Chapter

15

Retaining Structures

The Chapter is organized as follows (also see GDM Table of Contents for a complete list of section headings):

- **15.1 Introduction: General Information**
- **15.2 Retaining Wall Practices and Procedures:** Retaining wall categories and definitions, general steps in a retaining wall project, selection of retaining wall system types, proprietary designs, nonproprietary designs, unique wall designs, and details of contract documents.
- **15.3 Design Requirements: General Wall Design:** Design methods, wall face batter, horizontal and vertical alignment, tiered walls, back-to-back walls, walls-on-slopes, overall stability, static lateral earth pressure, compaction loads, construction loads, seismic design, minimum embedment, wall settlement, wall drainage, underground utilities, design life, corrosion, and Minor retaining wall systems.
- **15.4-15.13 Design Guidance: Specific Wall Types:** Design guidance specific to each type of retaining wall. For each specific wall type, topics such as geotechnical investigation requirements, selection criteria, wall location, geometry, and design requirements are covered.
- **15.14 References:** A list of useful references.
- **Appendix 15-A:** General Requirements for Proprietary Retaining Wall Systems
- **Appendix 15-B:** Preapproval Process and Submittal Requirements for Proprietary Retaining Wall Systems
- **Appendix 15-C:** Guidelines for Review of Proprietary Retaining Wall System Working Drawings and Calculations
- **Appendix 15-D:** Preapproved Proprietary Retaining Wall Systems

15.1 Introduction

Retaining structures are an important part of Oregon's transportation system. They are included in projects to minimize right of way needs, to reduce bridge lengths at water crossings and grade separations, to minimize construction in environmentally sensitive areas, and to accommodate construction on slopes.

The requirements described in Chapter 15 are based on the Design-Bid-Build method of contracting. ODOT also delivers projects with other contracting methods, such as Design-Build. Design-Build combines the design and construction phases of a project into a single contract. The Design-Build Request for Proposal (RFP) identifies the applicable standards, manuals, guidelines, and additional requirements. While there may be differences contracting methods, the governing design and construction standards are consistent.

Retaining structure performance specifications should reference Chapter 15, with modifications as necessary to fit the contracting method being used.

15.2 Retaining Wall Practices and Procedures

15.2.1 Retaining Wall Categories and Definitions

15.2.1.1 Retaining Wall Categories

The following retaining wall categories are used in this chapter: Bridge Abutment, Bridge Retaining Wall, Highway Retaining Wall, and Minor Retaining Wall. These categories assist in making decisions regarding retaining wall function, consequences of failure, design, asset management, drafting, and other ODOT practices and procedures. The criteria and guidance based on wall category are not intended to replace engineering analysis or sound engineering judgment—but only to ensure that wall design decisions are consistent, straightforward, and applied equally on all ODOT projects statewide.

The retaining wall categories presented above include "Bridge Retaining Walls" whose performance could adversely influence the stability of a bridge structure. The "Bridge Zone" is a simplified conservative boundary intended to allow quick and easy categorization of retaining walls for a variety of purposes (Figure 15-1). Retaining walls located partially or fully within the limits of the bridge zone shall by default be defined as "Bridge Retaining Walls" and subject to all applicable requirements in this chapter.

If it is determined that a retaining wall defined as a "Bridge Retaining Wall", by virtue of being located within the "Bridge Zone", does not actually influence the stability of the bridge, this default definition may be overridden by clearly identifying the retaining wall as a "Highway Retaining Wall" on the Project Plans. This change in wall category shall be adequately supported by calculations in the retaining wall calculation books.

The retaining wall categories and default definitions are included below:

Bridge Abutment: Defined as a structural element at the end of the bridge that supports the end of the bridge span, and provides lateral support for fill material on which the roadway rests, immediately adjacent to the bridge. A bridge abutment provides vertical, longitudinal, and/or transverse restraint through bridge bearings, shear keys, and/or an integral connection with the bridge superstructure.

A bridge abutment is considered to be part of the bridge, and is designed according to applicable sections of the "*ODOT Bridge Design and Drafting Manual (BDDM),"* the "*ODOT Geotechnical Design Manual (GDM),"* and the '*AASHTO LRFD Bridge Design Specifications (AASHTO LRFD)'*.

Wing walls that are monolithic with the bridge abutment are part of the bridge abutment.

In Chapter 15, the terms “end bent” and “abutment” are used interchangeably. On ODOT bridge drawings, however, all bridge support locations are referred to as “bents” and abutments are referred to as “end bents.”

Bridge Retaining Wall: A retaining wall that meets all of the following conditions:

1. The retaining wall is located partially or entirely within the Bridge zone (Figure 15-1).
2. The retaining wall does not meet the definition of bridge abutment.

Design and construction requirements for Bridge retaining walls must be consistent with those for the bridge, unless it is determined that the retaining wall does not influence the stability of the bridge as noted above.

Highway Retaining Wall: A retaining wall that meets all of the following conditions:

1. The wall is located entirely outside of the bridge zone (Figure 15-1).
2. The wall does not fully meet the definition of a Minor retaining wall.

Highway retaining walls shall not be located inside the Bridge Zone, unless the Agency EOR for the Bridge retaining wall determines that the retaining wall does not influence the stability of the bridge as noted above.

Minor Retaining Wall: A retaining wall that meets all of the following conditions:

1. The wall is located entirely outside of the bridge zone (Figure 15-1).
2. Wall height (H), does not exceed 4.0 feet at any point along the wall (Figure 15-2).
3. Wall fore slope and back slope are both flatter than 1V:4H within a horizontal distance of H, measured from the nearest point on the wall (Figure 15-2).
4. Surcharge loading is not allowed on the retaining wall back slope within a horizontal distance of H, measured from the nearest point on the wall (Figure 15-2).

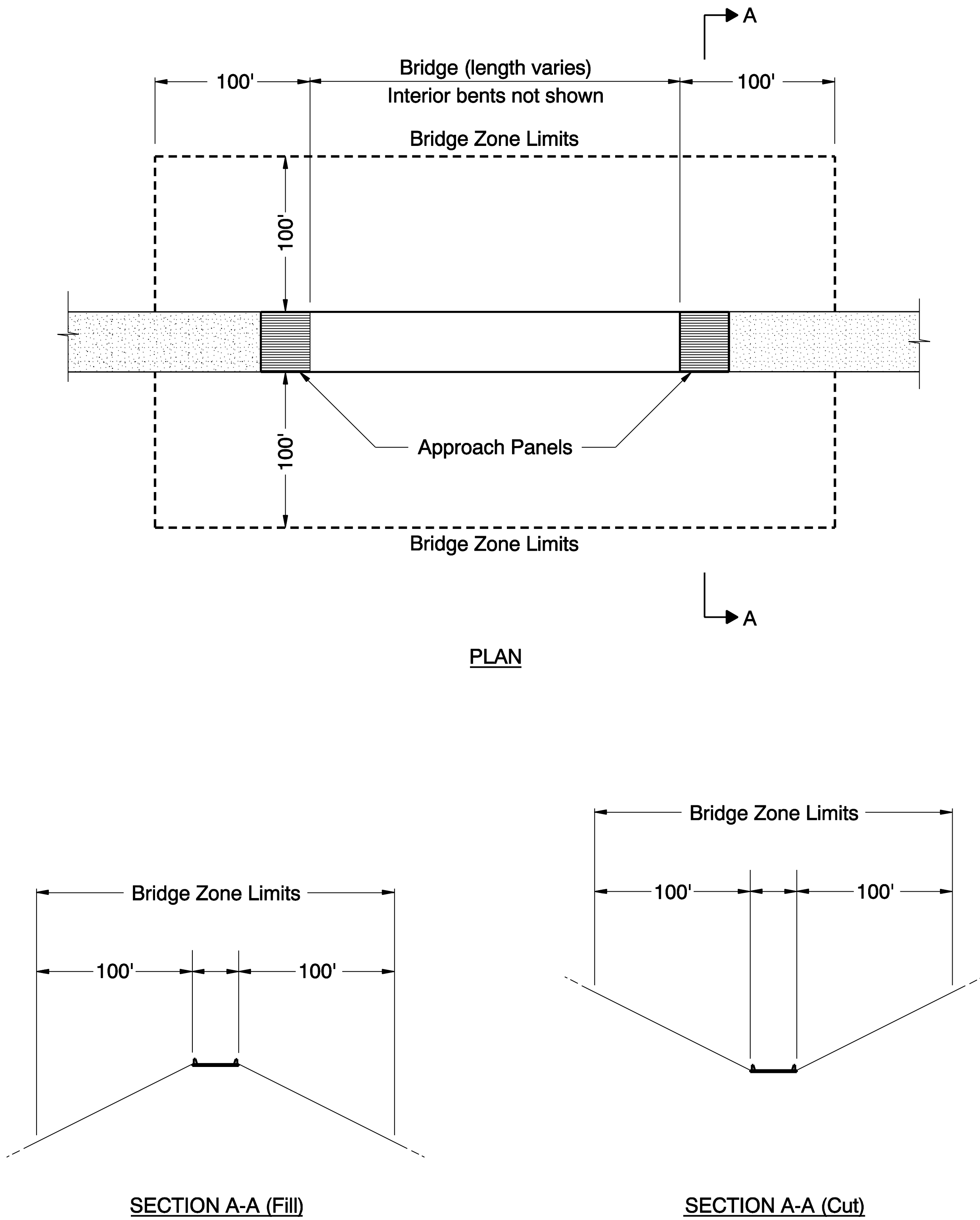
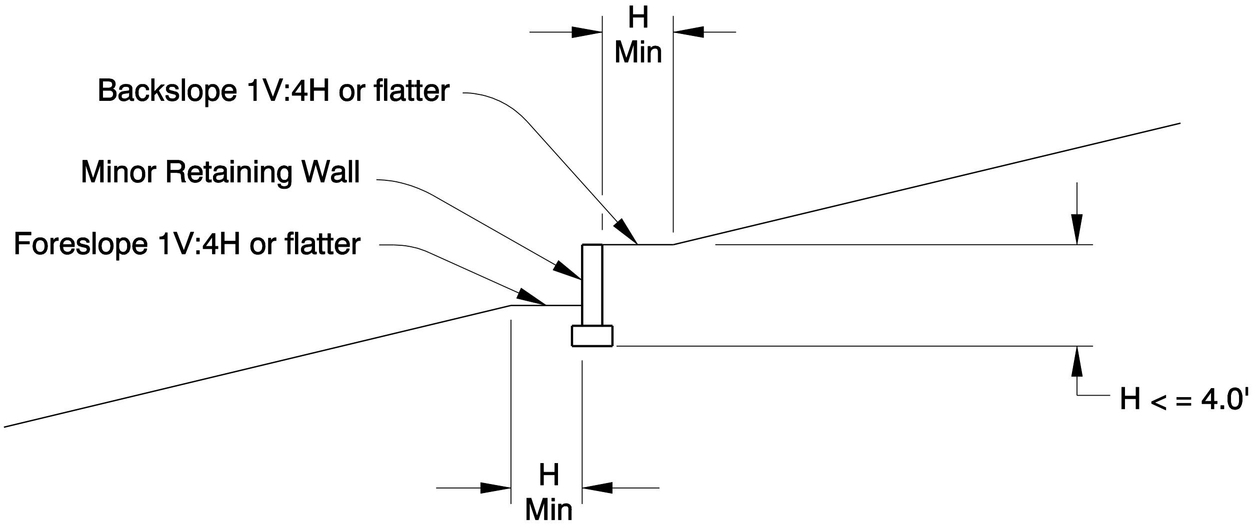
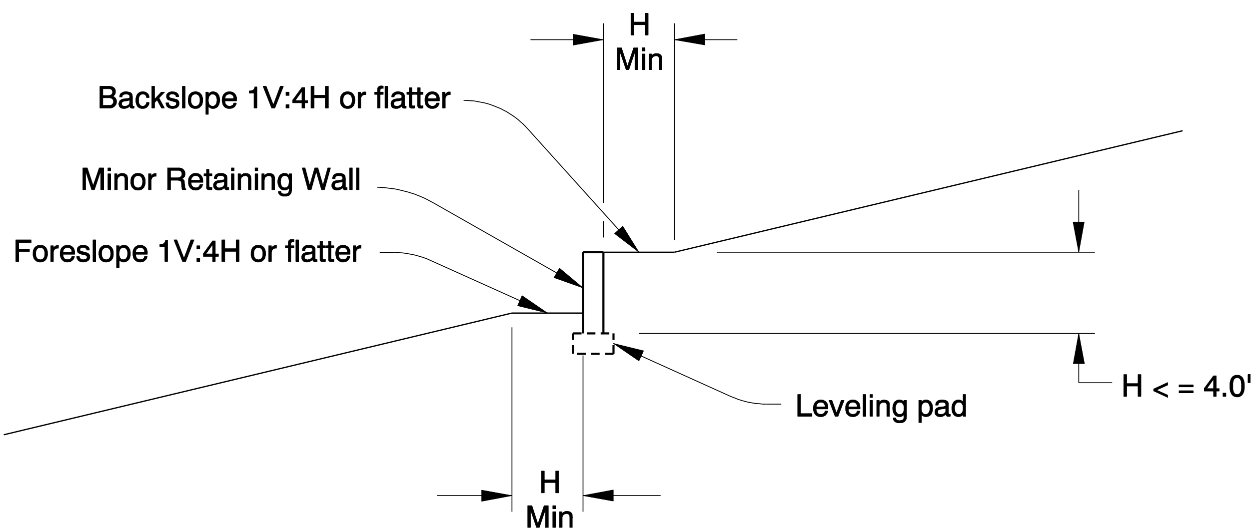


Figure 15-1. Bridge Zone



CAST IN PLACE OR PRECAST CONCRETE MINOR RETAINING WALL



PREFABRICATED MODULAR MINOR RETAINING WALL

Figure 15-2. Minor Retaining Wall

15.2.1.2 Definitions

In order to describe ODOT practices and procedures for retaining wall systems, the following terms are defined, as used in Chapter 15:

Bridge Abutment - See Section 15.2.1.1.

Bridge Retaining Wall System - See Section 15.2.1.1.

Bridge Zone - See Section 15.2.1.1.

Conditions of Preapproval for Proprietary Retaining Wall Systems - Appendix 15-D describes the conditions of preapproval for each proprietary retaining wall system. Other uses are not allowed.

Control Plans - Plans preparation method used for proprietary retaining wall systems. Control plans can be either “Conceptual” or “Semi-detailed” – See Section 15.2.5.2 and Section 15.2.8.1.

Cost Reduction Proposal - Agency procedure that can be used by the Contractor to propose an alternate proprietary retaining wall system. See SS00140.70 in the Oregon Standard Specifications for Construction.

DAP - Design Acceptance Phase. See Section 15.2.3.

Elements and components of Preapproved Proprietary Retaining Wall Systems - See Section 15.2.5.2.

GRS-IBS – Geosynthetic Reinforced Soil Integrated Bridge System. GRS, Geosynthetic Reinforced Soil, is an engineered, well-compacted granular fill with closely spaced layers of geosynthetic reinforcement. GRS-IBS is a method of bridge support that blends the approach roadway into the superstructure using GRS. See Section 15.6.15.4.

Highway Retaining Wall System – See Section 15.2.1.1.

Manufacturer - The proprietary owner of a retaining wall system or proprietary retaining wall component. Used interchangeably in Chapter 15 with *Vendor*.

Minor Retaining Wall System – See Section 15.2.1.1.

Nonproprietary Retaining Wall System - A retaining wall system that is fully designed by the Agency.

Nonproprietary Specification - A specification that does not specify proprietary products either by name or by specifying requirements that only one proprietary product can meet.

Preapproved Proprietary Retaining Wall System - A proprietary retaining wall system that has been granted “preapproved” status by the ODOT Retaining Structures Program, and that may be considered for use on ODOT projects, subject to the “Conditions of Preapproval” for the proprietary system in Appendix 15-D.

Preapproved Proprietary Retaining Wall System - When a fully detailed retaining wall system is not shown on the Agency plans, list acceptable preapproved proprietary retaining wall OPTIONS in project special provision SP0A596 or SP0B596.

Preapproved Proprietary Retaining Wall System Alternates - When a fully detailed retaining wall system is shown on the agency plans, list acceptable preapproved proprietary retaining wall ALTERNATES in project special provision SP00596.

Precast Concrete Large Panel Facing - MSE wall precast concrete facing panel with a face area greater than or equal to 30 square feet.

Precast Concrete Small Panel Facing - MSE wall precast concrete facing panel with a face area of 30 square feet or less.

Proprietary Product - General term including proprietary retaining wall systems and proprietary retaining wall elements and components.

Proprietary Retaining Wall System - A retaining wall system identified in the plans or specifications as a “brand” or trade name, or a retaining wall system so narrowly specified that only a single provider can meet the specification. See Section 15.2.5, Appendix 15-A, Appendix 15-B, Appendix 15-C, and Appendix 15-D.

Public Interest Finding - Agency process that can be used to justify the specification of less than three specific proprietary products. See Section 15.2.6.2.

Retaining Wall Elements and Components - Elements and components used in the design or construction of either a proprietary retaining wall system or a nonproprietary retaining wall system.

Retaining Wall Nonproprietary Elements and Components - Retaining wall elements and components that are not protected by a brand name, trademark, or patent.

Retaining Wall Proprietary Elements and Components - Retaining wall elements and components that are protected by a brand name, trademark, or patent. Also, see Sole Source Specification.

Retaining Wall System - An engineered system of interacting structural and geotechnical retaining wall elements and components designed to restrain a mass of earth, and satisfying all applicable design requirements. The terms *retaining wall system*, *retaining structure*, and *retaining wall* are used interchangeably throughout Chapter 15.

Retaining Wall System Type - See Section 15.2.4.2.

Sole Source Specification - Plans or specifications that require proprietary products either by name or by a requirement that only one proprietary product can meet. Sole source specifications are not allowed in the project plans or specifications, unless a sole source specification is justified by an approved Public Interest Finding. To assure competitive bidding when proprietary products are specified, as many acceptable proprietary products as possible should be listed. See Section 15.2.8.3 for proprietary items (including sole sourcing).

Standard Drawing Retaining Wall System - A non-proprietary retaining wall system for which a standard design is provided in the Oregon Standard Drawings (look in “Bridge 700 Walls”)

Internal and external stability have been designed in accordance with AASHTO Standard Specifications for Highway Bridges, except for bearing capacity, settlement, and overall stability, which are site specific. The wall designer is responsible for applying the standard drawing to a specific site, and for verifying all engineering assumptions stated on the standard drawing.

15.2.2 General Steps in a Retaining Wall Project

1. **Consider whether a retaining wall is the best solution.**

   Consider alternatives such as acquiring additional right of way, flattening the slope, or building a reinforced soil slope.

2. **Determine suitable retaining wall system type.**

   The designer for the retaining wall system determines which wall system type or types are suitable for a given wall location. See Sections 15.2.4.1 and Section 15.3 for general selection criteria, and see Section 15.4 through Section 15.13 for specific wall type selection criteria. Section 15.2.4.2 lists retaining wall system types that may be considered for use on ODOT projects.

3. **Select option.**

   - **Option 1:** Nonproprietary Design

     Under Option 1, the designer completely designs the retaining wall system, and provides fully detailed plans for one type of retaining wall system. See Section 15.2.6 for more information on nonproprietary retaining wall systems.

   - **Option 2:** Proprietary Design

     Under Option 2, the designer provides control plans, rather than a complete retaining wall system design, and the retaining wall system Manufacturer completes the design. See Section 15.2.5 for more information on proprietary retaining wall systems. Before selecting this option, verify that a sufficient number of preapproved proprietary retaining wall systems are available for competitive bidding of the retaining wall system type selected. Alternatively, a request to use a sole source specification may be submitted to the Agency. See Section 15.2.8.3 for competitive bidding of proprietary items, including sole sourcing.

4. **Perform design calculations as required.**

   See *AASHTO LRFD Bridge Design Specifications* and ODOT exceptions and additions to AASHTO in Section 15.3 and Sections 15.4 through Section 15.13. For proprietary retaining wall system design responsibilities, see Appendix 15-A.3.

5. **Prepare contract plans.**

   See Section 15.2.8.1 "Elements of Contract Plans for Retaining Wall Systems."

6. **Prepare contract special provisions.**

   Edit "Boilerplate" special provision SP0A596, SP0B596, and/or SP0C596 as appropriate, for the selected retaining wall system types and selected contract letting. For nonproprietary designs, include estimated quantities for the items listed in SP00596. For proprietary designs, details of the system are not known until after contract letting, so do not include estimated quantities in SP00596. See Section 15.2.8.3 for more information on special provisions.

7. **Prepare estimates.**

   The designer for the retaining wall system is responsible for estimating quantities for retaining wall bid items, and providing them to the project specifications writer. See Section 15.2.8.4 for more information on quantity estimates for retaining wall systems.

The designer for the retaining wall system is also responsible for estimating bid item unit prices. Include cost factors for location, size of wall, inflation, and complexity. Do not include cost factors for mobilization, engineering, and contingencies, all of which will be included by the specifications writer on a project wide basis (See Section 15.2.8.4).

Also, provide an estimate for the time required for construction using a graph format showing all critical stages of the construction, and for the cost of design assistance during construction.

8. **Prepare calculation book.**

   As required (See Section 15.2.8.5).

15.2.3 Retaining Wall Project Schedule

The ODOT Resource Management System (RMS) includes predefined project activities and timelines for project development. The project leader uses MS Project Professional and the appropriate schedule template to create and maintain the project schedule. See the project schedule for activity start and finish dates.

Retaining wall design deliverables:

- DAP Retaining Wall Design (RMS Activity ID 316)
- Preliminary Retaining Wall Plans, Specifications, and Estimate (RMS Activity ID 317)
- Advance Retaining Wall Plans, Specifications, and Estimate (RMS Activity ID 318)

Geotechnical exploration and geotechnical reporting deliverables for retaining walls:

- Geotechnical Exploration (RMS Activity ID 280)
- Preliminary Geotechnical Report (RMS Activity ID 295)
- Final Retaining Wall Plans, Specifications, and Estimate (RMS Activity ID 319)

15.2.4 Selection of Retaining Wall System Type

15.2.4.1 General Criteria for Selection of Retaining Wall System Type

When preparing a list of acceptable wall types for a specific project, the wall designer must consider Sections 15.3 through Section 15.13, as well as the general considerations listed below:

General Considerations include:

1. Project Category
   a. Permanent or temporary wall: A temporary wall must meet the physical requirements with very little concern for aesthetics or long-term design life.
   b. Bridge retaining wall, Highway retaining wall, or Minor retaining wall.
2. Site Conditions Evaluation
   a. Cut or fill: This condition needs to be evaluated because some wall types do not work well for one or the other. Determine if top down construction is required for a cut.

b. Soil profile and site geology: Evaluate the project for variations in wall height and blending the wall into the site. Also, evaluate slope instability and landslide hazards.

c. Foundation conditions and capacity: The foundation soil must be evaluated for capacity to support the wall system.

d. Foundation soil mitigation required/feasible: While certain soil conditions may not support certain wall types, it may be economical to mitigate foundation soil problems to accommodate these wall types.

e. Ground water table location: Consider whether ground water will increase lateral soil pressure on the wall or increase the corrosion potential. Also, evaluate the impact of surface run-off and subsurface drainage conditions.

f. Underground utilities and services: If utilities interfere with soil reinforcement or other wall elements, consider other wall systems.

g. Other structures adjacent to site: Determine if adjacent structures may be affected by wall construction such as pile driving or lack of lateral support.

h. Corrosive environment and effect on structural durability: Evaluate the site for conditions that may cause accelerated corrosion or degradation of the retaining wall system.

