned facilities, interlocking steel sheeting adequately braced and designed to carry E-80 live load is required. Soldier piles and lagging will be required penetration of steel sheet
piling cannot be obtained or when dry, non-running, stable material will be encountered.

3. Exploratory trenches, 3 ft. deep and 15 in. wide in the form of an "H" with outside dimensions matching the outside of sheeting dimensions are to be hand dug, prior to placing and driving steel sheeting, in areas where railroad underground installations are known to exist. These trenches are for exploratory purposes only and are to be backfilled and the backfill compacted immediately. This work must be done in the presence of a Railroad Inspector.

4. Absolute use of track is required while driving sheeting adjacent to running track. Procedure for arranging for use of track shall be as outlined in project special provisions.

5. Cavities adjacent to sheet piling, created by driving of sheet piling, shall be filled with sand and any disturbed ballast must be restored and tamped immediately.

6. Sheet piling shall be cut off at top of tie during construction. After construction and backfilling has been completed, piling within 10 ft. from centerline of track, or when bottom of excavation is below a line extending at 1V on 1H slope from end of tie to point of intersection with sheeting, the sheeting shall be cut off 18 in. below existing ground line and left in-place.

7. Any excavation adjacent to track shall be covered and ramped and provided with barricades and warning lights as required by the railroad.

8. Final backfilling of excavation shall be as required by project specifications.

9. The Contractor is to advise the railroad of the time schedule of each operation and obtain approval of railroad for all work to be performed adjacent to tracks so that it may be properly supervised by railroad personnel.

10. All drawings for temporary sheeting and shoring shall be prepared and stamped by a registered Professional Engineer and shall be accompanied by complete design computations when submitted for approval.

11. Where physical conditions of design impose insurmountable restrictions requiring the placing of sheeting closer than specified above, the matter must be submitted to the Chief Engineer for approval of any modifications.

12. See SK-1 dated December 6, 1988 for sheetpiling loading requirements.
Figure 17-49 Temporary Sheeting Requirements for Railroads
17.7.6 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 NYSDOT GDM Chapter 17 and its referenced design specifications and manuals.

17.7.6.1 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.

17.7.6.1.1 MSE Walls

MSE walls shall be designed for internal and external stability in accordance with NYSDOT GDM Section 17.5.3 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 $T_{dl}$ values (i.e., long-term tensile strength) of geosynthetics, accounting for creep, durability, and installation damage identified in the Geosynthetics Specifier’s Guide may be used for temporary wall design purposes.

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.

17.7.6.1.2 Prefabricated Wall Systems

Prefabricated wall systems (no soil reinforcement) are discussed in NYSDOT GDM Section 17.5.4 and should be designed as gravity retaining structures. The units/blocks shall meet the requirements in the NYSDOT Standard Specifications. Implementation of this specification will reduce the difficulties associated with placing units/blocks in a tightly fitted manner. Large concrete units/blocks should not be placed along a curve. Curves should be accomplished by staggering the wall in one-half to one full unit/block widths.

17.7.6.2 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.

17.7.6.2.1 Excavation Protection System

In accordance with the NYSDOT Standard Specifications, excavation protection system (i.e. trench boxes) are not considered to be structural shoring, as they generally do not provide full lateral support to the excavation sides. The excavation protection item is not appropriate for use in supporting stage construction operations and trench boxes in general, 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 NYSDOT GDM Sections 17.7.3 and 15.7.4, the excavation slopes outside of the trench box shall be designed to be stable.

17.7.6.2.2 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. Details for ground anchors are provided on Bridge Detail sheet BD-EE-12E Excavation and Embankment Tieback Wall Details.

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.

17.7.6.2.3 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 NYSDOT GDM Sections 17.3 and 17.5.3 is pertinent to the design of temporary soldier pile walls. Details for soldier pile and lagging walls are provided on Bridge Detail sheet BD-EE-11E Excavation and Embankment Soldier Pile and Lagging Wall Sample Details.
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. Details for ground anchors are provided on Bridge Detail sheet BD-EE-12E Excavation and Embankment Tieback Wall Details.

### 17.7.6.2.4 Prefabricated Wall Systems

Prefabricated wall systems (e.g. precast concrete units or dry cast 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 units/blocks placed without excessive caving or slope failure. The temporary excavation slope required to construct the prefabricated wall system shall be designed in accordance with NYSDOT GDM Sections 17.7.3 and 17.7.4.

See NYSDOT GDM Section 17.7.6.1.2 for additional special requirements for the design of this type of wall.

### 17.7.6.2.5 Braced Cuts

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

Details for braced excavations are provided on Bridge Detail sheet BD-EE-10E Excavation and Embankment Braced Excavation Details.

### 17.7.6.2.6 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 NYSDOT GDM Sections 17.3 and 17.5.7 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.
17.7.6.3 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 Department, or with special approval by the Geotechnical Engineering Bureau and Office of Structures.

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

17.7.7 Shoring and Excavation Design Submittal Review Guidelines

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

1. Shoring System Geometry
   a. Has the shoring geometry been correctly developed, and all pertinent dimensions shown?
   b. Are the slope angle and height above and below the shoring wall shown?
   c. Is the correct location of adjacent structures, utilities, etc., if any are present, shown?

2. Performance Objectives for the Shoring System
   a. Is the anticipated design life of the shoring system identified?
   b. Are objectives regarding what the shoring system is to protect, and how to protect it, clearly identified?
   c. Does the shoring system stay within the constraints at the site, such as the right of way limits, boundaries for temporary easements, etc?

3. Subsurface conditions
   a. Is the soil/rock stratigraphy consistent with the subsurface geotechnical data provided in the contract boring logs?
   b. Did the contractor/shoring designer obtain the additional subsurface data needed to meet the geotechnical exploration requirements for slopes and walls as identified in NYSDOT GDM Chapters 13 and 17, respectively?
   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 NYSDOT GDM Chapter 6 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 NYSDOT 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 NYSDOT GDM, and meet or exceed the minimum requirements of the NYSDOT 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)?

17.8 REFERENCES