3. Geometry and Physical Constraints

a. Height limitations for specific systems: Check the height limits for the wall systems as well as practical design limits.

b. Limit on radius of wall on horizontal alignment: Evaluate wall system to accommodate any radius situation or adjust radius to meet wall system.

c. Allowable lateral and vertical movements, foundation soil settlements, differential movements: Determine allowable movements and choose wall systems that will accommodate the movements.

d. Resistance to scour: If the hydraulics study determines potential scour condition exists, provide sufficient embedment depth or provide scour protection.

e. Wall is located near a bridge: Determine which wall systems are compatible with the bridge.

4. Constructability Considerations. The following items should be considered when evaluating the constructability of each wall system for a specific project:

a. Scheduling considerations (e.g. weather, preloads wait times)

b. Formwork, temporary shoring

c. Right of way boundaries

d. Complicated horizontal and vertical alignment changes

e. Site accessibility (access of material and equipment for excavation and construction)

f. Maintaining existing traffic lanes and freight mobility

g. Vibrations

h. Noise

i. Availability of materials (e.g., MSE backfill)

5. Environmental Considerations
    a. Minimum environmental damage or disturbance: Consider the impact of wall systems on environmentally sensitive areas.
    b. Consider the impact of wall type on the environmental permitting process.
6. Cost
    a. Right of way purchase requirements: Evaluate the cost of additional right of way if it is required to use a given wall system.
    b. Consider the total costs associated with wall construction, rather than the cost of individual wall systems.
7. Aesthetic Considerations
    a. Determine if wall type and/or architectural treatment meets aesthetic requirements at the site.
8. Mandates by Other Agencies
    a. Determine whether wall type complies with mandates by other agencies.
9. Requests made by the Public
    a. Determine if wall type is consistent with public input for the site.
10. Traffic Barrier
    a. Determine whether wall type can accommodate traffic barrier if required at the site.
11. Protective Fencing
    a. Determine whether wall type can accommodate protective fencing if required at the site.

15.2.4.2 Retaining Wall System Types

Retaining wall system types for which adequate design guidance is available are listed in this section. This list will be updated as new guidance becomes available.

Only the wall types listed below, or walls designed in accordance with Section 15.2.7, shall be considered for use on Agency projects:

- Type 1A: CIP Concrete Rigid Gravity Retaining Wall System
- Type 2A: Precast Concrete Crib Prefabricated Modular Retaining Wall System
- Type 2B: Precast Concrete Bin Prefabricated Modular Retaining Wall System
- Type 2C: Metal Bin Prefabricated Modular Retaining Wall System
- Type 2D: Gabion Prefabricated Modular Retaining Wall System
- Type 2E: Dry Cast Concrete Block Prefabricated Modular Retaining Wall System
- Type 2F: Wet Cast Concrete Block Prefabricated Modular Retaining Wall System
- Type 3A: MSE Retaining Wall System with Dry Cast Concrete Block Facing
- Type 3B: MSE Retaining Wall System with Wet Cast Concrete Block Facing
- Type 3C: MSE Retaining Wall System with Precast Concrete Small Panel Facing

- Type 3D: MSE Retaining Wall System with Precast Concrete Large Panel Facing
- Type 3E: MSE Retaining Wall System with Welded Wire Facing
- Type 3F: MSE Retaining Wall System Gabion Facing
- Type 3G: MSE Retaining Wall System with Two-Stage Facing - CIP or Precast Concrete (excluding Type 3H), or Sprayed on Concrete/Mortar Fascia (Constructed after Welded Wire Facing is Installed).
- Type 3H: MSE Retaining Wall System with Precast Concrete “Full Height Panel” Facing
- Type 3J: MSE Retaining Wall System with Geosynthetic Facing
- Type 3K: GRS-IBS Retaining Wall System with Dry Cast Concrete Block Facing
- Type 4A: CIP Concrete Cantilever Semi-Gravity Retaining Wall System
- Type 5A: Soldier Pile Retaining Wall System
- Type 5B: Sheet Pile Retaining Wall System
- Type 5C: Tangent Pile Retaining Wall System
- Type 5D: Secant Pile Retaining Wall System
- Type 5E: Slurry (Diaphragm) Retaining Wall System
- Type 5F : Micropile Retaining Wall System
- Type 6A: Soldier Pile Tieback Retaining Wall System
- Type 6B: Anchored Sheet Pile Retaining Wall System
- Type 7A: Soil Nail Retaining Wall System
- Type 8A: Temporary Geotextile Reinforced Wrapped Face MSE Retaining Wall System

Retaining wall Types 5A, 5B, 5C, 5D, 5E, 5F, 6A, 6B, and 7A listed above may be used as temporary shoring in accordance with the requirements in SP00510 and Section 15.3.26.

Retaining wall Type 8A (Temporary Geotextile Reinforced Wrapped Face MSE) shall be designed in accordance with Sections 15.6.16 and 15.3.27.

Retaining wall Types 2B, 2C, 2D, 2F, 3E, and 3F, used as temporary retaining wall systems, shall be designed in accordance with the criteria in Section 15.3.27.

Design two-stage facing for retaining wall Type 3G (MSE Retaining Wall with Two-Stage Facing) in accordance with the requirements in Section 15.6.11.

15.2.5 Proprietary Retaining Wall Systems

See Appendix 15-A**: General Requirements for Proprietary Retaining Wall Systems**

15.2.5.1 Agency Control Plans for Proprietary Retaining Wall Systems

“Control Plans” are prepared to show requirements for proprietary retaining wall systems. The specific details shown on control plans depend on the retaining wall system types selected.

If multiple dissimilar (proprietary) retaining wall system types are acceptable (e.g., Types 2A-2F and Types 3A-3G in Section 15.2.4.2), the plans should only show details that are generally applicable to all selected retaining wall system types. Plans showing only general details for multiple dissimilar wall system types are considered "Conceptual" control plans.

It is sometimes necessary to use conceptual control plans, but this option is generally not recommended. With this option, the system type is not known until after bid letting, which can lead to difficulties in coordination between design disciplines.

The primary advantage of this plan preparation method is increased competitive bidding because of specifying several proprietary wall types in a set of plans.

If it is determined that only very similar retaining wall system types are acceptable, the plans should show as many details as possible without infringing on proprietary details and without creating a sole source specification. See Section 15.2.8.3 for more information on sole source specifications.

Minimum information required on control plans is listed in Section 15.2.8.1.

15.2.5.2 Elements of Preapproved Proprietary Retaining Wall Systems

Elements and components of preapproved proprietary retaining wall systems are preapproved as part of a specific retaining wall system. Approval of a specific system does not constitute approval of individual elements and components for other use in other systems. Non-system approval of individual elements and components may be a prerequisite to system approval (as in the case of geogrids that must be on the ODOT QPL) but the component must still be specifically approved for use in a specific proprietary system.

15.2.6 Nonproprietary Retaining Wall Systems

Nonproprietary retaining wall systems shall meet the design requirements of Sections 15.3 through Section 15.14. Also, see Section 15.2.4.2.

15.2.6.1 Agency Detailed Plans for Nonproprietary Retaining Wall Systems

Project plans for nonproprietary retaining wall systems shall include all details that are needed to complete the work. Minimum information required on nonproprietary retaining wall systems is listed in Section 15.2.8.1.

15.2.6.2 Components of Nonproprietary Retaining Wall Systems

Nonproprietary retaining wall systems may contain both proprietary and nonproprietary elements and components. Clearly specify all requirements for both proprietary and nonproprietary elements and components of a nonproprietary retaining wall system in the project plans and specifications. Also, see Section 15.2.8.3.

15.2.7 Unique Nonproprietary Wall Designs

Nonproprietary retaining wall systems not listed in Section 15.2.4.2 are considered "Unique" retaining wall system types. These walls are not specifically addressed by AASHTO, FHWA, or Agency design manuals. It is recognized, however that unique retaining wall system types are sometimes needed.

Unique retaining wall system types may be considered for use on ODOT projects if all of the following requirements are met:

- The wall is a fully designed nonproprietary retaining wall system.
- The design is performed in accordance with the following list in order of precedence:
    - This ODOT Geotechnical Design Manual (GDM)
    - AASHTO Standard and Guide Design Specifications
    - U.S. Department of Transportation Federal Highway Administration (FHWA) design manuals
- The designer is required to meet Agency Policy and related technical guidance found at the following link:

  https://www.oregon.gov/ODOT/HWY/TECHSERV/Pages/BPDS/Toolbox-Policies.aspx

15.2.8 Details of Contract Documents

In Design-Bid-Build construction projects, bidding is very competitive, and it should be assumed that the contractor will base the bid strictly on the contract documents. Contract documents should show as much detail as possible (in line with design responsibility) and avoid omission of needed details.

15.2.8.1 Elements of Contract Plans for Retaining Wall Systems

Fully detailed plans used for nonproprietary retaining wall systems Section 15.2.6 should include all information and details needed to bid and build the wall. Control Plans used for proprietary retaining wall systems Section 15.2.5 are either “Conceptual” or “Semi-Detailed”:

- “Conceptual” control plans should include information that is generally applicable to all specified retaining wall system types.
- “Semi-Detailed” control plans should include as many details as possible without infringing on proprietary rights and without creating a sole source specification.

See Section 15.3.23 for proprietary Minor retaining walls. See Section 15.3.24 for nonproprietary Minor retaining walls.

Contract Plans Checklist

The following items should be included (as applicable) on all contract plans regardless of plan preparation method involved, unless noted otherwise:

- Plan
- Elevation
- Typical Section
- General Notes
- Calculation book number (if required in Section 15.2.8.5)
- Structure number (if required in Section 15.2.8.6)
- Vicinity Map
- Retaining wall category (Bridge, Highway, or Minor retaining wall)

- Wall control line
- Right of way and easement limits
- Existing utilities and existing drainage facilities
- A grade line diagram at the wall control line, including curve data, if applicable
- Stations at beginning and end of the retaining wall and at all profile break points along the wall control line
- Elevations at beginning and end of the retaining wall and at all profile break points along the top of the retaining wall at the wall control line
- Elevations along bottom of wall (if not footings), along top of footing (if footings), and along top of leveling pad (if leveling pad)
- Original and final ground elevations in front of and back of retaining wall
- At stream locations, extreme high water, and ordinary high water elevations
- Foundation data or geotechnical data
- Location, depth, and extent of any unsuitable material to be removed and replaced, and any ground improvement details
- Minimum wall embedment
- Minimum/Maximum front face batter
- Minimum reinforcement length for overall and external stability (MSE walls)
- Retaining wall loading diagram
- Magnitude, location and direction of applicable external loads including dead load surcharge, live load surcharge, construction loads (e.g., crane loads, material stockpile loads), barriers (vehicle, bicycle, and/or pedestrian), luminaire and sign supports, bridge end panels, and bridge abutments.
- Seismic design parameters
- Geotechnical design parameters
- Material requirements
- Design standards
- Aesthetic requirements
- Structural details
- Pay Limits for bid items
- Construction sequence requirements, if applicable, including traffic control, access, stage construction sequences, temporary shoring, and ground improvement
- Details of applicable retaining wall appurtenances including utilities and drainage facilities (e.g., storm sewer pipes), copings, barriers or rails (e.g., vehicle, bicycle, and/or pedestrian), guardrail posts, luminaire and sign supports (including conduit locations), fencing, bridge end panels, and bridge abutments. See Section 15.3.23 for proprietary Minor retaining walls. See Section 15.3.24 for nonproprietary Minor retaining walls.

Note:
Because of the interaction between bridges and Bridge retaining walls, bridge plans should show the locations of Bridge retaining walls as well as the wall structure number. Bridge retaining wall plans should show the location of bridge as well as the bridge structure number.

Drafting Related Items

For more information on drafting related items, see:

- Retaining Walls chapter in the Contract Plans Development Guide (most current version)
- http://www.oregon.gov/ODOT/HWY/GEOENVIRONMENTAL/Pages/drafting.aspx web page

15.2.8.2 Standard Specifications

Standard construction specifications for permanent retaining walls located in SS00596 of the 2008 *Oregon Standard Specifications* has been superseded by three special provisions: SP0A596 MSE Retaining Walls, SP0B596 Prefabricated Modular Retaining Walls, and SP0C596 Cast-in-Place Retaining Walls. Additional specifications are planned in the near future for Soil Nail Retaining Walls, Soldier Pile Retaining Walls, and Sheet Pile Retaining Walls.

15.2.8.3 Special Provisions

Include applicable retaining wall special provisions on all projects containing retaining walls. Always download the latest version of the applicable "boiler plate special provisions," edit as required, and include in the contract documents.

"Boiler Plate" special provisions include the latest updates to Standard Specifications as well as project-specific information such as acceptable preapproved proprietary retaining wall systems, geotechnical, and seismic design parameters for proprietary wall design, and estimated quantities.

Provide the estimated wall area for both proprietary and nonproprietary walls in the Special Provisions. For nonproprietary retaining wall systems, include estimated quantities for incidental items (shoring, excavation, reinforced backfill, leveling pads, wall drainage/filter systems, and standard coping). For proprietary retaining wall systems where details of the wall construction are not known until after the construction contract is awarded, do not include estimated quantities for incidental items.

When specifying proprietary retaining wall systems or elements and components, competitive bidding practices are required. Competitive bidding requirements (including sole sourcing) are discussed under the "Proprietary Items" section in the *PS&E Delivery Manual*

Also, include related special provisions such as SP00256, SP00330, SP00350, SP00430, SP00440, SP00510, SP00530, SP00540, and SP2320 as applicable. Download "Boilerplate Special Provisions".

15.2.8.4 Quantity and Cost Estimates

Each project that goes to bid letting includes a schedule of bid items. The schedule of bid items is a list of items that the Contractor must bid on, and includes the standard bid item number, standard description, and quantity for each bid item.

The bid item quantity for retaining wall systems is "Lump Sum," and includes all labor, materials, and incidentals necessary to complete the work as specified. A "pay area" diagram showing the limits of the retaining wall bid item should be provided on the project plans. The "pay area" is typically bounded by the beginning and end of the wall, top of the wall (excluding wall coping), and top of the footing or leveling pad. If no footing or leveling pad exists, the bottom of the wall is used (Figure 15-3). Standard copings are considered incidental to the wall pay item, but sidewalk copings, type "F" traffic barrier copings, moment slabs, and fencing are considered appurtenances and should be included as separate bid items. See Section 15.2.8.3 for more information on "estimated quantities."

The format of the quantity estimate and responsibility for estimating costs and cost factors such as inflation, job location, mobilization, engineering, and contingencies should be determined on a project specific basis by talking with the project specifications writer.

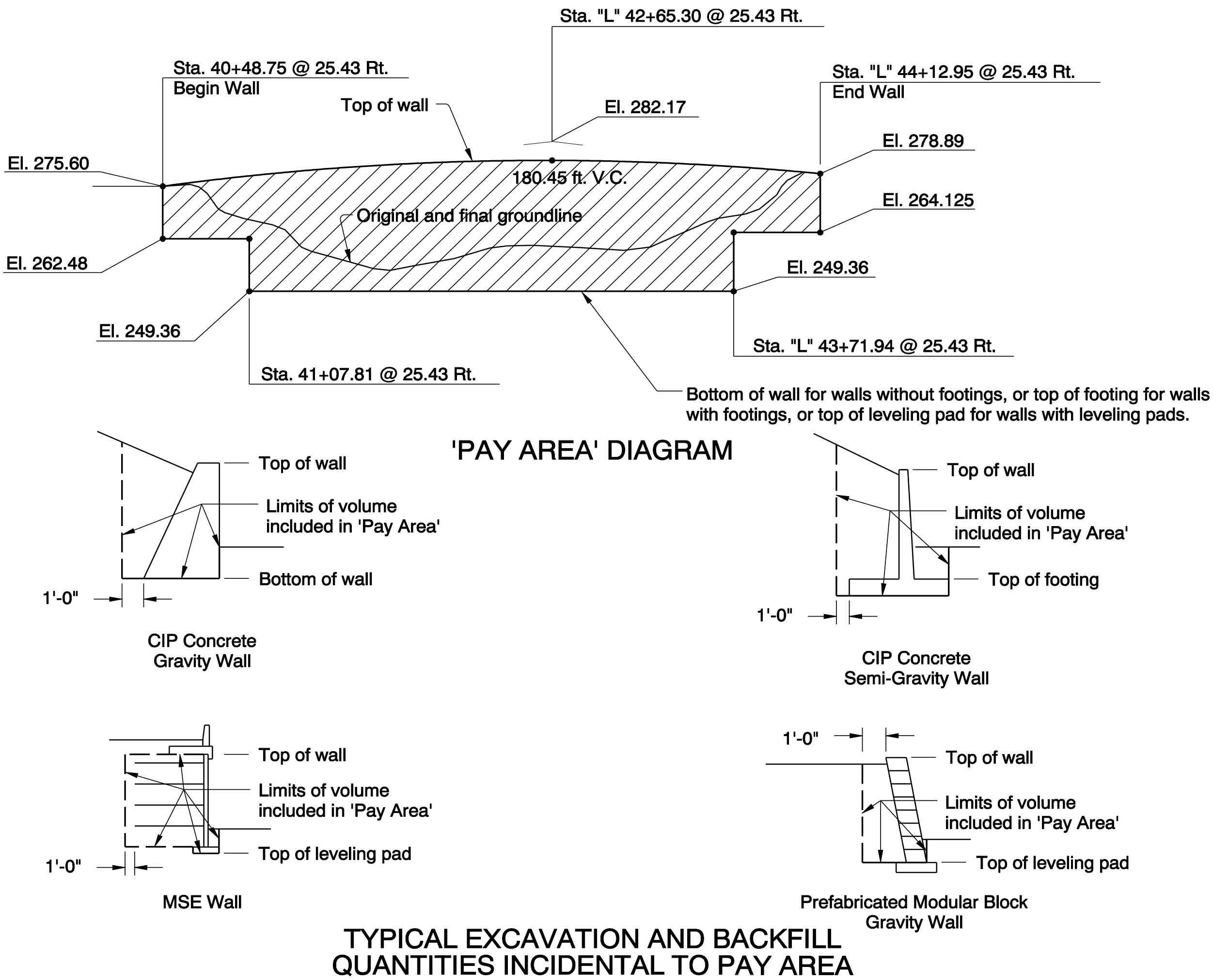


Figure 15-3. Pay Area

15.2.8.5 Calculation Books

This section contains calculation book guidelines for bridge abutments and retaining wall systems.

Retaining walls that require calculation books:

- **Bridge Abutments:** Bridge abutments are considered part of the bridge calculations, and bridge abutment calculations shall be included in the bridge calculation book. Bridge calculation books are covered in the ODOT *Bridge Design and Drafting Manual (BDDM*).
- **Bridge Retaining Walls:** Calculations for each retaining wall structure number shall be located in a separate calculation book or in a separate section of a calculation book. Because of the interaction between the bridge and the associated Bridge retaining wall(s), the bridge calculation book and the Bridge retaining wall calculation books should reference one another.
- **Highway Retaining Walls:** Calculations for each retaining wall structure number shall be located in a separate calculation book or section of a calculation book.
- **Minor Retaining Walls:** *C*alculation books are not required for Minor retaining walls.

Calculation Book Numbers

To obtain retaining wall calculation book numbers, send an email request to:

bridge@odot.state.or.us.

Calculation Book Contents

The following items should be included in calculation books:

- **Title Page:** Title page with structure number, drawing numbers, calculation book number, key number, and construction contract number.
- **Table of Contents**
- **Design Calculations:** Structural and geotechnical calculations performed by (or under the control of) the POR. Show all of your design assumptions, design steps and design methods. Include detailed explanations and sample hand calculations for all computer printouts.
- **Design Check:** Design check of design calculations. The level of detail to be checked varies with the complexity of the project and the experience levels of the Designer and Checker.
- **Design Calculations (by Manufacturer):** Calculations submitted by the Manufacturer for proprietary retaining wall systems, along with Agency review comments.
- **Geotechnical Report:** Include a copy of the Geotechnical Report.
- **Special Provisions:** Include Special Provisions that are applicable to retaining walls.
- **Cost Estimates**

Calculation Book Submittal

Submit the completed calculation book to the Retaining Wall Program for archiving:

Oregon Department of Transportation
Geo-Environmental Section
Engineering and Asset Management Unit
4040 Fairview Industrial Dr. SE, MS #6
Salem, OR 97302-1142
Phone 503-986-3252 Fax 503 986 3249

Calculation Book Responsibilities

Obtain the calculation book number, prepare the calculation book, and submit the completed calculation book.

For proprietary retaining wall systems, include a copy of the stamped design calculations submitted by the Manufacturer, as well the Agency review comments for the design calculations submitted by the Manufacturer of the proprietary retaining wall system.

15.2.8.6 Structure Numbers and Structure Naming Convention

Structure numbers are required for Bridge retaining wall systems and Highway retaining wall systems, but are not required for Minor retaining wall systems. For asset management purposes, the retaining wall structure number shall be unique to the retaining wall and shall not be shared with other structures.

Sometimes adjacent retaining walls must be considered separate walls for asset management purposes. The following sections provide guidance on whether adjacent retaining walls are considered a single structure (with a single structure number), or multiple structures (with multiple structure numbers).

Walls meeting all of the following conditions (as applicable) shall be considered a single structure and shall use a unique structure number:

- The wall must be continuous. Note that continuous walls may contain construction joints, expansion/contraction joints, slip joints, angle points, and steps.
- The wall must consist of a single retaining wall system type.
- For proprietary retaining wall systems, the wall must consists of a single proprietary retaining wall system.
- The wall must be constructed at the same time as part of one project.

Walls meeting any of the following conditions (as applicable) shall be considered separate structures, each with a unique structure number:

- Walls separated by gaps (except as noted above).
- Walls constructed at different times.
- Walls that are not part of the same retaining wall system type.
- Proprietary retaining walls that are not part of the same proprietary retaining wall system.

The drafter typically obtains structure numbers along with drawing numbers using the *ODOT Bridge Data System (BDS)*.

Provide the drafter with BDS input as needed. Also see Section 15.2.8.1.

Follow the wall naming convention contained in the current structure-naming document on the Bridge Engineering website.

15.3 Design Requirements: General Wall Design

15.3.1 Design Methods

Retaining structures shall be designed using the Load and Resistance Factor Design (LRFD) method whenever possible. Retaining structures shall be designed in accordance with the following documents:

- *ODOT Geotechnical Design Manual (GDM)*; and
- *AASHTO LRFD Bridge Design Specifications*

The most current versions or editions of the above referenced documents shall be used, including all interim revisions and technical bulletins modifying these documents. In case of conflict or discrepancy, the ODOT GDM design requirements shall supersede those in AASHTO LRFD. The references listed in this chapter provide additional design and construction guidance for retaining walls—but should be considered supplementary to the ODOT GDM and AASHTO LRFD documents listed above.

Most FHWA manuals listed as ODOT design references were not developed for LRFD design. Wall types for which LRFD procedures are not currently available shall be designed using Allowable Stress Design (ASD) or Load Factor Design (LFD) procedures as indicated (in full or by reference) in

This chapter. The following subsections describe ODOT exceptions and additions to the referenced standards for general retaining wall design, and include discussions of special design topics applicable to general retaining wall design.

15.3.2 Wall Facing Considerations

The wall facing must meet all project requirements, including appearance (aesthetics), face angle or batter, horizontal alignment, internal and external stability requirements, environmental conditions (e.g. UV exposure, corrosion, freeze-thaw, and runoff effects), and compatibility with the retaining wall system.

Typical MSE retaining wall facing options include the following:

- Dry cast concrete block (MSE and gravity wall systems)
- Wet cast concrete block (MSE and gravity wall systems)
- Precast concrete panel (small and large facing units)
- Welded wire
- Sprayed on concrete/mortar facing on welded wire facing
- Gabion (tied wire baskets filled with rock)
- Cast-in-place concrete
- Geotextile sheet (wrapped-face construction)

15.3.3 Wall Face Angle (Batter)

Wall face batter should take into consideration several factors, including constructability, maintenance, appearance, and the potential for negative batter. Negative batter typically results from poor construction practice, heavy construction loads near the wall face, and/or excessive post-construction differential foundation settlements. Typical design wall face batters for conventional retaining walls are as follows:

15.3.3.1 CIP Gravity and Cantilever Walls

The finish face batter is typically designed no steeper than approximately 5° (12v:1h). Steeper face batters have been used, however, for walls up to approximately 20 ft. in height and transitional wall sections that match existing vertical walls.

15.3.3.2 Mechanically Stabilized Earth (MSE) Walls

The finish face batter of precast concrete panel MSE walls is typically designed to be as steep as 0° (vertical). This may require a positive batter allowance during construction to prevent a negative wall face batter due to normal wall construction deformation, post-construction foundation settlement, and/or heavy surcharge loads.

The finish face batter of MSE retaining walls with dry cast concrete block facing units is typically designed to be no steeper than approximately 1° (57v:1h).

The finish face batter of MSE retaining walls with wet cast concrete block facing units is typically designed to be as steep as 3º (19v:1h) to 6° (10v:1h).

Temporary wrapped-face type geotextile MSE walls, where a small negative batter would not impair wall stability or function, are typically designed at a finish batter as steep as 0° (vertical).

15.3.3.3 Prefabricated Modular Walls

Prefabricated modular (gravity) retaining walls, which include crib, bin, gabion, dry cast concrete block, and wet cast concrete block walls, are typically battered between approximately 3° (19v:1h) and 10° (6v:1h).

15.3.4 Horizontal Wall Alignment

Retaining wall selection should consider project-specific horizontal alignment requirements. Smaller facing units, such as dry cast concrete blocks, typically can be constructed to meet a more stringent (smaller) radius of curvature requirement. Conversely, larger facing units, such as wet cast concrete blocks, typically require a larger radius of curvature. Typical horizontal alignment criteria, including minimum radius of curvature, for conventional retaining walls are as follows:

15.3.4.1 CIP Gravity and Cantilever Walls

Gravity and cantilever retaining walls can be formed to a very tight radius of curvature to meet almost any project-specific horizontal (or vertical) wall alignment requirement.

15.3.4.2 Mechanically Stabilized Earth (MSE) Walls

The horizontal alignment requirement of MSE walls depends on several factors:

- Facing element dimensions (length, height and thickness)

- Facing panel layout of larger block facing units
- The selection/availability of special facing shapes to meet wall alignment requirements.

MSE retaining walls with small precast concrete panel facing (5-ft-wide units) are typically designed with a radius of curvature of 50 ft., or greater. This assumes a joint width of at least ¾ in.

MSE retaining walls with dry cast concrete block facing can be formed to a tight radius and are typically designed assuming a radius of curvature of 10 ft., or greater.

15.3.4.3 Prefabricated Modular Walls

- Crib, bin, and gabion retaining walls are not well suited for alignments requiring a tight radius of curvature. *AASHTO Article 11.11.1* (*AASHTO LRFD Bridge Design Specifications*) recommends design using a radius of curvature of at least 800 ft.—unless the horizontal curve can be substituted by a series of chords.
- Dry cast concrete block gravity retaining walls (single block thickness) can be formed to a tight radius and are typically designed assuming a radius of curvature of 10 ft., or greater.
- Wet cast concrete block gravity retaining walls arranged in a single row configuration are typically designed using a radius of curvature of 75–100 ft.
- Wet cast concrete block gravity retaining walls, more than one block in thickness, should be designed with a radius of curvature at least 800 ft.

15.3.5 Tiered or Superimposed Walls

A tiered or superimposed retaining wall consists of a lower tier retaining wall that supports the surcharge or load from an upper wall.

Tiered or superimposed retaining wall stability analysis and design shall consider the effects of the loads from the upper tier wall (including seismic loads) on the lower retaining wall. The internal, external, compound, and overall stability of the lower tier wall, including foundation settlement and wall deformation, shall be evaluated for these additional loads.

Analysis of the combined tiered wall system shall include investigating internal, compound, and overall failure surfaces through walls; foundation soils; backfill materials; embankments; and the ground surface between, above, and/or below the tiered retaining walls. Perform overall stability analysis using a state-of-the-practice slope stability computer program, such as the most current versions of Slope/W® (Geo-Slope International), Slide® (Rocscience, Inc.), and ReSSA® (ADAMA Engineering, Inc.). Overall stability analysis of tiered wall systems shall be in accordance with the requirements of AASHTO Article 11.6.2.3.

Design guidance for tiered MSE walls is provided in Section 15.6.13.

15.3.6 Back-to-Back Walls

Design guidance for back-to-back MSE walls is provided in Section 15.6.14. See the sections on specific wall types for further guidance on designing for back-to-back walls.

15.3.7 Wall Bench

AASHTO Article 11.10.2.2 requires a horizontal bench with a minimum width of 4.0 ft. in front of MSE walls founded on slopes. Where practical, a 4.0-ft-wide bench should be provided at the base of all retaining walls to provide access for inspection, maintenance, and/or repair. The bench shall be 1v:6h, or flatter, and sloped to direct surface water to properly designed water collection facilities.

15.3.8 Wall Back Slope

Retaining wall back slopes shall be designed at 1v:2h (or flatter) unless a steeper back slope can be justified based on a project-specific geotechnical investigation and design.

15.3.9 Wall Stability

Design retaining walls for internal stability, external stability (sliding, bearing resistance, and settlement), overall (global) stability, and compound stability in accordance with the *AASHTO LRFD Bridge Design Specifications* and the *ODOT GDM*.

Overall and compound wall stability shall be evaluated using conventional limit equilibrium methods, and analyses shall be performed using a state-of-the-practice slope stability computer program such as the most current versions of Slope/W® (Geo-Slope International), Slide® (Rocscience, Inc.), and ReSSA® (ADAMA Engineering, Inc.).

Compound failure plane passing through the reinforced mass is not generally critical for simple, non-tiered MSE walls with rectangular geometry, with uniform reinforcement spacing and length, and without significant surcharge. Compound failures must be considered for complex situations such as changes in reinforced soil types or reinforcement lengths, high surcharge loads such as from sloping backfill or spread footing abutments, sloping faced structures, a slope at the toe of the wall, or tiered walls. (See AASHTO LRFD Article 11.10.1, its commentary, and AASHTO Figure 11.10.2-1)

Overall stability analysis shall investigate all potential failure surfaces passing behind and under the wall. Compound stability analysis shall investigate all potential failure surfaces that pass partially behind, under, or through the wall.

The overall stability of temporary cut slopes to facilitate retaining wall construction shall be evaluated in accordance with the requirements of Section 15.3.26.

Overall and compound wall stability shall be evaluated at the Service I and Extreme I limit states as follows:

- Service I Limit State: A resistance factor of $\varphi = 0.65$ shall be used for overall and compound stability when designing retaining walls. AASHTO Article 11.6.2.3 requires retaining wall be designed using a resistance factor for global (overall) stability ranging from $\varphi = 0.75$ (where geotechnical parameters are well defined and the wall does not support a structural element) to $\varphi = 0.65$ (where geotechnical parameters are based on limited information, or the wall supports a structural element).
- Extreme I Limit State: A resistance factor of $\varphi = 0.90$ shall be used for overall and compound stability when designing retaining walls.

15.3.10 Lateral Earth Pressures

Active, at-rest, and passive lateral earth pressures for retaining wall design shall be calculated based on project-specific geotechnical data such as the subsurface profile, water head/groundwater levels, geotechnical soil properties (based on project-specific lab data), backslope/foreslope profiles, and soil-wall movement considerations as discussed below. Calculate lateral earth pressures on walls in accordance with AASHTO Article 3.11.

If live loads, including traffic, compaction, or construction equipment, can occur within a horizontal distance behind the top of a wall equal to one-half of the wall height, the design lateral load should be increased to account for the additional lateral earth pressure that will act on the wall

The lateral active earth pressure thrust on retaining walls, which stabilize landslides (used to calculate external stability), shall be estimated from conventional limit equilibrium analysis using a state-of-the-practice slope stability computer program such as the most current versions of Slope/W® (Geo-Slope International) and Slide® (Rocscience, Inc.).

15.3.10.1 Active Earth Pressure

Calculate active earth pressures on walls based on Coulomb or Rankine theories in accordance with AASHTO Article 3.11.5.3. Active earth pressures acting behind a retaining wall will depend on the ability of the wall to rotate and/or translate laterally (see AASHTO Article 3.11.1). An active earth pressure coefficient is appropriate when the top of the retaining wall can displace laterally at least 0.001*H (dense sand backfill) to 0.004*H (loose sand backfill) in accordance with AASHTO Table C3.11.1-1 where H is the height of the wall. Active lateral earth pressures on retaining walls shall be increased to include the effects of a sloping backfill in accordance with AASHTO Article 3.11.5.3.

The lateral active earth pressure thrust on retaining walls with a broken backslope, point load(s) or surcharge(s), groundwater effects, and/or with a non-uniform soil (backfill) profile, may be calculated using conventional limit equilibrium analysis using a state-of-the-practice slope stability computer program such as the most current versions of Slope/W® (Geo-Slope International) and Slide® (Rocscience, Inc.), or the Culmann or Trial Wedge methods such as presented in Soil Mechanics in Engineering Practice (Terzaghi and Peck, 1967) and NAVFAC DM-7.01 and DM-7.02 (U.S. Navy, 1986).

15.3.10.2 At-Rest Earth Pressure

The at-rest earth pressure coefficient shall be used to calculate the lateral earth pressure for non-yielding retaining walls restrained from rotation and/or lateral translation in accordance with AASHTO Article C3.11.1. Non-yielding walls include, for example, integral abutment walls, wall corners, cut-and-cover tunnel walls, and braced walls or walls that are cross-braced to another wall or structure. Where bridge wing walls join the bridge abutment, at-rest earth pressures should also be used.

15.3.10.3 Passive Earth Pressure

Calculate passive earth pressures on walls based on Log Spiral and Trial Wedge theories in accordance with AASHTO Article 3.11.5.4. Calculate the lateral passive earth pressure thrust against walls adjacent to a broken back foreslope, point load(s) or surcharge(s), and/or with a non-uniform soil profile using the *Culmann or Trial Wedge* methods such as presented in Soil Mechanics in Engineering Practice (Terzaghi and Peck, 1967) or NAVFAC DM-7.01 and DM-7.02 (U.S. Navy, 1986).

When the Trial Wedge method is used to calculate the passive earth pressure thrust, the wall interface friction angle shall not be greater than 50 percent of the peak soil friction angle in accordance with AASHTO Article 3.11.5.4.

Neglect any contribution from passive earth pressure in stability calculations unless the base of the wall extends below the depth to which foundation soil or rock could be weakened or removed by freeze-thaw, shrink-swell, scour, erosion, construction excavation, or any other means. In wall stability calculations, only the embedment below this depth, known as the effective embedment depth, shall be considered when calculating the passive earth pressure resistance. This is in accordance with AASHTO Article 11.6.3.5.

Lateral wall footing displacements of approximately 0.01*H (dense sand) to 0.04*H (loose sand) and 0.02*H (low plasticity silt) to 0.05*H (high plasticity clay) are required to mobilize the maximum passive earth pressure resistance, where H is the effective embedment depth below foundation soils that could be weakened or removed as defined above. This is in accordance with AASHTO Article C3.11.1. Passive earth pressure resistance assumed in wall stability analysis shall be reduced or neglected, unless the wall footing has been designed to translate the minimum distances provided in AASHTO Table C3.11.1-1.

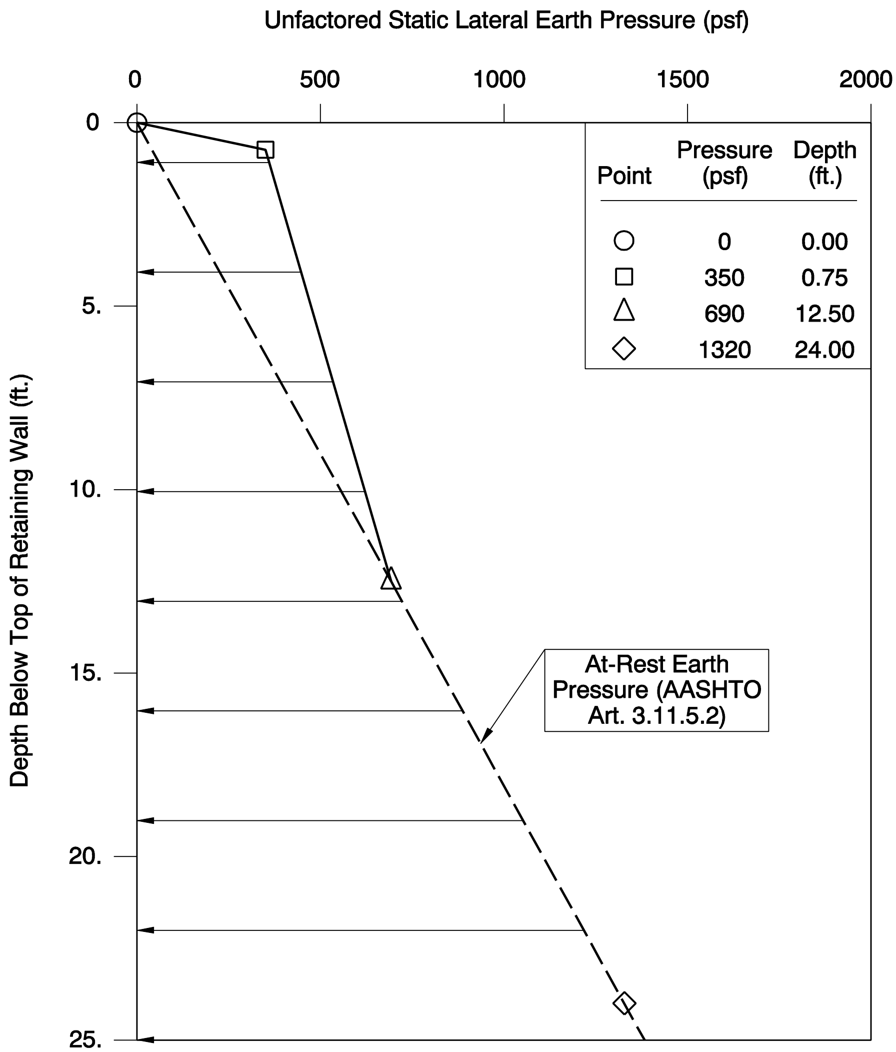
15.3.11 Compaction Loads

Compaction equipment operated behind non-deflecting (restrained) semi-gravity cantilever and rigid gravity retaining walls can cause lateral earth pressures acting on the wall to exceed at-rest lateral earth pressures. The closer the compaction equipment operates to the wall, and the larger the total (static plus dynamic) compaction force, the higher will be the compaction induced lateral earth pressures on the wall.

Figure 15-4 shows a lateral earth pressure diagram that includes the combined effects of residual lateral earth pressures from compaction and at-rest lateral earth pressures on non-deflecting semi-gravity (cantilever) and rigid gravity retaining walls.

Residual lateral earth pressure from compaction need not be considered in external stability design if walls can deflect sufficiently to develop active earth pressures in accordance with Section 15.3.10.1 - but should be considered for internal stability (structural) design since residual lateral earth pressures can cause overstress in structural elements before sufficient deflection associated with the active state occurs.

Consider the lateral earth pressures from a compacted backfill to be “EH” loads, and use the corresponding load factors.



Notes:
1. Compaction-induced lateral earth pressures estimated using Peck and Mesri (1987) method. This method was developed for relatively stiff CIP Semi-Gravity (Cantilever) and Rigid Gravity retaining walls.

2. Recommended unfactored static lateral earth pressures assume backfill peak soil friction angle of 34°, compacted backfill unit wt. of 125 pcf, and backfill compaction with hand-operated, vibratory roller (combined operational weight plus dynamic or centrifugal force not greater than 5,000 lbs). Additionally, it was assumed this hand-operated compaction equipment is operated within a distance of 0.2 ft (2 in) from the back of the retaining wall.

3. Recommended unfactored static lateral earth pressures shall be considered "EH" loads to be used with the applicable load factors.

Figure 15-4. Unfactored Static Lateral Earth Pressure with Residual Horizontal Compaction Pressures on Non-deflecting CIP Semi-Gravity and Rigid Gravity Retaining Walls.

15.3.12 Construction Surcharge Loads

Design retaining walls for increased lateral earth pressures due to typical construction surcharge loads in accordance with AASHTO Article 3.11.6 and the *ODOT GDM*.

Retaining walls shall be designed for construction surcharge loads, including construction equipment operation and storage loads behind the wall if the ground surface behind the wall is sloped at 1v:4h or flatter. Apply a uniform live load surcharge of at least 250 pounds per square foot (psf) along the ground surface behind the wall to represent typical construction loads. Additionally, design walls for lateral earth pressures resulting from any anticipated special construction loading condition, such as the operation of a large or heavily loaded crane, materials storage, or soil stockpile near the top of the wall.

Design shall assume that seismic loads do not act concurrently with construction surcharge loads.

15.3.13 Seismic Design

Seismic design of retaining walls shall be in accordance with the requirements in Section 11 (Walls, Abutments, and Piers) of the 2014 AASHTO LRFD Bridge Design Specifications, 7th Edition (AASHTO, 2014. See Chapter 6 for the seismic design performance objectives for Bridge retaining walls and Highway retaining walls.

Unless stated otherwise, seismic design of retaining walls shall assume a vertical acceleration coefficient (k_v) = 0.0.

For Extreme Event I limit state, the load factor for live load (γ_{EQ}) shall be equal to 0.5.

Where retaining walls cannot be fully drained, lateral pressure force effects due to water pressure head shall be added to seismic lateral earth pressures calculated in accordance with AASHTO LRFD and the ODOT GDM.

When the M-O method is not applicable and external seismic lateral loads are calculated using the GLE method, provide the seismic coefficient (k_h) and the external seismic lateral thrust (P_{AE}) in the project special provisions.

See the sections on specific wall types for further guidance on designing for seismic effects.

15.3.14 Minimum Footing Embedment

Unless otherwise indicated, retaining wall footing embedment shall be no less than 2.0 ft. below lowest adjacent grade in front of the wall.

The final footing embedment depth shall be based on the required geotechnical bearing resistance, wall settlement limitations, and all internal, external, and overall (global) wall stability requirements in *AASHTO LRFD* and the *ODOT GDM*. Additionally, footing embedment shall meet requirements in the ODOT BDDM.

The minimum wall footing embedment depth shall be established below the maximum depth foundation soils (or rock) could be weakened or removed by freeze-thaw, shrink-swell, scour, erosion, construction-excavation, or any other means. The potential scour elevation shall be established in accordance with *AASHTO LRFD*, the *ODOT BDDM*, and the *ODOT Hydraulics Manual*.

15.3.15 Foundation Settlement Serviceability Criteria

Retaining wall structures shall be designed for the effects of the total and differential foundation settlements at the Service I limit state, in accordance with *AASHTO LRFD* and the *ODOT GDM*. Maximum foundation settlements shall be calculated along longitudinal and transverse lines through retaining walls. In addition to the requirements for serviceability in AASHTO LRFD, Tables 15-1 and

15-2 shall be used to establish acceptable settlement criteria (includes settlement that occurs during and after wall construction). However, settlement criteria more stringent than that indicated in Tables 15-1 and 15-2 may be applicable based on project specific requirements for retaining walls, including aesthetics.

Maximum tolerable retaining wall total and differential foundation settlements are controlled largely by the potential for cosmetic and/or structural damage to facing elements, copings, barrier, guardrail, signs, pavements, utilities, structure foundations, and other highway construction supported on or near the retaining wall.

Table 15-1. Foundation Total Settlement Criteria

Wall Type	Maximum Total Settlement, Inch.	
	Criteria A[1]	Criteria B[2]
MSE walls with cast-in-place facing or large precast concrete panel facing (panel front face area ≥30 ft²)	1	2
Crib walls (precast concrete)	1	2
CIP concrete gravity and semi-gravity cantilever walls	1	2
Non-gravity cantilever walls and anchored walls	1	2
Bin or gabion walls	2	4
MSE walls with small precast concrete panel facing (panel front face area <30 ft²)	2	4
MSE walls with dry cast concrete block facing units	2	4
MSE Walls with geotextile/welded-wire/gabion basket facing	4	12
MSE walls with structural facing installed during a second construction stage after MSE wall settlement is complete (MSE retaining wall system with two-stage facing)	4	12

[1] Criteria A – Maximum settlement within accepted tolerance – proceed with structure design and construction.

[2] Criteria B – Maximum settlement exceeds accepted tolerance – ensure structure can tolerate settlement.

Table 15-2 provides maximum foundation differential settlements for selected retaining wall types:

Table 15-2. Foundation Differential Settlement Criteria

Wall Type	Maximum Differential Settlement Over 100 Feet, Inch	
	Criteria A	**Criteria B**
CIP concrete gravity and semi-gravity cantilever walls	¾	2
MSE walls with cast-in-place facing or full-height precast facing panels	¾	2
Crib walls (precast concrete)	¾	2
Wet and dry cast concrete block gravity retaining walls	1½	3
Bin (precast concrete or metal)	1½	3
MSE walls with large precast concrete panel facing (panel front face area ≥30 ft^2)	1½	3
MSE walls with small precast concrete panel facing (panel front face area <30ft^2)	1½	3
MSE walls with dry cast concrete block facing units	1½	3
MSE Walls with geotextile/welded-wire/gabion basket facing	3	9
Gabion	3	9

Select a retaining wall type that meets both the total and differential foundation settlement tolerance criteria provided above. If the selected wall type does not meet the settlement tolerance criteria, then select a more settlement-tolerant wall type. For example, an MSE wall with dry cast concrete block facing is more tolerant of foundation settlement than an MSE wall with large precast concrete facing.

When project requirements dictate the use of a specific retaining wall type, irrespective of foundation settlement tolerance considerations, then the following options should be considered for accommodating or reducing excessive foundation settlements:

- Use of a MSE wall system with two-stage facing designed in accordance with Section 15.6.11. A relatively flexible geotextile or welded-wire face MSE wall (first-stage wall) is built to near final grade and a surcharge used as needed to reduce long-term foundation settlements. MSE wall stability and settlement is carefully evaluated for all stages of construction in accordance with the ODOT GDM. After monitoring indicates the time-rate of foundation settlement has been adequately reduced, settlement-sensitive, cast-in-place or precast wall facing elements, coping and appurtenances are installed for the completed (second-stage) MSE wall.
- Partial to complete removal of the compressible soil layer(s) and replacement with granular structure backfill meeting the requirements of 00510.
- Ground improvement techniques to reduce foundation settlements. Chapter 11 in the *ODOT GDM* provides guidance for selection of an appropriate ground improvement method and preliminary ground improvement design criteria.
- Use of lightweight retaining wall backfill to reduce the wall surcharge.
- Deep foundation support of the retaining wall.
- Where longitudinal differential settlement in excess of 3 inches is anticipated, consider use of full-height, slip joints along MSE walls with precast concrete panel facing.

15.3.16 Groundwater Monitoring

Install at least one piezometer at each retaining wall site to monitor fluctuations in groundwater elevations. This data is required for the following reasons:

- Seismic hazard assessment and mitigation design—liquefaction/lateral spread;
- Foundation design—bearing resistance and settlement;
- Design lateral earth pressure(s);
- Internal, external, compound and global (overall) stability analysis;
- Seepage analysis for design of retaining wall subdrainage system;
- Evaluate construction dewatering requirements; and
- Analysis and design of temporary excavation (back-cut) and long-term slope stability.

15.3.17 Seismic Hazards

The most common causes of poor seismic performance of properly constructed retaining walls are foundation failures and severe strength loss in the wall backfill. The geotechnical designer shall first focus on evaluating the strength loss potential of earth materials comprising and surrounding the retaining structure and its foundation, including assessment of liquefaction, lateral spread, and other seismic hazards at the wall site in accordance with *AASHTO LRFD* and *ODOT GDM* Chapter 6, Chapter 8, and Chapter 15. Analysis and design for assessment and mitigation of seismic hazards shall be in accordance with *AASHTO LRFD* and the *ODOT GDM*.

15.3.18 Wall Subsurface Drainage

Retaining walls shall include an adequate wall subsurface drainage system designed to resist the critical combination of water pressures, seepage forces, and backfill lateral earth pressure(s) in accordance with AASHTO LRFD and the ODOT GDM.

Inadequate wall subdrainage can cause premature deterioration, reduced stability, and failure of a retaining wall. A properly designed wall subdrainage system is required to control potentially damaging hydrostatic pressures and seepage forces behind and around a wall. Redundancy in the subdrainage system is required where subsurface drainage is critical for maintaining retaining wall stability. Properly designed and constructed wall subdrainage systems provide the following benefits:

- Improve appearance and reduce deterioration rates of retaining wall components subject to wetness;
- Protect MSE wall steel and geosynthetic reinforcements from exposure to aggressive subsurface and surface water;
- Increase density and strength of wall backfill materials;
- Increase wall backfill resistance to liquefaction and loss of strength under seismic loads;
- Increase wall foreslope, backslope, and global stability; and
- Increase density and strength of wall foundation soils.

The sizing of subdrainage system components (i.e., permeable layers, collector/outlet pipes, and drainage ditches) shall be based on project-specific calculated seepage volumes. Design the selected subdrainage system using methods such as those presented in *Soil Mechanics NAVFAC DM-7.01* (U.S. Navy, 1986), *Soil Mechanics in Engineering Practice* (Terzaghi and Peck, 1967), or *Seepage, Drainage, and Flow Nets, 3rd Edition* (H. R. Cedergren, 1989).

Provide retaining wall drainage for conventional cast-in-place concrete (CIP), semi-gravity (cantilever) and gravity retaining walls in accordance with AASHTO Article 11.6.6. Drainage for CIP cantilever and gravity retaining walls typically consists of a positive-flow, perforated collector drainpipe installed in a permeable layer along the wall heel. The collector pipe is typically connected to a solid outlet pipe at a sag (or the low end) of the collector pipe. The solid pipe discharges water to an approved, maintained drainage ditch or storm drain system. Provide clean outs at the high end of the collector pipe, or at other suitable locations. A drainage geotextile shall encapsulate the collector pipe and surrounding permeable layer to prevent the migration of surrounding soils into the subdrainage system that could result in clogging of the collector pipe and/or permeable layer(s) and reduced wall subdrainage capacity.

Drainage for soldier pile/lagging, sheet pile, soil nail, and other non-gravity cantilever and anchored retaining wall systems shall meet all the requirements in AASHTO Article 11.8.8 and Chapter 15.

Drainage for permanent soldier pile/lagging or soil nail walls typically includes vertical strip drains (prefabricated composite drainage material) to transport drainage to weep holes and/or drainage collector pipes located near the base of the wall. The collector pipe is connected to a solid outlet pipe that should discharge into an approved drainage ditch or storm drain system. Provide properly located clean outs for the collector and outlet pipes.

Perforated collector and solid pipes shall be Schedule 40 PVC pipe meeting all applicable ASTM requirements, including D1784 and D1785. The PVC pipe shall be at least 6in diameter to allow for periodic pipe flushing and cleaning, irrespective of discharge capacity requirements. Pipe discharge and clean out locations shall be readily accessible to maintenance personnel. Provide metal screens or secure caps at pipe ends to prevent rodent entry.

Porewater pressures from static groundwater levels shall be added to effective horizontal earth pressures to determine total lateral pressures on retaining walls in accordance with AASHTO Article C3.11.3. The effects of water pressures on retaining walls such as the potential for piping instability, a “quick condition”, and/or loss of soil strength from seepage forces can be approximated using procedures in *Soil Mechanics NAVFAC DM-7.01* (U.S. Navy, 1986), *Soil Mechanics in Engineering Practice* (Terzaghi and Peck, 1967), *Soil Mechanics* (Lambe and Whitman, 1969), and/or *Seepage, Drainage, and Flow Nets, 3rd Edition* (H. R. Cedergren, 1989).

15.3.19 Underground Utilities

Mechanically stabilized earth (MSE) walls, soil nail or any type of anchored retaining wall should be avoided when existing or future (planned) underground utilities are located within or below the reinforced backfill or anchorage zone behind walls. Utilities encapsulated within the reinforced or anchored zone will not be accessible for replacement or maintenance. Removal (cutting) of ground support elements for new utility construction could result in wall failure. Soil nail and anchor installation could damage in-place utilities.

15.3.20 Design Life

The minimum design life for Highway Retaining Walls shall be 75 years. The design life of Bridge Retaining Walls shall be consistent with the structures they stabilize, but not less than 75 years.

15.3.21 Corrosion Protection

Corrosion protection consistent with the intended design life of the retaining wall is required for all walls based on the criteria in AASHTO Articles 11.10.6.4.2a or 11.10.6.4.2b. The level of effort to prevent corrosion of metallic components in retaining wall systems depends mainly on the potential for exposure to a corrosive environment. In Oregon, retaining wall sites with aggressive corrosive environments are typically snow/ice removal zones or marine environment zones as described below.

15.3.21.1 Snow/Ice Removal Zones

Snow/ice removal zones are sections of highway where seasonal snow and ice removal requires the use of de-icing materials containing aggressive compounds that may meet retaining walls. Provide appropriate corrosion protection consistent with the recommendations in Section 15.3.21.2 and the design guidance in Section 15.3.21.3.

15.3.21.2 Marine Environment Zones

Marine environment zones are sections of highway in close proximity to the ocean, a saltwater bay, river or slough, where airborne saltwater spray or saline precipitation could come in contact with the wall. In accordance with 00560.29(b)(1), “On projects within 25 miles of the Pacific ocean, all high strength fasteners shall be galvanized in accordance with 02560.40”, and “In areas visible to the public, clean and prepare fasteners and coat according to Section 00594”. For the purposes of determining when special corrosion protection is required, a Marine Environment is defined as any of the following:

- A location in direct contact with ocean water, salt water in a bay, or salt water in a river or stream at high tide;
- A location within ½ mile of the ocean or a salt water bay with no physical barrier such as hills and forests to prevent strong winds from carrying salt spray generated by breaking waves; or
- A location crossing salt water in a river or stream where there are no barriers such as hills and forests to prevent strong winds from generating breaking waves.

Provide the following minimum protection system for concrete retaining walls and concrete components of retaining walls in a Marine Environment:

- Minimum 2 in. cover on all cast-in-place members.
- HPC (High-Performance Concrete), also known as Microsilica, to be used for all precast and cast-in-place concrete elements.

For retaining walls in a Marine Environment, consider using retaining wall systems that do not use steel soil reinforcements, components, and connections, or provide additional corrosion protection for steel in order to achieve the specified design life. Corrosion protection measures shall consider the following:

- Increase concrete cover;
- Isolate dissimilar metals;
- Use increased corrosion rates for design and increase sacrificial steel thickness accordingly;
- Prevent entry of corrosive runoff into the reinforced backfill;
- Use stainless steel;

- Use cathodic protection;
- Encapsulate steel components; and
- Concrete sealers.

15.3.21.3 Corrosion Protection Design Guidance

AASHTO Articles 11.8.7 (Non-Gravity Cantilever walls), 11.9.7 (Anchored walls), and 11.10.2.3.3 (MSE walls) provide design guidance for corrosion protection.

Subsequent sections of Chapter 15 provide selection and design guidance for corrosion protection of specific retaining wall types.

Corrosion protection should be reviewed with the Corrosion Specialist on a project-by-project basis.

15.3.22 Traffic Railing

Drop-offs at the top of retaining walls shall be protected with traffic railing (barrier) in accordance with the criteria in Section 4.6.2 of the ODOT Highway Design Manual (Current Edition). As a minimum, traffic railing located at the top of retaining walls on ODOT projects shall meet Test Level 3 (TL-3) requirements. A higher Test Level may be required for high-speed freeways, expressways, and interstates where traffic includes a mix of trucks and heavy vehicles, or when unfavorable conditions justify a higher level of rail resistance. Traffic railing options for protection of retaining wall drop-offs include:

- Fixed Bridge Rail on Self Supporting (Moment) Slab: This option consists of a Type “F” 32 in. Bridge rail (BR200) on a self-supporting (moment) slab. The Type “F” 32 in. railing has been crash tested and satisfies TL-4 test criteria in AASHTO LRFD Chapter 13 Railings. The moment slab must be designed in accordance with AASHTO LRFD and the GDM, and must be strong enough to resist the ultimate strength of the railing. The moment slab must also be designed to resist overturning and sliding by its own mass when subjected to a 10-kip static equivalent design load in accordance with AASHTO LRFD 11.10.10.2. ODOT also has a Type “F” 42 in. railing that has been crash tested and satisfies TL-5 criteria, but the static equivalent design load has not been determined.
- Anchored Precast Wide Base Median Railing: Where TL-3 traffic railing is acceptable, anchored precast wide base median barrier (ODOT Standard Dwg. RD500) may be used when designed in accordance with AASHTO LRFD and the GDM. Anchored precast barriers shall be located at least 3.0 ft. clear from the back of the wall face, and each precast section shall be anchored with four vertical anchors as shown on the “Median Installation” option on ODOT Standard Dwg. RD515, and ODOT Standard Dwg. RD516.
- Guardrail: Where TL-3 traffic railing is acceptable, standard guardrail (ODOT Standard Dwg. RD400) may be used when designed in accordance with AASHTO LRFD and the GDM. Locate guardrail posts at least 3.0 ft. clear from the back of the wall face, drive or place posts at least 5.0 ft. below grade, and place at locations that do not conflict with retaining wall elements and components.

See sections of Chapter 15 on specific wall types for wall type specific guidance on design of traffic railings.

15.3.23 Proprietary Minor Retaining Wall Systems

Proprietary minor retaining wall systems are defined in Section 15.2.

Design proprietary minor retaining wall systems in accordance with Chapter 15, except as follows:

- Proprietary Minor retaining wall systems shall be one of the following wall types:
  - Dry cast concrete block prefabricated modular retaining wall systems;
  - Wet cast concrete block prefabricated modular retaining wall systems; or
  - Gabion prefabricated modular retaining wall systems.
- Walls shall include adequate subdrainage to maintain ground water level below bottom of wall and the wall backfill (show on control plans). The sub drainage system shall include a perforated drainage pipe (6-in. diameter PVC) installed near the heel of the retaining wall.
- The retaining wall shall be embedded at least 12 in. below the lowest grade in front of the wall, measured to the bottom of the leveling pad.
- Passive pressure resistance shall be neglected when calculating sliding resistance of the wall.
- Calculate the active lateral earth pressure coefficient (k_a) for wall design using Coulomb earth pressure theory in accordance with AASHTO Article 3.11.5.3 and Section 15.3.10.
- Seismic design is not required.
- A geotechnical investigation is not required.
- Assume foundation soil bearing resistance is adequate at all applicable limit states.
- Assume settlement is tolerable for all applicable limit states.
- Assume backfill soil friction angle (φ) = 34°.
- Assume backfill cohesion (c) = 0 psf.
- Assume backfill moist unit weight (γ_{wet}) = 125 pcf.
- Assume gravel leveling pad angle of internal friction should equal 34°.
- Assume no sliding stability failure within the foundation soil below the gravel-leveling pad.
- Assume only minor cut-and-fill grading for wall construction, as shown in Figure 15-2 that will have no significant effect on overall (global) stability.
- On the project plans, label the wall as a “Minor Retaining Wall”.

15.3.24 Nonproprietary Minor Retaining Wall Systems

Nonproprietary minor retaining wall systems are defined in Section 15.2.

Design nonproprietary minor retaining wall systems in accordance with Chapter 15, except as follows:

- Nonproprietary Minor retaining wall systems shall be one of the following wall types:

- Cast-in-place concrete gravity and semi-gravity retaining wall systems;
- Dry cast concrete block prefabricated modular retaining wall systems;
- Wet cast concrete block prefabricated modular retaining wall systems; or
- Gabion prefabricated modular retaining wall systems.

- Walls shall include adequate subdrainage to maintain ground water levels below the bottom of wall and the wall backfill (show on control plans). The subdrainage system shall include a perforated drainage pipe (6-in. diameter PVC) installed near the heel of the retaining wall.
- The retaining wall shall be embedded at least 12 in. below the lowest grade in front of the wall, measured to the bottom of the foundation or leveling pad.
- Passive pressure resistance shall be neglected when calculating sliding resistance of the wall.
- Calculate active lateral earth pressure coefficient (k_a) for wall design using Coulomb earth pressure theory in accordance with AASHTO Article 3.11.5.3 and Section 15.3.10.
- Seismic design is not required.
- A geotechnical investigation is not required.
- Assume foundation soil bearing resistance is adequate at all applicable limit state.
- Assume settlement is tolerable at all applicable limit states.
- Assume backfill soil friction angle (φ) = 34°.
- Assume backfill cohesion (c) = 0 psf.
- Assume backfill moist unit weight (γ_{wet}) = 125 pcf.
- Assume gravel leveling pad angle of internal friction = 34°.
- Assume no sliding stability failure within the foundation soil below the gravel-leveling pad.
- Assume only minor cut-and-fill grading for wall construction, as shown in Figure 15-2, that will have no significant effect on overall (global) stability.
- On the project plans, label the wall as a "Minor Retaining Wall".

15.3.25 Wall Backfill Testing and Design Properties

Retaining walls may be designed using a higher soil friction angle based on shear strength test measurements performed on representative backfill samples in lieu of using the lower-bound presumptive backfill strength parameters. Measure retaining wall backfill frictional strength by triaxial or direct shear testing methods, ASTM D4767 or AASHTO T236, respectively. Fabricate triaxial or direct shear test samples to within minus 4 percent to plus 2 percent of the optimum moisture content, and to 95 percent of the maximum density determined according to AASHTO T99 Standard Proctor Method A with coarse particle correction according to AASHTO T224. A design friction angle of greater than 40° shall not be used even if the measured friction angle is greater than 40°.

15.3.26 Temporary Shoring and Cut Slopes

15.3.26.1 General Considerations

Temporary shoring is defined as an earth retention and support system that is installed prior to or during excavation using top-down construction techniques. Temporary shoring provides lateral support of in-situ soils and limits lateral movement of soils supporting adjacent structures or facilities, such as bridge abutments, roadways, utilities, and railroads, such that these facilities are not damaged as a result of the lateral soil movements.

Temporary shoring systems include driven cantilever sheet piles, sheet piles with tiebacks, sheet pile cofferdams with wale beams or struts, cantilever soldier piles with lagging, soldier piles with lagging and tiebacks, and multiple tier tieback systems. Temporary cut slopes are also considered shoring, and are included in the definition of Temporary Shoring for contractual purposes. Temporary shoring systems are defined as the following retaining wall system types listed in Section 15.2.4.2:

Table 15-3. Temporary Shoring Systems

Retaining Wall System Type[1]	Retaining Wall System Name	Design Requirements (GDM Section or Special Manual Reference)[2]
5A	Soldier Pile/Lagging Walls	15.8.3
5B	Sheet Pile Walls	15.8.4
5C	Tangent Pile Wall	15.12
5D	Secant Pile Wall	15.12
5E	Slurry (Diaphragm Wall)	15.13
5F	Micropile	FHWA-NHI-05-039
6A	Tie Back Soldier Pile Walls	15.9\15.10
6B	Anchored Sheet Pile Walls	15.9\15.10
7A	Soil Nail Walls	15.11

Notes:
1. **Retaining wall systems listed in Section 15.2.4.2.**
2. **In case of conflict, design requirements in Section 15.3.26 shall take precedence.**

Trench boxes, sliding trench shields, jacked shores, shoring systems that are installed after excavation, and soldier pile, sheet pile, or similar shoring walls installed in front of a pre-excavated slope, are not allowed as shoring.

Unless otherwise noted in the contract plans and specifications, the contractor is responsible for internal and external stability design of temporary shoring, including design of structural components and geotechnical design elements, such as bearing capacity, settlement, sliding, overturning, and compound/global stability. In addition, the shoring design shall address the performance measure requirements in Section 15.3.26.4.

The Agency Professional of Record (POR) may elect to design the temporary shoring in cases such as special construction loading conditions, where shoring provides support of critical adjacent structures or facilities, and/or where shoring is planned within railroad right-of-way, which typically requires railroad review prior to advertisement of the construction contract.

15.3.26.2 Geotechnical Investigation

Geotechnical investigations for temporary shoring and temporary cut slopes shall be in accordance with GDM. 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. However, this is not always the case, and additional explorations and laboratory testing may be needed to complete the shoring design.

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

The scope of the geotechnical investigation for temporary shoring systems shall address any special conditions associated with temporary shoring, construction equipment with high static and/or dynamic loads, elevated hydrostatic/seepage forces from dewatering, and potential ground heave, instability, and/or internal erosion due to seepage gradients from dewatering.

15.3.26.3 Design Requirements

Temporary shoring shall be designed in accordance with the requirements in Division I, Section 5 of the *AASHTO Standard Specifications for Highway Bridges*, 17th Edition (2002) for allowable stress or load factor design, or the *AASHTO LRFD Bridge Design Specifications* (current Edition) including current interims for load and resistance factor design. The shoring design shall also be in compliance with the BDDM and the GDM. In case of conflict or discrepancy between these design specifications and manuals, the GDM shall govern. Temporary shoring design must address all aspects of internal and external stability, including assessment of overturning, sliding, bearing resistance, settlement and compound\global stability. The stability of temporary cut slopes or excavations required for shoring installation shall be assessed and stabilized as needed. Temporary cut slopes, with or without temporary shoring, shall be designed in accordance with the GDM.

Temporary shoring systems maybe designed and constructed utilizing all structural steel or in combination of different materials. All structural steel members can be designed with AASHTO Standard Specifications for Highway Bridges, 17th Edition (2002) or the most current Steel Construction Manual (AISC) for allowable stress design in sizing structural steel members.

FHWA retaining wall design manuals referenced in the GDM (based on allowable stress design) may be used if an approved AASHTO LRFD methodology is not available. The *USS Steel Sheet Piling Design Manuals* (United States Steel, 1984) may be used for shoring walls that do not support other structures and are 15ft or less in height. Whichever design methodology is used for temporary shoring, the design input parameters, including assumed external loads, geotechnical soil/rock properties, and wall material properties, must be clearly stated in the required submittal.

If the temporary shoring design life is 3 years or less, shoring need not be designed for seismic loading. Sufficient corrosion protection should be provided in consideration of the design life of the shoring.

Temporary shoring shall be designed for actual construction-related loads, which can be significantly higher than those assumed in design of permanent structures, such as operation of large cranes or other large equipment near the shoring system. In this case, 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 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.

In accordance with the AASHTO LRFD requirements, compound\global stability analysis shall assume a resistance factor of 0.65, or a factor of safety of 1.5, for temporary shoring systems and/or cut slopes which provide a critical support function, such as support of a structure such as a bridge, retaining wall, sound wall, or building - or any highway embankment which supports an important section of highway. Use a resistance factor of 0.75, or a factor of safety of 1.3, for temporary shoring or cut slopes systems, which do not provide a critical support function.

15.3.26.4 Performance Requirements

Temporary shoring and cut slopes shall be designed to prevent excessive deformation that could result in damage to bridges, buildings, pavements, and other adjacent structures and facilities. The shoring design shall include the determination of actual threshold limits of differential foundation settlement and/or lateral movement that could result in structural damage to adjacent construction. Typical highway structures, including bridge spread footings and CIP concrete retaining walls, can experience unacceptable cracking, displacement, and/or structural damage at a threshold differential settlement of between 1 and 2 inches over a distance of 50 feet. If analysis indicates differential foundation settlement and/or lateral movement will exceed permissible magnitudes, remedial works will be redesigned to prevent damage.

15.3.26.5 OSHA Excavation Safety Requirements

Temporary cut slopes are used extensively to accelerate construction schedules and minimize costs. Since the contractor has control of construction operations, the contractor is responsible for the stability of cut slopes, as well as the safety of the excavations, unless otherwise specifically stated in the contact documents. Because excavations are recognized as one of the most hazardous construction operations, temporary cut slopes must be designed to meet Federal and State regulations in addition to the requirements stated in the GDM. Federal regulations regarding temporary cut slopes are presented in Code of Federal Regulations (CFR) Part 29, Sections 1926.

15.3.26.6 Submittal Requirements

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

1. *Shoring System Geometry*:
    - Has the shoring geometry been correctly developed and all pertinent dimensions shown?
    - Are the slope angle and height above and below the shoring wall shown?
    - Are correct locations of adjacent structures shown?
2. *Performance Objectives for the Shoring System*:
    - Is the anticipated design life of the shoring system identified?
    - Are objectives regarding what the shoring system is to protect, and remedial works to protect it, clearly identified and detailed?

- o Does the shoring system stay within the constraints at the site, such as the right of way limits and boundaries for temporary easements?

3. *Subsurface Conditions:*
   - o Is the design soil/rock profile consistent with the subsurface geotechnical data provided in the contract boring logs?
   - o Did the contractor/shoring designer obtain the additional subsurface data needed to meet the geotechnical exploration requirements for slopes and walls as identified in the GDM?
   - o 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 GDM and the subsurface field and lab data obtained at the shoring site?
   - o Were ground water conditions adequately assessed by comparison of field measurements 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:*
   - o Have the anticipated loads on the shoring system been correctly identified, considering all applicable limit states?
   - o If construction or public traffic is near or directly above shoring system, has a minimum traffic live load surcharge of 250 psf been applied?
   - o 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?
   - o If the shoring system is to be in place longer than three years, have loads from extreme events such as seismic and scour been included in the shoring system design?
5. *Shoring System Design:*
   - o Have the correct design procedures been used (i.e., the GDM and referenced design specifications and manuals)?
   - o 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)?
   - o 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)?
6. *Shoring System Monitoring/Testing:*
   - o Inadequate performance of critical shoring could result in damage to bridges, buildings, pavements, and other adjacent structures and facilities. If critical shoring is planned, is a monitoring/testing plan, such as installation/monitoring of survey points

and/or tension tests of tiebacks, provided to verify adequate performance of the shoring system throughout the design life of the system?

- Have appropriate displacements or other performance triggers been provided that are consistent with the performance objectives of the shoring system?

7. *Shoring System Removal:*

   - Have any elements of the shoring system to be left in place after construction of the permanent structure is complete been identified?
   - 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, and/or create a surface of weakness regarding slope stability)?

15.3.27 Temporary Retaining Walls

15.3.27.1 General Considerations

Temporary retaining walls are defined as any of the following retaining wall system types listed in Section 15.2.4.2:

Table 15.4. Temporary Retaining Walls

Retaining Wall System Type[1]	Retaining Wall System Name	Design Requirements (GDM Section)[2]
Retaining Wall System Types Commonly Used as Temporary Retaining Walls		
2B	Precast Concrete Bin - Prefabricated Modular	15.7.1
2C	Metal Bin - Prefabricated Modular	15.7.1
2D	Gabion - Prefabricated Modular	15.7.3
2F	Wet Cast Concrete Block - Prefabricated Modular	15.7.5
3E	MSE - Welded Wire Facing	15.6
3F	MSE - Gabion Facing	15.6, 15.7.3
8A	MSE - Temporary Geotextile Reinforced Wrapped Facing	15.6.16
Retaining Wall System Types Less Frequently Used as Temporary Retaining Walls		
1A	CIP Concrete Rigid Gravity	15.4
2A	Precast Concrete Crib Prefabricated Modular	15.7.2
2E	Dry Cast Concrete Block Prefabricated Modular	15.7.4
3A	MSE - Dry Cast Concrete Block Facing	15.6
3B	MSE - Wet Cast Concrete Block Facing	15.6
3C	MSE - Precast Concrete Small Panel Facing	15.6

3D	MSE - Precast Concrete Large Panel Facing	15.6
3G	MSE - Two-Stage Facing	15.6
3H	MSE - Precast Concrete "Full Height Panel" Facing	15.6
3K	GRS-IBS Abutment with Dry Cast Concrete Block Facing	15.6.15
4A	CIP Concrete Cantilever Semi-Gravity	15.5

Notes:

1. **Retaining wall systems listed in Section 15.2.4.2.**
2. **In case of conflict, design requirements in Section 15.3.27 shall take precedence.**

Temporary retaining walls shall have a maximum design life of between 6 months and 3 years and are used in construction applications; typically to provide grade separation for approach fills or embankments required for temporary detours.

Unless otherwise noted in the contract plans and specifications, the contractor is responsible for internal and external stability design of temporary retaining walls, such as bearing capacity, settlement, sliding, overturning, and compound/global stability.

The Professional of Record (POR) may elect to design temporary retaining walls in cases of special construction loading conditions or when the wall provides critical structure support - such as temporary detour bridge abutment foundation.

15.3.27.2 Design Requirements

Temporary retaining walls shall be designed in accordance with the requirements in Division I, Section 5 of the *AASHTO Standard Specifications for Highway Bridges*, 17th Edition (2002) for allowable stress design, or the *AASHTO LRFD Bridge Design Specifications* (current Edition) including current interims for load and resistance factor design. The wall design shall also be in compliance with the BDDM and GDM. In case of conflict or discrepancy between these design specifications and manuals, the GDM shall govern. If the wall design life is 3 years or less, the wall need not be designed for seismic loading. Sufficient corrosion protection should be provided in consideration of the temporary wall design life. Design *Temporary Geotextile Reinforced Wrapped Face MSE* retaining walls (Type 8A) in accordance with Section 15.6.16.

Temporary retaining walls shall be designed for actual construction-related loads, which can be significantly higher than those assumed in design of temporary structures, such as operation of large cranes of other large equipment near the wall. In this case, 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 a minimum, the temporary walls shall be designed for a live load surcharge of 250 psf to address routine construction equipment traffic above the wall.

15.3.27.3 Performance Requirements

Temporary walls shall be designed to prevent excessive deformation that could result in damage to temporary detour bridge abutment foundations, pavements, and other adjacent structures and facilities. The temporary retaining wall design shall include the determination of actual threshold limits of differential foundation settlement and/or lateral movement that could result in structural damage to adjacent construction. Typical highway structures (including bridges, pavements, and retaining walls) can tolerate 1 to 2 inches of differential foundation settlement and lateral movement prior to unacceptable cracking, displacement, and/or structural damage. If analysis indicates differential foundation settlement and lateral movement will exceed the threshold magnitudes, the contractor shall design remedial works to prevent damage.

15.4 CIP Concrete Rigid Gravity Walls

15.4.1 General Considerations

Cast-in-place (CIP) gravity retaining walls are reinforced concrete structures that rely on self-weight to resist overturning and sliding forces. Internal stability and external stability (overturning, sliding, bearing capacity, and settlement), and overall (global) stability design of gravity retaining walls shall be performed in accordance with the *AASHTO LRFD Bridge Design Specifications* and the *ODOT GDM*.

15.4.2 Geotechnical Investigation

Design of CIP concrete rigid gravity retaining walls requires a geotechnical investigation to explore, sample, characterize and test foundation soils and measure site ground water levels. Geotechnical investigation requirements for wall foundation design are outlined in Chapter 3.

15.4.3 Wall Selection Criteria

The decision to select a CIP concrete rigid gravity retaining wall should be based on project specific criteria. This decision should also consider the general wall design requirements contained in Section 15.3. CIP gravity walls are not recommended for soft ground sites, or at any location where significant foundation settlements are anticipated.

15.4.4 Wall Height, Footprint and Construction Easement

CIP concrete rigid gravity retaining walls are typically designed to a maximum height of 12 ft. CIP gravity walls typically require an additional lateral construction easement of at least 1.5*H behind the wall to accommodate open-cut construction, drainage installation, backfill placement and compaction behind the wall. A lateral easement restriction and/or the presence of an existing roadway, structure, or utility within the construction limits could require shoring, underpinning and/or right-of-way acquisitions that can affect the construction budget and/or schedule.

15.4.5 Design Requirements

1. CIP concrete rigid gravity retaining walls shall include adequate subdrainage, including drainage blankets, chimney drains, perforated collector pipes and/or weep holes, to relieve hydrostatic pressures and seepage forces on walls in accordance with AASHTO Article 11.6.6 and Section 15.3.18. Additionally, provide adequate surface drainage facilities, including ditches, gutters, curbs and drop inlets, to intercept and direct water to suitable surface water disposal facilities.

2. Calculate static active lateral earth pressures for CIP concrete rigid gravity wall design using Coulomb earth pressure theory in accordance with AASHTO Article 3.11.5.3 and Section

15.3.10. Calculate static passive earth pressures on walls based on Log Spiral and Trial Wedge theories in accordance with AASHTO Article 3.11.5.4 and Section 15.3.10. Calculate seismic active and passive lateral earth pressures in accordance with AASHTO Article 11.6.5.

3. CIP concrete rigid gravity wall design shall assume the maximum wall-backfill friction angle in accordance with AASHTO Article 3.11.5.3 and AASHTO Table 3.11.5.3-1.
4. Development of an active lateral earth pressure assumes the top of the wall can move outward (translate or rotate about the wall base) a distance of at least 0.001*H (dense sand backfill) to 0.004*H (loose sand backfill), where H is the wall height. CIP concrete rigid gravity walls restrained from adequate movement are considered to be non-deflecting walls. Design non-deflecting CIP concrete rigid gravity retaining walls for the at-rest lateral earth pressures and compaction induced lateral earth pressures shown on Figure 15-4 in Section 15.3.11.
5. Calculate base sliding resistance in accordance with AASHTO Article 10.6.3.4.
6. In sliding, lateral resistance shall neglect any contribution from passive earth pressure resistance against the embedded portions of the wall if the soil in front of the wall can be removed or weakened by scour, erosion, construction-excavation, freeze-thaw, shrink-swell, or any other means.
7. Assess external stability (overturning, bearing resistance, sliding, and settlement) and overall (global) slope stability for CIP concrete rigid gravity walls in accordance with the AASHTO LRFD Bridge Design Specifications and the requirements of the ODOT GDM.
8. Where practical, a minimum 4.0-ft-wide horizontal bench shall be provided in front of CIP concrete rigid gravity walls.
9. Design CIP concrete rigid gravity retaining walls for seismic design forces in accordance with AASHTO Article 11.6.5.

15.4.6 ODOT CIP Gravity Retaining Wall Standard Drawing

ODOT has developed the following standard drawing for cast-in-place (CIP) concrete rigid gravity retaining walls:

Standard Drawing BR 720: Standard Gravity Retaining Wall, General Details

Standard Drawing BR 720 is available online at the following website:

http://www.oregon.gov/ODOT/HWY/BRIDGE/standards_manuals.shtml

15.5 CIP Concrete Semi-Gravity Cantilever Walls

15.5.1 General Considerations

Cast-in-place (CIP) semi-gravity cantilever retaining walls are reinforced concrete structures that rely on wall base reaction and friction to resist overturning and sliding forces. Internal stability and external stability (overturning, sliding, bearing capacity, and settlement) and overall stability design of CIP cantilever walls shall be performed in accordance with the *AASHTO LRFD Bridge Design Specifications* and the *ODOT GDM*.

15.5.2 Geotechnical Investigation

The design of CIP cantilever retaining walls requires a geotechnical investigation to explore, sample, characterize and test the wall foundation soils and measure site groundwater levels. Geotechnical investigation requirements are outlined in Chapter 3.

15.5.3 Wall Selection Criteria

The decision to select a CIP cantilever retaining wall should be based on project specific criteria. This decision should also consider the general wall design requirements contained in Section 15.3. CIP cantilever retaining walls can be formed to meet the most demanding vertical and horizontal alignment requirements. A major disadvantage of the CIP cantilever wall is the relatively low tolerance to post-construction foundation settlements. Cantilever walls are not well suited for soft ground sites—or any location where significant foundation settlements are anticipated.

15.5.4 Wall Height, Footprint, and Construction Easement

CIP semi-gravity cantilever retaining walls are typically designed to a maximum height (H) of 24 ft. CIP cantilever walls typically require an additional lateral construction easement of at least 1.5*H behind the wall heel to accommodate open-cut construction, drainage installation, backfill placement and compaction behind the wall.

A lateral easement restriction and/or the presence of a roadway, structure, or utility within the construction limits could require shoring, underpinning and/or right-of-way acquisitions with impacts to the construction budget and/or schedule.

15.5.5 Design Requirements

1. CIP concrete semi-gravity cantilever retaining walls shall have an adequate subdrainage system, including drainage blankets, chimney drains, perforated collector pipes and/or weep holes, to relieve hydrostatic pressures and seepage forces. The subdrainage system shall be designed based on project-specific data and requirements in accordance with AASHTO Article 11.6.6 and Section 15.3.18. Additionally, provide adequate surface drainage facilities, including ditches, gutters, curbs and drop inlets, to intercept and direct water to suitable surface water disposal facilities.
2. The active lateral earth pressure coefficient (k_a) for design of CIP concrete semi-gravity cantilever walls should be calculated using either Coulomb or Rankine earth pressure theory in accordance with the criteria presented in AASHTO Article 3.11.5.3. The active lateral earth pressure shall be applied to a plane extending vertically up from the wall base at the back of the heel. Guidance on application of Coulomb and Rankine theories to cantilever wall design is presented in Figure C3.11.5.3-1 (AASHTO Article 3.11.5.3).
3. Calculate seismic active and passive lateral earth pressures in accordance with AASHTO Article 11.6.5
4. CIP concrete semi-gravity cantilever wall design shall assume the maximum wall-backfill friction angle in accordance with AASHTO Article 3.11.5.3 and AASHTO Table 3.11.5.3-1.
5. CIP concrete semi-gravity cantilever walls restrained from sufficient movement to achieve the active earth pressure condition in accordance with AASHTO Article 3.11.5.3, such as walls bearing directly on bedrock or supported on a deep foundation, are considered to be non-deflecting walls. Design non-deflecting CIP concrete semi-gravity cantilever retaining walls to

satisfy internal and external stability under the combined effects of at-rest lateral earth pressure and compaction lateral earth pressure using Figure 15-4 (Section 15.3.11).

6. Design stems of CIP concrete semi-gravity cantilever retaining walls to satisfy internal stability under effects of compaction lateral earth pressures using Figure 15-4 (Section 15.3.11).
7. Assess external stability (overturning, bearing resistance, sliding, and settlement) and overall (global) slope stability of CIP concrete semi-gravity cantilever walls in accordance with the *AASHTO LRFD Bridge Design Specifications* and the *ODOT GDM*.
8. Where practical, a minimum 4.0-ft-wide horizontal bench shall be provided in front of CIP concrete semi-gravity cantilever walls.
9. Design CIP concrete semi-gravity retaining walls for seismic design forces in accordance with AASHTO Article 11.6.5.

15.5.5.1 Sliding Resistance

Calculate base sliding resistance in accordance with AASHTO Article 10.6.3.4.

In sliding, lateral resistance shall neglect any contribution from passive earth pressure resistance if the soil in front of the wall can be removed or weakened by scour, erosion, construction-excavation, freeze-thaw, shrink-swell, or any other means. If wall base sliding resistance is inadequate, increase the base width. If this does not produce adequate sliding resistance, increase the contribution from passive earth pressure resistance by increasing wall embedment.

A shear key (base key) at least 2.0 feet wide at the bottom and at least 12 inches in depth may be installed along the base of CIP cantilever walls to provide additional sliding resistance. Sliding resistance may include passive earth pressure resistance in front of the base key for foundation materials consisting of stiff to hard, cohesive soil or “extremely soft” to “soft” rock[3] or granular soils in accordance with Figure 10-20, Section 10.5.5 of *Soils and Foundations, Reference Manual – Volume II, FHWA NHI-06-089* (FHWA, 2006).

Neglect any contribution to sliding resistance from passive earth pressure against the base key unless the wall footing base is embedded at least 2.0 ft. below subgrade and the ground in front of the footing will not be weakened or removed by freeze-thaw, shrink-swell, scour, erosion, construction excavation, or any other means.

15.6 Mechanically Stabilized Earth (MSE) Walls

Mechanically stabilized earth (MSE) walls shall be designed (in order of precedence) in accordance with the following:

- *AASHTO LRFD Bridge Design Specifications (*as modified by the *ODOT Geotechnical Design Manual (GDM)*; and
- *Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*, NHI-10-024 and NHI-10-025 (FHWA, 2009).

Unless otherwise noted, MSE wall analysis and design shall assume the following geotechnical properties for the reinforced MSE wall backfill:

[3] “Extremely soft” and “soft” rock refers to the scale of relative rock hardness in accordance with the ODOT Soil and Rock Classification Manual (1987). An “extremely soft” rock has an unconfined compressive strength of less than 100psi, while “soft” rock has an unconfined compressive strength between 1,000 and 4,000psi.

- Friction angle of backfill: $\varphi = 34°$
- Backfill cohesion: c = 0 psf
- Wet unit weight of backfill: γ_{wet} = 130.0 pcf
- Active lateral earth pressure coefficient (k_a) for wall design shall be calculated using the Coulomb earth pressure theory in accordance with AASHTO Article 3.11.5.3 and Section 15.3.10.

15.6.1 General Considerations

MSE walls are internally stabilized by the frictional resistance of layers of steel (inextensible) or geosynthetic (extensible) reinforcement layers embedded within well-compacted, gravel (crushed rock) backfill. MSE walls rely on self-weight to resist overturning and sliding forces. MSE walls are relatively flexible compared to other wall systems and can tolerate relatively large lateral deformations and differential vertical settlements. MSE walls are potentially better suited for earthquake loading effects than other wall systems because of their inherent flexibility and energy absorbing capacity.

MSE wall facing options include small (face area < 30 ft^2) to large (face area ≥ 30 ft^2) square or cruciform-shaped precast concrete panels, full-height precast concrete panels, cast-in-place concrete facing, dry cast and wet cast concrete blocks, welded-wire facing, and rock-filled gabion baskets. Geotextile-reinforced, wrapped-faced MSE walls are frequently used for construction staging and other temporary works.

15.6.2 Geotechnical Investigation

Design of MSE walls requires a geotechnical investigation to explore, sample, characterize and test wall foundation soils and the adjacent ground conditions. Geotechnical investigation requirements are outlined in Chapter 3. At a minimum, the geotechnical information required for wall design includes a subsurface profile including SPT N-values (depth intervals of 5 ft., or less), unit weight, natural water content, Atterberg limit, sieve analysis, soil pH/resistivity, shear strength parameters, settlement/consolidation parameters, foreslope and back slope inclinations, and groundwater levels.

15.6.3 Wall Selection Criteria

MSE walls are relatively wide and heavy structures that frequently require large backcuts, shoring, and/or right-of-way acquisitions.

MSE walls are not recommended at locations where erosion or scour may undermine or erode the leveling pad, facing or MSE reinforced backfill.

Do not place underground utilities in the reinforced backfill zone behind MSE walls. Excavations for utility construction could damage or rupture MSE wall reinforcements - reducing wall stability and causing a failure or collapse of the retaining wall. Fluids from leaking or ruptured utilities could damage or destroy steel or geosynthetic MSE reinforcements and/or wash out of the retaining wall backfill.

15.6.4 Wall Height, Footprint, and Construction Easement

MSE wall heights, including the total wall height of tiered or superimposed MSE walls (Section 15.6.13), shall not exceed 50 ft.

Preliminary reinforcement length (AASHTO Article 11.10.2.1) shall be at least 0.70*H (where H is the wall height shown in AASHTO Figure 11.10.2-1), but not less than 8.0 ft. The minimum AASHTO reinforcement lengths are frequently increased for the following reasons:

- Meet internal, external, compound, and global stability requirements;
- Resist loads from high embankments or sloping backfills, heavy surcharges (both temporary and permanent), and bridge footing or minor structure loads; and
- Meet additional or special requirements for tiered or superimposed walls (Section 15.6.13), back-to-back walls (Section 15.6.14) and MSE bridge retaining walls (Section 15.6.15).

MSE wall backfill slopes shall be no steeper than 1v:2h.

A minimum 4.0-ft-wide horizontal bench shall be provided in front of MSE walls in accordance with AASHTO Article 11.10.2.2. AASHTO Figure 11.10.2-1 provides a sectional view showing a typical MSE wall leveling pad, front face embedment and the required horizontal bench.

15.6.5 Minimum Wall Embedment

Minimum MSE wall embedment depth below lowest adjacent grade in front of the wall shall be in accordance with AASHTO Article 11.10.2.2, including the minimum embedment depths indicated in Table C11.10.2.2-1.

The minimum MSE wall embedment depth, as shown in AASHTO Figure 11.10.2-1, shall be based on external stability analysis (sliding, bearing resistance, overturning, and settlement) and the global (overall) stability requirements in *AASHTO LRFD* Chapters 10 and 11 and the *ODOT GDM*.

The embedment depth of MSE walls along streams and rivers shall be at least 2.0 ft. below the potential scour elevation in accordance with AASHTO Article 11.10.2.2. The potential scour elevation shall be established in accordance with *AASHTO LRFD*, the *ODOT BDDM*, and the *ODOT Hydraulics Manual*.

15.6.6 External Stability Analysis

External stability analysis shall include calculation of sliding resistance, soil bearing resistance, overturning, and settlement at the applicable LRFD load factor combinations and resistance factors. External stability analysis shall also consider compound stability failure surfaces that pass through the MSE wall reinforced backfill. Overall (global) stability shall be in accordance with Section 15.6.6.3.

15.6.6.1 Sliding Resistance

Sliding resistance along the base of the MSE wall shall be calculated using the procedures in AASHTO Article 10.6.3.4. Calculate sliding resistance using the using parameters in AASHTO Tables 10.5.5.2.2-1 and 11.5.7-1.

Neglect any beneficial effect of external loads on MSE walls (such as live load traffic surcharge) that increase sliding resistance.

At a minimum, sliding stability analysis shall determine the minimum resistance along the following potential failure surfaces:

- Surface within reinforced backfill;
- Surface within foundation soil or rock material;

- Interface between reinforced backfill and foundation soil/rock material;
- Interface between reinforced backfill and reinforcement; and
- Interface between foundation soil or rock and reinforcement.

In sliding, neglect any contribution to stability from passive earth pressure resistance. Neglect any benefit the wall facing elements provide to sliding stability.

15.6.6.2 Soil Bearing Resistance, Overturning, and Settlement

Soil bearing resistance design shall be in accordance with Chapter 10 in AASHTO LRFD and the ODOT GDM. The effective footing dimensions of eccentrically loaded MSE walls shall be evaluated in accordance with AASHTO Article 10.6.1.3. Calculate foundation settlement in accordance with AASHTO Article 10.6.2.4 and Chapter 6 and Chapter 8.

Excessive MSE wall foundation settlement can result in damage to the wall facing, coping, traffic barrier, bridge superstructure, bridge end panel, pavement, and/or other settlement-sensitive elements supported on or near the wall. Techniques to reduce damage from post-construction settlements and deformations include:

- A "two-stage" MSE wall system where the first stage is a flexible-faced MSE wall (e.g., geotextile wrapped face or welded-wire) to preload and/or surcharge the foundation, followed by the permanent wall facing in front of the first-stage MSE wall. A wall minimum "wait period"

  is required after construction of the first-stage MSE wall to allow enough time for soil consolidation to reduce or eliminate damaging, long-term (post-construction) foundation settlements.
- Prefabricated vertical drains or wick drains may be appropriate to accelerate the time-rate of foundation soil consolidations and reduce total construction time. *Prefabricated Vertical Drains, Volume I, Engineering Guidelines,* FHWA/RD-86/168 (FHWA, 1986) provides detailed guidance for the planning, design and construction of prefabricated vertical drains. Material and construction requirements for wick drains are provided in Section 00435.
- Full-height vertical sliding joints through the rigid wall facing elements and appurtenances.
- Ground improvement or reinforcement techniques as described in Chapter 11. Staged preload/surcharge construction, using suitable onsite materials and/or imported fill, may be a relatively cost-effective method to increase MSE wall stability and/or reduce settlement.

15.6.6.3 Global Stability

The overall (global) stability of MSE walls shall be evaluated in accordance with AASHTO Articles 11.6.2.3 and 11.10.4.3, and ODOT GDM Chapter 6, Chapter 8, and Chapter 15. The mass of the MSE wall (or the "foundation load") may be assumed to contribute to the overall stability of the slope. MSE wall stability analysis shall consider the internal, compound, and overall stability failure surfaces shown in AASHTO Figures 11.10.2-1 and 11.10.4.3-1.

15.6.6.4 Seismic External Stability

MSE walls have performed relatively well during earthquakes—tolerating large lateral deformations and differential vertical settlements without failure or collapse. MSE walls are potentially better suited for earthquake loading than other retaining wall types because of their inherent flexibility and energy absorbing capacity.

Design MSE retaining wall seismic external stability in accordance with AASHTO Article 11.10.7 and Section 15.3.13.

15.6.7 Internal Stability Analysis

Internal stability analysis shall include calculation of reinforcement loading, pullout, reinforcement and reinforcement-facing connection strengths.

15.6.7.1 Loading

The maximum factored tension loads in MSE wall reinforcements (T_{max}) shall be calculated at each reinforcement level using either the *Simplified Method* or Coherent Gravity Method approach in accordance with AASHTO Article 11.10.6.2. The factored load applied to the reinforcement-facing connection (T_o) shall be equal to the maximum factored tension reinforcement load (T_{max}) in accordance with AASHTO Article 11.10.6.2.2.

15.6.7.2 Reinforcement Pullout

Calculate MSE wall reinforcement pullout capacity in accordance with AASHTO Article 11.10.6.3.

The location of the maximum surface of stress for steel (inextensible) and geosynthetic (extensible) reinforced MSE walls shall be determined in accordance with AASHTO Figure 11.10.6.3.1-1. Reinforcement pullout shall be checked at each reinforcement level in accordance with AASHTO Article 11.10.6.3.2 and the effective pullout length in the reinforcement zone shall be calculated using AASHTO Equation 11.10.6.3.2-1.

The pullout friction factor (F*) for geosynthetic reinforcement shall be from product-specific laboratory testing that measures the interface friction by direct shear method in accordance with ASTM D5321-02. The rate of horizontal displacement shall be adjusted to result in drained shearing conditions during the test. Alternatively, F* may be estimated using conservative values from AASHTO Figure 11.10.6.3.2-1. The scale effect correction factor (α) may be estimated from AASHTO Table 11.10.6.3.2-1.

15.6.7.3 Reinforcement Strength

Design steel and geosynthetic reinforcement strength in accordance with AASHTO Article 11.10.6.4.

In accordance with AASHTO Article 11.10.6.4, the maximum factored reinforcement loads shall be calculated at each reinforcement level in the MSE wall based on AASHTO Equation 11.10.6.4.1-1. The maximum factored load at reinforcement-facing connections shall be calculated based on AASHTO Equation 11.10.6.4.1-2.

The nominal, long-term reinforcement design strength shall be calculated at each reinforcement level in accordance with AASHTO Articles 11.10.6.4.3a (steel reinforcement) and 11.10.6.4.3b (geosynthetic reinforcement).

15.6.7.4 Reinforcement-Facing Connection Strength

The nominal, long-term reinforcement-facing connection design strength (T_{ac}) shall be calculated as specified in AASHTO Article 11.10.6.4.4a (steel reinforcement) and AASHTO Article 11.10.6.4.4b (geosynthetic reinforcement).

The reinforcement-facing connection strength of MSE walls shall be designed to resist lateral loads on the facing from the following factors:

- Lateral earth pressure and water pressure loads;
- Compaction and construction loads;
- Live loads and surcharges, including traffic loads;
- Dead loads and surcharges, including backslope and approach fill;
- Structure foundation loads; and
- Seismic loads.

The reinforcement-facing connection strength of MSE walls shall also be designed to resist stresses due to differential movement between the facing and the reinforcement resulting from backfill compaction, differential settlement between the wall facing and reinforced backfill, or other effects.

15.6.7.5 Seismic Internal Stability

Design MSE retaining wall seismic internal stability in accordance with AASHTO Article 11.10.7.2.

15.6.8 Wall Drainage

MSE walls shall include an internal drainage system that meets the following requirements:

- Subsurface drainage design requirements in *ODOT GDM* Section 15.3.18, *AASHTO LRFD*, and *NHI-10-024 (*FHWA, 2009*)*;
- Prevents infiltration of aggressive runoff, seepage and/or groundwater into the facing or reinforced backfill zone - avoiding the resulting damage from corrosion or degradation effects; and
- Intercepts surface and subsurface water from around and beneath the MSE wall, including the reinforced backfill zone, and rapidly removes the water to a suitable discharge location.

MSE wall subdrainage typically consists of a suitable-placed trench, chimney, and/or blanket drain with perforated collector drainpipes to intercept and remove groundwater seepage and percolating surface runoff. The collector pipe is connected to a solid pipe that should discharge into an approved drainage ditch or storm drain system. Provide properly located clean outs for the collector pipe. Permeable materials used in drainage systems shall be encapsulated in a drainage geotextile (geotextile filter) layer. The drainage system shall be designed to maintain groundwater levels below the base of the MSE wall reinforced backfill zone.

Perforated collector and solid pipes shall be at least 6in diameter to allow for periodic pipe flushing and cleaning, irrespective of discharge capacity requirements. Pipe discharge points shall be readily accessible to maintenance personnel. Provide metal screens or secure caps at pipe ends to prevent rodent entry.

Design of walls along rivers, creeks, canals, detention basins, retention basins, or other situations with potential for water level fluctuation shall apply a 3.0 ft. (min.) differential hydrostatic head to the MSE wall to simulate rapid drawdown conditions in accordance with AASHTO Article 11.10.10.3. A greater hydrostatic head should be used to model larger river or tidal level fluctuations if supported by hydraulics data.

See Section 5.3 Drainage, in *NHI-10-024* (FHWA, 2009) for examples of common drainage details for MSE walls.

15.6.9 Traffic Railing

The requirements of this section are for traffic railing on MSE walls, and are supplemental to the requirements of Section 15.3.22.

Fixed Bridge Rail on Self Supporting (Moment) Slab for MSE walls:

Where TL-4 traffic railing is acceptable, a self-supporting moment slab with 32-in. Type "F" bridge rail (ODOT Standard Drawing BR760) may be used on MSE walls in accordance with the GDM.

Self-supporting moment slabs with Type "F" 32 in. bridge railing should be designed in accordance with all of the following bullets:

1. Meet the requirements of AASHTO Article 11.10.10.2.
2. Be externally stable when subjected to a static equivalent vehicle collision force of 10.0 kips.
3. The minimum total length of the moment slab should be 30.0 ft. For moment slabs longer than 30.0 ft., a length of moment slab assumed to be effective in resisting overturning and sliding should not exceed 30.0.
4. Moment slab overturning calculations should assume that the slab rotates about "Point A" shown on Figure 15-5. Please note that in order to satisfy eccentricity limits, the required width of a moment slab with 12-in.-thick facing will be greater than the required width of a moment slab with 6-in.-thick facing.
5. Be in conformance with ODOT Standard Drawing BR200 Type "F" 32 in. Bridge Railing.
6. Live load should be neglected when it acts to resist eccentricity (AASHTO 10.6.4.2).
7. Moment slab eccentricity under Extreme Event II limit state should not exceed 0.33*B, where B is the width of the moment slab (measured perpendicular to wall) bearing on the reinforced backfill. The method used in AASHTO Article 11.6.5 to calculate the eccentricity limit for Extreme Event I (seismic) is assumed to also be applicable to Extreme Event II.
8. The moment slab should be isolated from the MSE wall facing so that loads from the moment slab are not transferred directly to the MSE wall facing. Isolate slab by placing a 1-in. minimum thickness of compressible neoprene foam between slab and wall facing.
9. Provide ¾" transverse expansion joints (perpendicular to wall face) in the moment slab at a maximum spacing of 90 ft. Provide coinciding expansion or open joints in the Type "F" bridge rail in accordance with BR200. Moment slab expansion joint design should include corrosion resistant shear transfer dowels designed to transfer moment slab forces to adjacent moment slabs, and designed to accommodate moment slab longitudinal expansion. For slabs covered by asphalt, the asphalt should be saw cut at the location of the expansion joint and filled with poured rubber-asphalt joint filler. For slabs not covered with asphalt, the top 1½ in. of the slab joint should be filled with poured rubber-asphalt joint filler. Stop slab longitudinal reinforcing bars 3 in. from joint.
10. Provide transverse contraction joints in the moment slab at a maximum spacing of 30 ft., equally spaced between expansion joints, and provide coinciding contraction joints in the Type "F" bridge rail in accordance with BR200. Moment slab contraction joint design should include corrosion resistant shear transfer dowels designed to transfer moment slab shear forces to adjacent moment slabs. Moment slab contraction joints should be formed or should include joints saw cut to a depth of one third of the slab depth. For slabs covered by asphalt, the asphalt should be saw cut at the location of the contraction joint and the joint filled with

poured rubber-asphalt joint filler. For slabs not covered with asphalt, the slab joint should be filled with poured rubber-asphalt joint filler. Stop longitudinal reinforcing bars 3 in. from joint.

11. Design slab reinforcement in accordance with AASHTO LRFD Chapter 5 Concrete Structures.

12. Figure 15-5 shows the dimensions for a self-supporting (moment) slab with Type "F" 32 in. bridge rail. Figure 15-5 is based on 12-in.- thick MSE wall facing and is in accordance with the recommendations in numbers 1 through 8, above. Figure 15-5 does not provide details for the recommendations in numbers 9 through 11 above.

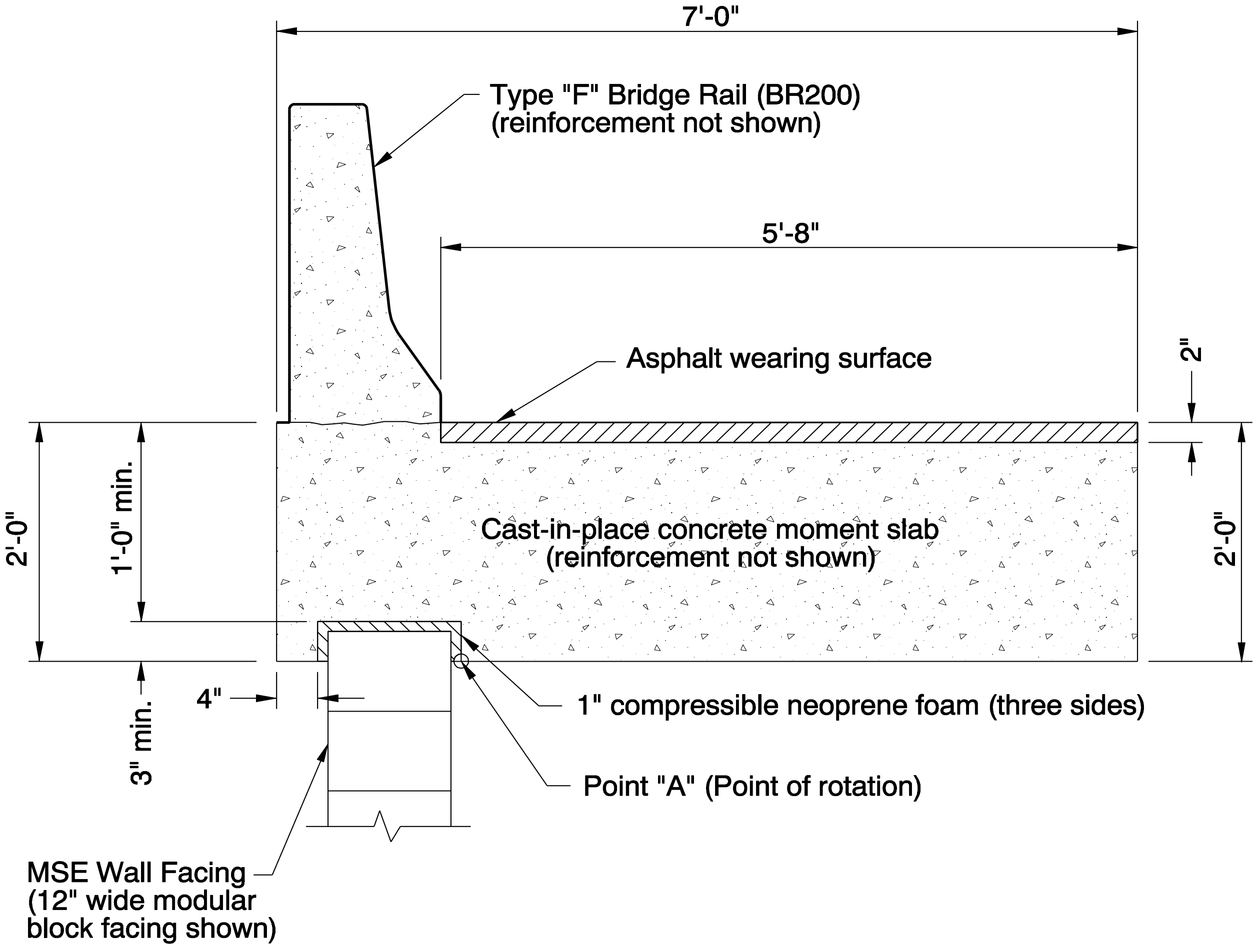
Design MSE walls to ensure soil reinforcements do not rupture or pullout due to vehicle impact loads on traffic railing using the following criteria:

- Dynamic vehicle impact loads should be distributed to the upper two layers only of soil reinforcement below the traffic railing as described in FHWA-NHI-10-024, 025 (FHWA, 2009) using the following unfactored line loads:

    - *Soil Reinforcement Rupture*: Design the upper soil reinforcement layer for a rupture impact load equivalent to a static load of 2,300 lb.\ft. of wall; and the second soil reinforcement layer designed for a rupture impact load equivalent to a static load of 600 lb.\ft. of wall.

    - *Soil Reinforcement Pullout*: Design the upper soil reinforcement layer for a pullout impact load equivalent to a static load of 1,300 lb.\ft. of wall; and the second soil reinforcement layer designed for a pullout impact load equivalent to a static load of 600 lb.\ft. of wall.

- In calculating the factored long-term tensile resistance of soil reinforcements, use a resistance factor of 1.0 for the "Combined static/traffic barrier impact" loading condition (Table 4-7, FHWA-NHI-10-024) for both metallic grid and geosynthetic soil reinforcements. Add the factored line loads for soil reinforcement rupture or reinforcement pullout to the factored static earth pressure loads to calculate the total reinforcement loading.



Note 1 - Dimensions shown assume a minimum moment slab length of 30 feet.

Note 2 - See GDM Sections 5.3.22 and 5.6.10.

Figure 15-5. Fixed Type “F” 32” Bridge Rail on Self Supporting (Moment) Slab – MSE Wall

Precast Median Barrier:

Where TL-3 traffic railing is acceptable, anchored precast wide base median barrier (RD500) may be used when designed in accordance with AASHTO LRFD and the GDM. Anchored precast barriers shall be located at least 3.0 ft. clear from the back of the wall face, and shall be anchored with two vertical anchors on each side of each precast section, in accordance with the “Median Installation” option shown on ODOT Standard Drawings RD515 and RD516.

Guardrail:

Where TL-3 traffic railing is acceptable, standard guardrail (BR400) may be used when designed in accordance with AASHTO LRFD and the GDM.

Design MSE wall soil reinforcement in accordance with AASHTO Articles 11.10.10.2 and 11.10.10.4 and the *ODOT GDM*.

Where guardrail posts are required to be constructed in MSE Walls, the posts shall be placed at a minimum horizontal distance of 3.0 ft. from the back of the wall face to the back of the guardrail post, driven or placed at least 5.0 ft. below grade and spaced at locations to miss the reinforcement materials where possible. Installation of the guardrail shall not damage any portion of the retaining wall. If the reinforcement cannot be missed, the wall shall be designed accounting for the presence of an obstruction in the reinforced soil zone using one of the following methods:

1. Assuming the reinforcement must be partially or fully severed in the location of the guardrail post, design the surrounding reinforcement layers to carry the additional load that would have been carried by the severed reinforcements. The portion of the wall facing in front of the guardrail post shall be made stable against a toppling (overturning) or sliding failure. If this cannot be accomplished, the soil reinforcements between the guardrail post and the wall face can be structurally connected to the obstruction such that the wall face does not topple, or the facing elements can be structurally connected to adjacent facing elements to prevent this type of failure.
2. Place a structural frame around the guardrail post capable of carrying the load from the reinforcements connected to the structural frame in front of the obstruction to the reinforcements connected to the structural frame behind the obstruction.
3. If the soil reinforcements consist of discrete, inextensible (steel) strips and depending on the size and locations of the guardrail posts, it may be possible to splay the reinforcements around the guardrail posts. The splay angle, measured from a line perpendicular to the wall face, shall be small enough that the splaying does not generate moment in the reinforcement or the connection of the reinforcement to the wall face. The tensile resistance of the splayed reinforcement shall be reduced by the cosine of the splay angle.

Method 3 above would be effective if guardrail posts are installed at the same time as the MSE wall is constructed (i.e., the wall is built around the guardrail posts). If the guardrail posts are installed after the wall is constructed, it is possible that the splayed reinforcements were installed in the wrong location and the guardrail post installation could damage them. It may also be possible to build the wall around casings or guides, such as Corrugated Metal Pipe (CMP), into which the guardrail posts could be installed after the wall is completed.

15.6.10 Corrosion

Corrosion protection is required for all permanent MSE walls and for temporary MSE walls (design life of three years or less) in aggressive environments as defined in AASHTO Article 11.10.6.4.2. AASHTO Article 11.10.2.3.3 provides design guidance for corrosion protection of MSE walls.

As discussed in Section 15.3.21, aggressive environmental conditions in Oregon are typically associated with snow or ice removal zones and marine environment zones. In snow/ice removal zones, where aggressive deicing materials are likely to be used, protect MSE wall steel reinforcements from the corrosive effects of aggressive runoff with a properly designed and detailed impervious membrane layer placed below the pavement and above the top level of backfill reinforcement. The membrane shall be sloped to quickly move runoff seepage to a drainage collector pipe located behind the reinforced backfill zone.

15.6.11 Two-Stage Facing (MSE Retaining Wall Systems)

Design two-stage facing for MSE retaining wall system in accordance with the methodologies and procedures presented in Section 5.4.7 (Two-Stage Facing), Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes (FHWA, 2009).

15.6.12 (Reserved for Future Use)

15.6.13 Tiered or Superimposed Walls

Design tiered or superimposed MSE walls in accordance with the methodologies and procedures presented in *Section 6.2 Superimposed (Tiered) MSE Walls, Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes* (FHWA, 2009). FHWA (2009) shall be used to address aspects of tiered wall design not covered in the ODOT GDM or AASHTO LRFD.

The total height (H) of a tiered or superimposed MSE retaining wall shall be the sum of the heights of the lower tier wall (H_2) and the upper tier (H_1) as shown on Figure 6.7, Section *6.2* (FHWA, 2009). The total height (H) of a tiered MSE retaining wall height shall not exceed 50 ft. In accordance with FHWA (2009), where the face-to-face distance (D) between the lower and upper MSE wall tiers exceeds at least $1.5*H_2$, these walls are not considered tiered and may be designed independently.

Perform seismic external stability design of tiered MSE walls according to Section 15.6.6.4. Perform seismic internal stability design of tiered MSE walls according to Section 15.6.7.5.

General design guidance that applies to tiered MSE walls is provided in Section 15.3.5.

15.6.14 Back-to-Back Walls

Design back-to-back MSE walls in accordance with the methodologies and procedures presented in "Section 6.4 Back-to-Back MSE Walls, Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes" (FHWA, 2009).

15.6.15 MSE and GRS-IBS Bridge Retaining Walls

15.6.15.1 MSE Bridge Retaining Walls

MSE bridge retaining walls with either steel or geogrid reinforcements may be designed to support bridge abutments with spread footings or pile foundations.

MSE bridge retaining walls shall meet the following requirements as needed to provide for adequate personnel access for maintenance and inspection of bridge bearings and shear lugs:

- Provide a horizontal clear distance from the MSE wall backface to the front of the adjacent bridge spread footing or pile cap of at least 3 ft.; and
- Provide a vertical clear distance from finish grade behind the wall facing to the base of the overhead bridge superstructure of at least 4 ft.; for bridges with a solid bottom, such as a concrete box girder, provide 5 ft. minimum.

The above minimum distance requirements for personnel access are in addition to all other applicable design requirements in the ODOT GDM. These minimum distances are shown in Figure 15-6.

Design MSE bridge retaining walls in accordance with the following:

- ODOT GDM;
- AASHTO LRFD; and
- "Section 6.1 (Bridge Abutments) Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes" (FHWA, 2009).

In case of a conflict or discrepancy between the above design references, the order of precedence shall be ODOT GDM, AASHTO LRFD, and then FHWA.

Design MSE walls supporting bridge abutment spread footings using the following values of bearing resistance of the reinforced backfill zone:

- For Service Limit State, bearing resistance = 4,000 psf
- For Strength Limit State, factored bearing resistance = 7,000 psf
- For Extreme Event Limit State, factored bearing resistance = 8,000 psf

Design MSE bridge abutment walls at pile or drilled shaft supported abutments for horizontal bridge loads dependent on the type of deep foundation support if recommended in the geotechnical report.

- Isolating piles from MSE construction by placing casing larger than the pile size is one way to mitigate the downdrag as well as horizontal stresses while the casing remains unfilled. Piles are driven and isolation casing is placed prior to MSE wall construction. The space between the pile and isolation casing is filled after MSE wall construction and before pile cap construction. Corrugated metal pipe and pea gravel infill are materials that have been successfully used for pile isolation.
- For deep foundations without isolation casing and constructed before the MSE wall, evaluate downdrag forces induced by MSE wall construction and compression of foundation soil.
- In all cases consider the effect of pile spacing, skewed abutment corners, and other obstructions that may interfere with soil reinforcement. Where reinforcement layers at wall corners stagger/intersect, consider cover required between reinforcement layers, where metal reinforcement is near steel piles, casings, culverts, or other metal obstructions consider cover required between dissimilar metals.
- Consider the effect of seismic displacement and seismic forces transferred from the bridge.

Facing shall be CIP reinforced concrete, reinforced precast concrete panels, dry cast concrete blocks, wet cast concrete blocks, or sprayed on concrete/mortar fascia constructed after welded wire facing (two-stage wall). Installing one of these facing types in front of a wire-faced MSE system complies with this requirement.

Do not place integral abutment bridge foundations on top of, or through, MSE walls.

Full-height precast concrete facing panels shall not be used for MSE bridge retaining walls.

15.6.15.2 MSE Bridge Retaining Walls with Steel Reinforcements

The following design requirements apply to spread footing abutments:

- Provide a clear distance of at least 18 in. between the back of the MSE wall facing and the front edge of the bridge abutment spread footing.

The following design requirements apply to pile supported abutments:

- Provide a clear distance of at least 18 in. between the back of MSE wall facing and the front edge of the nearest pile.
- Provide a clear distance of at least 6 in. between the back of the wall facing and the pile cap.

15.6.15.3 MSE Bridge Retaining Walls with Geogrid Reinforcements

The following design and construction requirements apply to spread footing abutments:

- Figure15-6 provides a typical sectional view of a geogrid-reinforced MSE wall supporting a bridge abutment spread footing.
- Geogrid-reinforced MSE walls supporting bridge abutments shall use a geogrid reinforcement product listed under the product category name *Type 1 MSEW Geogrid* on the ODOT Qualified Products List (QPL).
- The facing/reinforcement connection system shall be an approved mechanical connection system that does not rely on the frictional resistance between the soil reinforcement (geogrid) and the facing blocks.
- The geogrid-reinforced MSE wall height (H_1 in Figure15-6) shall not exceed 23 ft.
- The bridge abutment height (H_2 in Figure15-6) shall not exceed 10 ft.
- Total wall height (H' in Figure15-6) shall not exceed 33 ft.
- Geogrid reinforcement vertical spacing (S_v) shall not exceed 16 in. between layers.
- MSE walls shall be reinforced with uniformly spaced, horizontal geogrid layers along the entire height of the wall as indicated in Figure15-6.
- The vertical distance between the uppermost geogrid reinforcement layer and the top surface grade behind the wall facing shall not exceed 16 in.
- The depth of wall facing below the lowest reinforcement layer shall not exceed 8 in.
- The width of the bridge abutment spread footing, supported on the geogrid-reinforced MSE wall, shall be at least 2.0 ft., but not greater than 15.0 ft.

The following design requirements apply to pile supported abutments:

- For pile foundations through MSE walls, provide a clear distance of at least 18 in. between the back of MSE wall facing and the front edge of the nearest pile; and
- Provide a clear distance of at least 6 in. between the back of the wall facing and the pile cap.

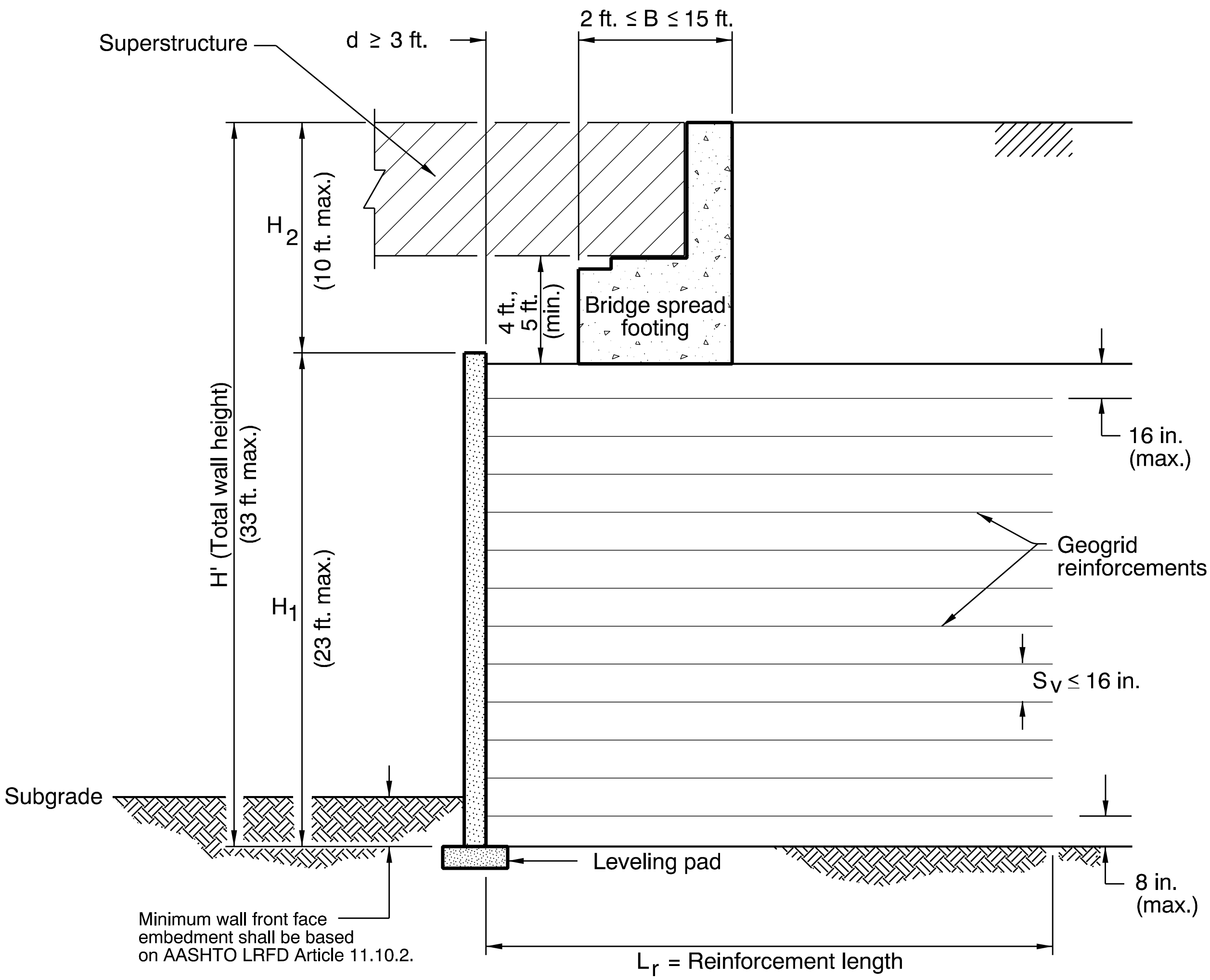


Figure 15-6. MSE Wall Supporting Bridge Abutment Spread Footing (Geogrid Reinforcements)

15.6.15.4 Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS) Bridge Abutment

Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS) bridge abutments are part of FHWA' s Every Day Counts (EDC) initiative to reduce bridge construction time and cost. FHWA prepared design report/manuals, and sample construction specifications and drawings for its use. These resources may be found on [FHWA' s Accelerating Innovation]() website. GRS-IBS integrates the bridge structure with the approach roadway to create a jointless system. GRS-IBS abutments have been constructed in several states across the nation since 2005.

GRS-IBS components and functions:

- GRS abutment and wingwalls with closely spaced geotextile or geogrid reinforcement layers comprise the GRS mass. A bearing bed beneath the beam seat serves as an embedded footing. Concrete on a compressible foam board creates a buffer between the block facing and the beam seat bearing area. The bearing bed consists of more densely spaced geosynthetic reinforcement layers to distribute load in the GRS mass. The GRS mass behaves as a composite, internally stabilized unit. The facing elements of the GRS mass serve as surface protection; the facing is not a primary support of the GRS mass. The wall/abutment embedment, bearing width, and foundation improvements (if needed) are based on the required geotechnical bearing resistance, wall settlement limitations, and all internal, external, and overall (global) wall stability requirements.

- The transition from the bridge to the approach roadway (integrated approach) is composed of a GRS mass compacted behind the end of the bridge beams. Provide integrated approach for a minimum length of 12-feet or 3-feet beyond the cut-slope excavation limit, whichever is greater. Concrete approach slabs are not required.

Design GRS-IBS bridge abutments and retaining walls using LRFD methodology in accordance with the following in order of precedence:

- ODOT GDM, BDDM and Hydraulics Manuals
- FHWA-HRT-11-026, “Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide”, except as noted otherwise in the above ODOT manuals
- AASHTO, 2014, LRFD Bridge Design Specifications, American Association of State Transportation and Highway Officials, 7th Edition (with current Interims)
- FHWA NHI-10-024 Volume I and NHI-10-025 Volume II, “ Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes”, (Berg et al., 2009)

Figure 15-7 provides a typical sectional view of a GRS-IBS bridge abutment and approach for an application in cut. In cut applications, the base may be truncated to reduce excavation, backfill and reinforcement.

Overview of design and construction constraints for use of GRS-IBS:

- Geotechnical investigations outlined for MSE walls are applicable for GRS-IBS.
- Follow the ODOT BDDM 1.10.5.3 and related sections when GRS-IBS is considered at water crossings.
- GRS-IBS may be considered for locations with a low seismic hazard ($As \leq 0.15g$ for the 1000 year return period).
- Maximum 30 ft. wall height
- Maximum reinforcement spacing:
    - GRS backfill: 8-inch maximum spacing based on typical dry cast block size. The FHWA manual limits the primary reinforcement to less than or equal to12 inches.
    - Beam seat and bearing bed: one half the GRS backfill reinforcement spacing
    - Integrated approach behind the bridge superstructure: 12-inch maximum wrap layers with fill placed and compacted in not more than 6-inch lifts and an intermediate reinforcement layer with each lift.
- The base of the wall may be truncated to reduce excavation. Wall embedment, bearing width, and foundation improvements (if needed) are based on geotechnical evaluation, wall settlement limitations, and all internal, external, and overall (global) wall stability evaluation. Provide 8 ft. minimum wall bearing width and 2 ft. minimum embedment.

- Use a concrete or gravel leveling pad to provide a uniform, flat bearing surface to support the facing blocks. Foundation settlement serviceability criteria for MSE walls in Section 15.3.15 apply. Unless otherwise specified, or evaluated with refined analysis, use the allowable relative distortions between adjacent foundations given in AASHTO C10.5.2.2.
- The facing blocks are frictionally connected to the reinforcement layers, except the top three courses of facing block are also filled with concrete and pinned with rebar.
- Use geosynthetic reinforcement with strength not less than 400 lb/in and meeting the requirements of Boilerplate Special Provision SP02320 Geosynthetics Table 02320-7, Geotextile Property Values for Geosynthetic Reinforced Retaining Walls. Limit the required reinforcement strength to less than the reinforcement strength at 2 percent strain.
- The bearing reinforcement zone serves as an embedded footing within the GRS mass. The reinforcement density is doubled in this zone and extends at least 2 ft. beyond the beam seat. The bearing bed depth is based on internal stability and should not be less than 5 courses.
- The beam seat is designed to satisfy the 4,000 psf service limit bearing stress and 7,000 psf strength limit bearing capacity. The purpose of the beam seat is to ensure that the superstructure bears on the GRS abutment and not the wall facing block, and to provide the necessary clear space between the superstructure and the wall face.
- Except as noted below, compact reinforced backfill that is placed within 3 feet behind wall facing units to 95% of maximum density using walk-behind vibratory rollers or vibratory plate compactors that have sufficient static and dynamic forces to achieve compaction without causing distortion of the wall facing units. Compact reinforced backfill that is placed 3 feet or more behind wall facing units to 95% of maximum density using riding smooth drum vibratory roller or other suitable equipment made specifically for compaction. The top 5 feet should be compacted to 100% of maximum density, Granular Structure Backfill is recommended for its demonstrated ability to achieve the required compaction using AASHTO T-99.
- For GRS-IBS at water crossings, use open-graded aggregate as the reinforced backfill for the full reinforced width and to the height of the design flood elevation for bridge. Encapsulate this material with a geotextile filter along the interface of the subgrade beneath it, the retained backfill behind it, and the reinforced backfill above it. As field density tests are not suitable for open-graded backfill, development of a procedural specification and greater inspection presence for the compaction quality assurance is needed. NTPEP reports have been prepared for geosynthetic tested using well-graded backfill material; therefore additional geosynthetic testing is also required where open-graded backfill is used. Long term tensile strength of reinforcement should include a reduction factor for installation damage based on the specific reinforcement and open-graded aggregate properties (i.e., maximum particle size, gradation, angularity).
- Limit use to short span bridges and beam seats meeting the bearing requirements above. The majority of bridges built with GRS-IBS have spans less than 100 ft. The FHWA implementation guide recommends that engineers limit bridge spans to 140 ft until more research has been completed.

Provide adequate background information and documentation in the TS&L Report for the structure type selections. Minimum requirements include:

- Survey – Right of Way, buried obstructions, wetlands limits
- Geotechnical Subsurface Exploration and Preliminary Geotechnical Report – Evaluate settlement (total and differential), identify groundwater levels, discuss the need for foundation improvements
- Structure - Design life, durability and corrosion protection measures
- Environmental – Discuss environmental constraints
- Constructability- Discuss constructability challenges, if present
- Backfill - Compaction and drainage provisions
- Cost Comparison of structure options considered

Other relevant project specific information required by the Bridge TS&L template

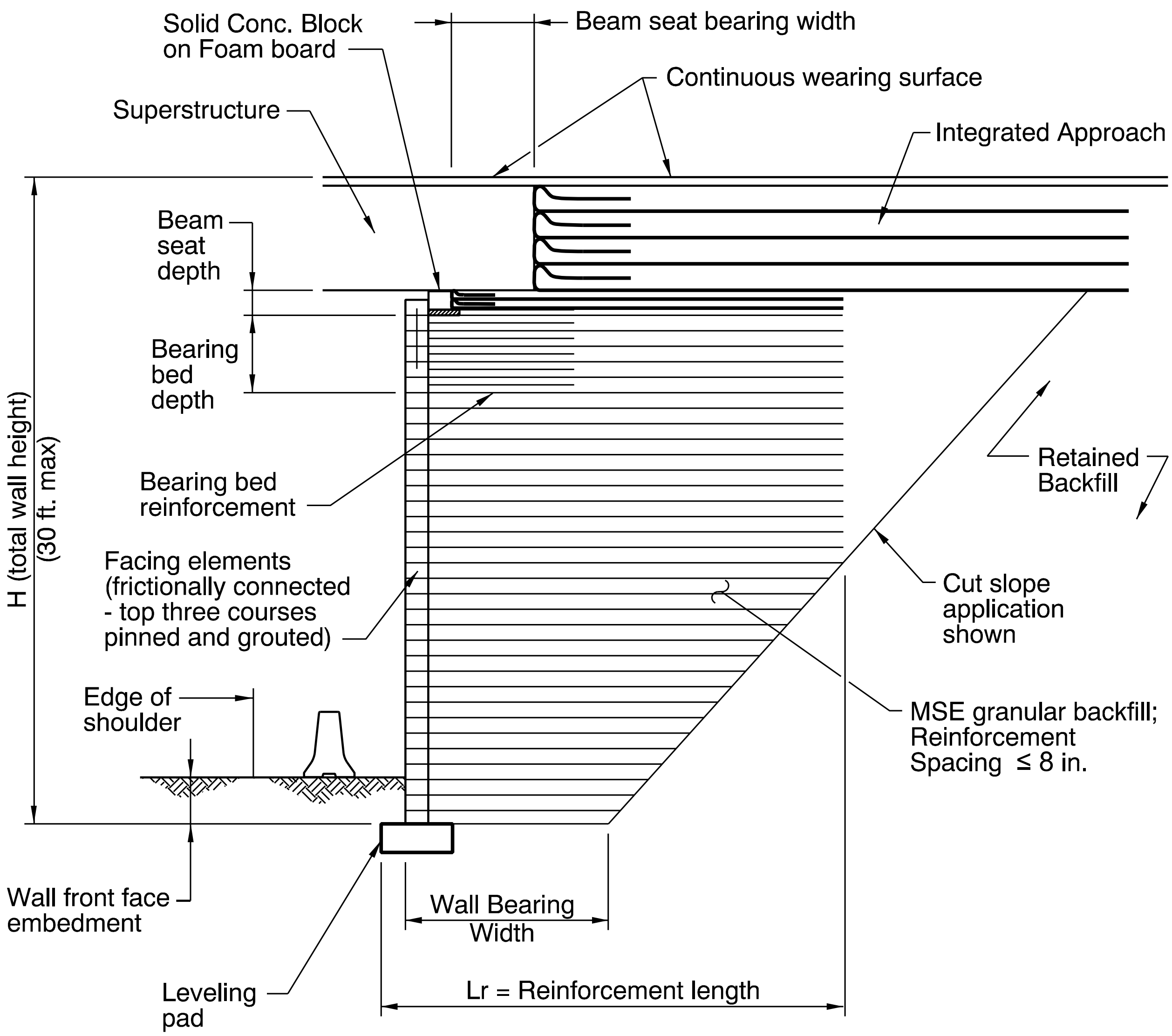
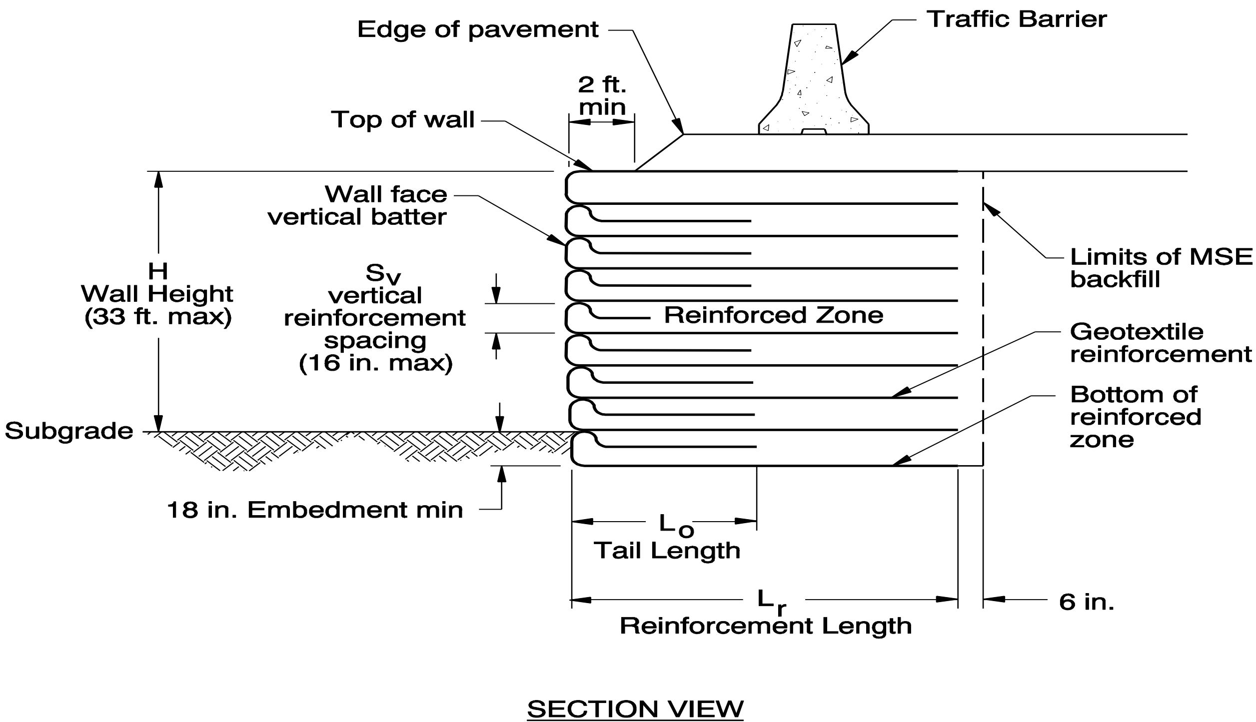


Figure 15-7 GRS-IBS Abutment Section

15.6.16 Temporary Geotextile-Reinforced MSE Wall

This section presents design and construction requirements for temporary wrapped-face, geotextile-reinforced MSE walls. Temporary geotextile walls consist of continuous, sheet-type geotextile reinforcement layers constructed alternatively with horizontal layers of compacted MSE wall backfill. The wall face is formed by wrapping each geotextile layer around and back into the overlying lift of backfill. A typical temporary geotextile wall is shown in Figure 15-8.



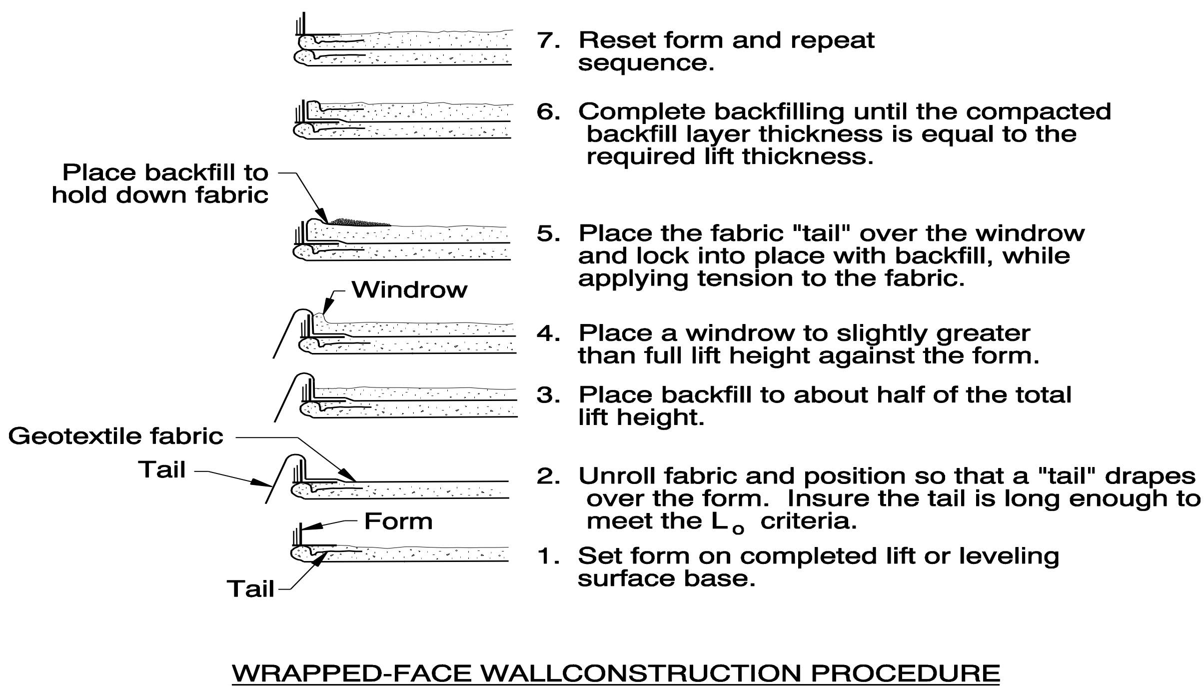


Figure 15-8. Temporary Geotextile-Reinforced MSE Wall

Note the welded wire mesh forms may also be left in place for ease of construction and improved facing alignment of temporary wrapped-face geotextile - reinforced MSE walls.

Temporary geotextile walls are typically used for detours, bridge construction staging, and roadway widening. These walls are relatively low cost and use lightweight materials. Construction is relatively rapid and does not require specialized labor or equipment. As indicated in Section 15.3.15, geotextile wrapped-face MSE walls can tolerate relatively large magnitudes of settlement without significant damage.

Design requirements presented below assume temporary geotextile walls support roadway construction that is relatively settlement-tolerant, such as guardrail, ditches, traffic barrier, and flexible pavements. Temporary walls supporting relatively settlement-sensitive structures, such as bridges, sound walls, retaining walls, critical utilities, and buildings, for which the consequences of excessive foundation movement, adverse performance or failure are severe, shall be designed for the level of safety and/or performance consistent with permanent construction in accordance with the *AASHTO LRFD Bridge Design Specifications* and the *ODOT Geotechnical Design Manual (GDM)*.

15.6.16.1 Design Requirements

Temporary geotextile-reinforced MSE retaining walls shall be designed in accordance with the requirements in Division I, Section 5 of the *AASHTO Standard Specifications for Highway Bridges*, 17th Edition (2002) for allowable stress design or the *AASHTO LRFD Bridge Design Specifications*. Design shall be in compliance with the ODOT GDM. In case of conflict or discrepancy between these design specifications and manuals, the GDM shall govern. The following additional design requirements apply to temporary geotextile-reinforced MSE walls:

1. Design temporary geotextile-reinforced MSE walls for the period of the project construction, or a service life of three years, whichever is greater. Walls remaining in service for more than 3 years shall be designed as permanent MSE walls.
2. Design geotextile reinforcement for temporary walls using Total Reduction Factor (RF) values from Table 11.10.6.4.3b-1 in the AASHTO LRFD Bridge Design Specifications.
3. Design of temporary geotextile-reinforced MSE walls with construction penetrating the wall (i.e. utilities, drainage pipes and culverts) shall explicitly consider local internal and external wall stability effects from the penetration. Design temporary geotextile walls with penetrations in accordance with the requirements of *Chapter 5 MSE Wall Details* in NHI-10-024 (FHWA, 2009). Provide project-specific plans and details showing modifications to MSE wall construction at the wall penetration(s).
4. The maximum wall height (H) shall not exceed 33 ft. (Figure 15-8).
5. The minimum wall reinforcement length (L_r shown in Figure 15-8) shall be the greater dimension of the following:
    - 70 percent of the total wall height (H) in accordance with AASHTO Article 11.10.2.1;
    - 8.0 ft. in accordance with AASHTO Article 11.10.2.1; or
    - The minimum reinforcement length required to meet all external, internal, and overall (global) stability requirements.

6. Temporary geotextile walls shall have uniformly spaced, horizontal geotextile reinforcement layers from wall bottom to top as indicated in Figure 15-8. The geotextile reinforcement vertical spacing (S_v) shall not exceed 16in between adjacent layers.
7. Fill construction along the top of temporary geotextile-reinforced MSE walls shall be set back a horizontal distance of at least distance 2.0ft from the top of the wall as indicated in Figure 15-8.
8. Calculate the lateral stress σ_h (max) and associated geotextile reinforcement loads for temporary geotextile wall internal stability design using the Simplified Method in accordance with AASHTO Article 11.10.6.2.1.
9. Internal and external stability design of temporary geotextile walls shall be performed using the most current versions and updates of the computer programs MSEW and ReSSA® (ADAMA Engineering, Inc.). Wall sliding (external stability consideration) frequently controls the minimum required wall reinforcement length (L_r).
10. Design submittal and construction drawings shall indicate design geotechnical properties assumed for the reinforced MSE backfill, wall backfill and/or back-cut materials, and wall foundation soils. Also provide the following information: design minimum and maximum groundwater levels; type, size, and location of wall subdrainage system(s); geotextile reinforcement properties; assumed location and magnitude of wall surcharge and fill; live and dead loads assumed in internal and external stability design.
11. External, internal and global stability design shall evaluate all applicable limit states in accordance with *AASHTO LRFD* during construction and over the design service life of the temporary geotextile wall. External, internal and global stability design shall consider potential impacts on the wall stability, including:
    - Loss of ground support in front or adjacent to the temporary wall from excavation or any construction activity;
    - High point load or surcharge from the operation of heavy construction equipment operation or material storage within a horizontal distance H from the wall (H = wall height);
    - Effects of full hydrostatic pressures and seepage forces on wall;
    - Damage or removal of geotextile reinforcement layers from construction activities; and
    - Damage or removal of portions or all geotextile walls facing from vandalism, vehicle impact, debris impact, fire, and/or other reason.
12. Global stability design shall include investigation of internal, compound, and overall shear failure surfaces that penetrate the MSE wall reinforced backfill, cover fill, backfill or back-cut, and/or foundation soils. Global (overall) stability design shall be performed using any state-of-the-practice computer program, such as the most current versions of Slope/W® (Geo-Slope International), or ReSSA® (ADAMA Engineering, Inc.).
13. Evaluation of sliding resistance (external stability) shall neglect any contribution from passive earth pressure resistance provided by embedment of temporary geotextile walls.

14. Calculate foundation bearing capacity and settlement of geotextile-reinforced MSE walls in accordance with *Chapter 10* of *AASHTO LRFD* and the *ODOT GDM*. Design ground

improvement for temporary geotextile wall construction, as needed, to mitigate inadequate foundation support conditions.

15.7 Prefabricated Modular Walls

Prefabricated modular walls without soil reinforcement, such as metal and precast concrete bins, precast concrete cribs, dry cast concrete blocks, wet cast concrete blocks, and gabions shall be considered prefabricated modular walls. Design prefabricated modular walls as gravity retaining structures in accordance with *AASHTO LRFD* and the *ODOT GDM*.

Design prefabricated modular gravity walls for seismic design forces in accordance with AASHTO LRFD Articles 11.6.5 and 11.11.6, Section 15.3.13, and the following recommendations:

- Prefabricated modular wall seismic design shall include global, external (i.e., sliding, overturning, and bearing), and internal stability analyses.
- Global and external stability checks need to consider failure surfaces that pass through the wall section at joints and significant changes in wall cross-sectional geometry - as well as surfaces passing below the wall base. Stability checks should include the additional shear resistance from the structural interlocking that occurs along joints between modular wall components.
- Check wall sliding, overturning, and toppling stability for modular walls along and above joints between wall elements, especially at significant changes in wall cross-section for stacked and/or multi-depth walls.

Prefabricated modular walls shall not be used as a Bridge Abutment or Bridge Retaining Wall unless designed to meet the seismic design performance requirements in accordance with Chapter 6.

15.7.1 Metal and Precast Concrete Bin Retaining Walls

15.7.1.1 General Considerations

Metal and precast concrete bin retaining walls are typically rectangular, interlocking, prefabricated concrete modules or bolted lightweight steel members stacked like boxes to form retaining walls. The bin wall modules are filled with well-graded, compacted gravel (crushed rock) to create heavy gravity structures with sufficient mass to resist overturning and sliding forces. Metal and concrete bin walls come in a variety of dimensions.

15.7.1.2 Geotechnical Investigation

Design of metal and precast concrete bin walls requires a geotechnical investigation to explore, sample, characterize and test the wall foundation soils and measure site groundwater levels. Geotechnical investigation requirements are outlined in Chapter 3.

15.7.1.3 Wall Selection Criteria

Metal bin walls are subject to corrosion damage from exposure to aggressive surface water runoff, infiltration or seepage typically associated with snow/ice removal or marine environment zones see Section 15.3.21, or potentially from exposure to aggressive backfill materials, in-place soils along wall back-cuts, or in-place foundation soil or rock. Open-faced bin walls are subject to damage from erosion (backfill loss through face) where the wall face is exposed to flowing water, excessive hydrostatic pressures and/or seepage forces.

15.7.1.4 Layout and Geometry

The wall base width of bin walls shall not be less than 3.0 ft. An additional horizontal easement is required behind the wall to accommodate the wall backcut. Bin walls are not recommended for applications that require a radius of curvature less than 800ft. The wall face batter shall not be steeper than 10° or 6v:1h.

15.7.1.5 Design Requirements

1. Metal and precast concrete bin retaining walls shall be designed in accordance with the *AASHTO LRFD Bridge Design Specifications* and the *ODOT GDM*.
2. The minimum wall embedment depth shall meet all requirements in *AASHTO LRFD* and the *ODOT GDM*.
3. Wall backfill slopes shall be no steeper than 1v:2h.
4. Where practical, a minimum 4.0ft wide horizontal bench shall be provided in front of walls.
5. Unless otherwise noted, external and internal stability analysis and design of metal and precast concrete bin retaining walls shall assume the following geotechnical properties for bin module fill and wall backfill:
    - Friction angle of backfill: $\varphi = 34°$;
    - Backfill cohesion: c = 0 psf; and
    - Backfill moist unit weight ($\gamma_{\mathbf{wet}}$) = 120 pcf.

15.7.1.6 External Stability Analysis

Active earth pressures shall be calculated using Coulomb earth pressure theory in accordance with *AASHTO LRFD*. Lateral earth pressures shall be calculated in accordance with AASHTO Articles 3.11.5.3 and 3.11.5.9. Apply calculated lateral earth pressure along the back of bin walls in accordance with AASHTO Article 3.11.5.9 (Figures 3.11.5.9-1 and 3.11.5.9-2).

Calculate the lateral active earth pressure thrust on metal and precast concrete bin retaining walls with a broken backslope, point load(s) or surcharge(s), groundwater effects, and/or with a non-uniform soil (backfill) profile, using the *Culmann* or *Trial Wedge* methods such as presented in Soil Mechanics in Engineering Practice (Terzaghi and Peck, 1967) or NAVFAC DM-7.01 and DM-7.02 (U.S. Navy, 1982).

Bin walls require a properly designed subdrainage system in accordance with AASHTO Article 11.11.8 and Section 15.3.18, including a drainage geotextile layer along the backside of metal and precast concrete bin walls to prevent the intrusion of fine-grained soil into or through the bin modules.

External stability analysis shall include sliding, overturning, soil bearing resistance, settlement, and overall (global) stability based on the applicable *AASHTO LRFD* load factor combinations and resistance factors. Additionally, evaluate bin wall sliding and overturning stability at each module level of the wall. The wall base may be slightly sloped into the backfill to improve overturning stability in accordance with AASHTO Article 3.11.5.9.

Calculate sliding lateral resistance in accordance with AASHTO Article 10.5.5.2.2 and Table 10.5.5.2.2-1.

The maximum eccentricity limits of the resultant force acting on the base shall meet the requirements of AASHTO Article 10.6.3.3. These requirements apply to each module level of the wall.

The effective footing dimensions of eccentrically loaded bin walls in overturning shall be evaluated in accordance with *AASHTO LRFD Specifications*. Design shall assume no greater than 80 percent of the weight of the bin module backfill is effective in resisting bin wall overturning forces in accordance with AASHTO Article 11.11.4.4. Soil bearing resistance design shall be in accordance with AASHTO Article 10.6.3.1.

The overall (global) stability shall be evaluated in accordance with AASHTO Article 11.6.2.3 and the *ODOT Geotechnical Design Manual (GDM)*, with the exception that the mass of the bin wall (or the "foundation load"), may be assumed to contribute to the overall stability of the slope.

External stability analysis shall also meet seismic design requirements in accordance with AASHTO Article 11.6.5 and Chapter 6, Chapter 8 and Chapter 15.

15.7.2 Precast Concrete Crib Retaining Walls

15.7.2.1 General Considerations

Precast concrete crib walls are interlocking, concrete stretcher and header elements cross-stacked to form rectangular modules. The front and rear stretchers form the front and rear sides of the wall with headers placed transverse to the stretcher units. Crib wall modules are filled with well-graded, compacted gravel (crushed rock) backfill to create a gravity wall with sufficient mass to resist overturning and sliding forces. Precast concrete crib walls come in a variety of dimensions.

15.7.2.2 Geotechnical Investigation

Design of precast concrete crib walls requires a geotechnical investigation to explore, sample, characterize and test the wall foundation soils and measure site groundwater levels. Geotechnical investigation requirements are outlined in Chapter 3.

15.7.2.3 Wall Selection Criteria

Open-faced crib walls are subject to damage from loss of backfill materials through the face and developing root systems that can cause uplift, cracking or separation of bin modules. Open-faced crib walls are also subject to damage from erosion (backfill loss through face) where the wall face is exposed to flowing water, excessive hydrostatic pressures and/or seepage forces.

15.7.2.4 Layout and Geometry

The crib wall base width shall not be less than 3.0 ft. An additional horizontal easement is required behind the wall to accommodate the wall backcut. Crib walls are not recommended for applications that require a radius of curvature less than 800 ft. The wall face batter shall not be steeper than 4v:1h.

15.7.2.5 Design Requirements

1. Precast concrete crib retaining walls shall be designed in accordance with the *AASHTO LRFD Bridge Design Specifications* and the *ODOT GDM*. Minimum wall embedment depth shall meet all requirements in *AASHTO LRFD*.

2. Wall backfill slopes shall be no steeper than 1v:2h.
3. Where practical, a minimum 4.0-ft-wide horizontal bench shall be provided in front of wall
4. Precast concrete bin retaining walls shall meet all seismic design requirements in *AASHTO LRFD* and the *ODOT GDM*.
5. Unless otherwise noted, external and internal stability analysis and design of precast concrete crib retaining walls shall assume the following geotechnical properties for crib module fill and wall backfill:
   - Friction angle of backfill: $\varphi = 34°$;
   - Backfill cohesion: c = 0 psf; and
   - Backfill moist unit weight (γ_{wet}) = 120 pcf

15.7.2.6 External Stability Analysis

Active earth pressures for single-cell crib walls shall be calculated using Coulomb earth pressure theory in accordance with *AASHTO LRFD*, and Rankine earth pressure theory shall be used for multi-depth walls. Lateral earth pressures shall be calculated in accordance with AASHTO Article 3.11.5.3 and 3.11.5.9. Apply calculated lateral earth pressure along the back of crib walls in accordance with AASHTO Article 3.11.5.9 (Figures 3.11.5.9-1 and 3.11.5.9-2). Use maximum wall friction angles in Table C3.11.5.9-1.

Crib walls require a properly designed subdrainage system in accordance with Section 15.3.18, including a drainage geotextile layer along the back stretcher and end header units of crib walls to prevent fine-grained soil intrusion into or through the modules.

External stability analysis shall include sliding, overturning, soil bearing resistance, settlement, and overall (global) stability based on the applicable LRFD load factor combinations and resistance factors.

The maximum eccentricity limits of the resultant force acting on the crib wall base shall meet the requirements of AASHTO Article 10.6.3.3. These requirements apply to each module level of the wall.

Check crib wall sliding stability along the following potential failure planes:

- Interface between foundation base (gravel or concrete leveling pad) and the subsoil;
- Between lowest crib base stretcher and header elements and the leveling pad; and
- Within the crib structure (including all changes in wall section for multi-depth walls).

Ignore benefit from lugs, interlocking dowels, or other crib wall modifications when assessing sliding resistance between crib elements.

In sliding, lateral resistance shall neglect any contribution from passive earth pressure resistance.

Check crib wall sliding and overturning stability at the following points:

- Toe of the crib wall (stretcher or header);
- Toe of the rigid concrete leveling pad (crib wall foundation) below the crib wall; and
- Any joint between crib wall elements at the wall face—including changes in wall section for multi-depth walls.

Check crib wall for toppling failure above joints between crib wall elements—including changes in wall section for multi-depth walls.

The wall base may be slightly sloped into the backfill to improve overturning stability in accordance with AASHTO Article 3.11.5.9.

The effective footing dimensions of eccentricity loaded crib walls in overturning shall be evaluated in accordance with AASHTO LRFD Specifications. Design shall assume no greater than 80 percent of the weight of the crib module backfill is effective in resisting crib wall overturning forces in accordance with AASHTO Article 11.11.4.4. Soil bearing resistance design shall be in accordance with AASHTO Article 10.6.3.1.

The overall (global) stability shall be evaluated in accordance with AASHTO Article 11.6.2.3 and the ODOT GDM—with the exception that the mass of the crib wall (or the "foundation load") may be assumed to contribute to the overall stability of the slope.

15.7.2.7 Internal Stability Analysis

Design crib wall headers and stretchers as beams with fixed ends supported at their intersections and subjected to loads and pressures from the module fill, wall backfill, and base reactions. Design shall consider any potential failure mode, including tension, compression, shear, bending, and torsion.

Crib wall members shall be designed for lateral pressures as indicated on Figure 5.10.4.1-1 in *Section 5 - Retaining Walls, Bridge Design Specifications* (Caltrans, 2004). Design forces on front, intermediate and rear stretchers, headers and base members shall be in accordance with Figure 5.10.4.1-1 through 6.

15.7.3 Gabion Walls

15.7.3.1 General Considerations

Gabion walls consist of heavy wire mesh baskets filled with hard, durable stone to form rectangular modules referred to as gabion baskets. The standard ODOT gabion basket unit has a depth, height and length of 36 in.

Gabion walls are typically less than 18 ft. in height and are designed as gravity structures in accordance with *AASHTO LRFD*.

15.7.3.2 Geotechnical Investigation

Design of gabion walls requires a geotechnical investigation to explore, sample, characterize and test the wall foundation soils and the adjacent ground conditions. Geotechnical investigation requirements are outlined in Chapter 3.

15.7.3.3 Wall Selection Criteria

Gabion walls are vulnerable to corrosion damage from aggressive foundation soils and backfill and where runoff, stream or river water is acidic or aggressive. Gabions are also vulnerable to damage due to abrasion from rock impacts and debris in flowing water.

Gabion baskets are subject to corrosion damage from exposure to aggressive surface water runoff, infiltration or seepage typically associated with snow/ice removal or marine environment zones see Section 15.3.21, or potentially from exposure to aggressive in-place soils along wall back-cut or foundation areas. Corrosion protection for gabion baskets typically requires the use of stainless steel materials or galvanized metal materials with polyvinyl chloride (PVC) coating. Epoxy coating of gabion baskets is not recommended as the primary method of corrosion protection due to limited design life. Project specific conditions should be evaluated to determine the required level of corrosion protection for gabion basket walls.

Gabions are most economical if there is a local source of suitable stone for basket fill. Gabion walls are well suited for developing vegetation cover.

Gabion walls are relatively free draining and well suited for stream and riverbank applications. A drainage geotextile layer is typically required behind between gabion modules and the surrounding backfill and foundation soil to prevent the intrusion of finer-grained soil particles through the open stone gabion basket fill.

15.7.3.4 Layout and Geometry

The wall base width shall not be less than 3.0 ft. An additional horizontal easement is required behind the wall to accommodate the backcut. The wall face batter shall not be steeper than 6° or 10v:1h.

15.7.3.5 Design Requirements

1. Gabion walls shall be designed as gravity structures in accordance with the AASHTO LRFD Bridge Design Specifications and the ODOT Geotechnical Design Manual (GDM).
2. Wall backfill slopes shall be no steeper than 1v:2h.
3. Where practical, a minimum 4.0 ft. wide horizontal bench shall be provided in front of walls.
4. Gabion baskets shall be arranged so vertical seams are staggered and not aligned. The gabion steel wire mesh material shall have adequate strength, flexibility, and durability for the project site conditions and intended use. Gabion walls shall meet all seismic design requirements in accordance with the *AASHTO LRFD* and the *ODOT GDM*.
5. To prevent internal erosion and excessive migration of soil particles through the gabion units, place drainage geotextile filter (or drainage geotextile) layers around portions of gabion units in contact with soil.

15.7.3.6 External Stability Analysis

Active earth pressures for gabion wall design shall be calculated using Coulomb earth pressure theory in accordance with *AASHTO LRFD* Specifications. Lateral earth pressures shall be calculated in accordance with AASHTO Article 3.11.5.3 and 3.11.5.9.

Apply calculated lateral earth pressure along the back of gabion walls in accordance with AASHTO Article 3.11.5.9 (Figures 3.11.5.9-1 and 2). Use maximum wall friction angles in Table C3.11.5.9-1. Groundwater conditions creating unbalanced hydrostatic pressures shall be considered in external stability analysis.

Unless otherwise noted, gabion wall analysis and design shall assume the following geotechnical properties for the wall backfill:

- Friction angle of backfill: $\varphi = 34°$;
- Backfill cohesion: c=0 psf; and
- Backfill moist unit weight (γ_{wet}) = 125 pcf.

The wall face batter shall not be steeper than 10v:1h to maintain the resultant wall force towards the back of the wall.

Calculate lateral sliding resistance in accordance with AASHTO Article 10.5.5.2.2 (Table 10.5.5.2.2-1).

In sliding, lateral resistance shall neglect any contribution from passive earth pressure resistance.

Provide durable, 4- to 8-in.-diameter rock fill material for gabion baskets meeting the requirements of 00390.11(b). Gabion basket material shall consist of suitable rock materials (i.e. Basalt, Sandstone, or Granite) meeting the requirements of Section 00390 (Riprap Protection), except suitable rounded rock material is permitted.

Unless project specific data are available, external stability analyses shall assume the rock-filled gabion baskets have a bulk density (total unit weight) in accordance with Table 15-5.

Table 15-5. In-Place Porosity vs. Bulk Density, Gabion Basket Rock Fill

Rock Type	Rock Specific Density (pcf)	Gabion Basket Rock Fill Porosity (n)[4]		
		n = 0.30	n = 0.35	n = 0.40
Basalt	170.0	119.0	110.5	102.0
Sandstone	150.0	105.0	97.5	90.0
Granite	160.0	112.0	104.0	96.0

Rock filled gabion baskets require a properly designed geotextile filter fabric material to prevent the intrusion of fine-grained soil into the stone filled baskets.

External stability analysis shall include sliding, overturning, soil bearing resistance, settlement, and overall (global) stability based on the applicable LRFD load factor combinations and resistance factors.

Soil bearing resistance design shall be in accordance with *AASHTO LRFD* and the *ODOT GDM*. Calculate foundation settlement in accordance with AASHTO Article 10.6.2.4.

The overall (global) stability shall be evaluated in accordance with AASHTO Article 11.6.2.3 and the ODOT GDM. The mass of the gabion wall (or the "foundation load") may be assumed to contribute to the overall stability of the slope.

15.7.4 Dry Cast Concrete Block Gravity Walls

15.7.4.1 General Considerations

Dry cast concrete block gravity retaining walls consist of a single row of dry stacked blocks (without mortar) that resist overturning, base sliding, and shear forces through self-weight of the blocks and the retained backfill. Design of dry cast concrete block gravity retaining walls shall be performed in accordance with the *AASHTO LRFD Bridge Design Specifications*.

[4] The in-place bulk density (γ_g) is calculated from rock specific density (γ_s) and in-place porosity (n) based on the following relationship: $\gamma_g = \gamma_s*(1-n)$.

15.7.4.2 Geotechnical Investigation

Design of dry cast concrete block gravity retaining walls requires a geotechnical investigation to explore, sample, characterize and test foundation soils and measure groundwater levels. Geotechnical investigation requirements for wall foundation design are outlined in Chapter 3.

15.7.4.3 Wall Selection Criteria

The decision to select a dry cast concrete block gravity retaining wall should be based on project specific criteria. This decision should consider the general wall design requirements contained in Section 15.3. Dry cast concrete block gravity retaining walls can be formed to a tight radius of curvature of 10 ft or greater see Section 15.3.4.

Dry cast concrete block gravity retaining walls shall only be considered if used in conjunction with properly designed surface water drainage facilities and a subdrainage system see Section 15.3.18 that prevents surface water runoff or groundwater seepage contact with the dry cast concrete face and maintains groundwater levels below the base of the wall.

15.7.4.4 Wall Height, Footprint and Construction Easement

Dry cast concrete block gravity walls are typically designed to a maximum height of 10 ft. Dry cast concrete block gravity retaining walls typically require an additional lateral construction easement of at least 1.5*H (H = wall height) behind the wall heel to accommodate open-cut construction, drainage installation, backfill placement and compaction behind the wall. A lateral easement restriction and/or the presence of an existing roadway, structure, or utility within the construction limits could require shoring, underpinning and/or right-of-way acquisitions that can affect the construction budget and/or schedule.

15.7.4.5 Design Requirements

1. Dry cast concrete block gravity retaining walls shall include adequate subdrainage, including drainage blankets, chimney drains, and/or perforated collector pipes to relieve hydrostatic pressures and seepage forces on walls in accordance with Section 15.3.18. Additionally, provide adequate surface drainage facilities, including ditches, gutters, curbs and drop inlets, to intercept and direct water towards suitable discharge locations as described below.
2. Dry cast gravity retaining walls shall have backfill slopes no steeper than 1v:2h.
3. Where practical, a minimum 4.0 ft.-wide horizontal bench shall be provided in front of dry cast gravity walls.
4. The dry cast wall subdrainage system and surface drainage facilities shall prevent surface water runoff or groundwater seepage contact with the dry cast concrete face and maintain groundwater levels below the base of the wall.
5. Assess internal stability, external stability (soil bearing resistance, settlement, eccentricity and sliding), and overall (global) slope stability for dry cast concrete block gravity retaining walls in accordance with the AASHTO *LRFD Bridge Design Specifications* and the *ODOT Geotechnical Design Manual (GDM)*.
6. Active earth pressures acting on dry cast concrete block gravity retaining walls should be calculated using Coulomb earth pressure theory in accordance with AASHTO Article 3.11.5.3 and Section 15.3.10.

7. Calculate the lateral active earth pressure thrust on dry cast concrete block walls with a broken backslope, point load(s) or surcharge(s), groundwater effects, and/or with a non-uniform soil or backfill profile using the *Culmann* or *Trial Wedge* methods as presented in Section 5, *Retaining Walls, Bridge Design Specifications* (Caltrans, 2004), *Soil Mechanics in Engineering Practice* (Terzaghi and Peck, 1967), or *Foundation Analysis and Design, 5th Edition* (J. E. Bowles, 1996).
8. Unless otherwise noted, dry cast concrete block gravity wall analysis and design shall assume the following geotechnical properties for the wall backfill:
   - Friction angle of backfill: $\varphi = 34°$;
   - Backfill cohesion: c = 0 psf;
   - Backfill moist unit weight (γ_{wet}) = 125 pcf; and
   - Friction angle of gravel leveling pad fill: $\varphi = 34°$.
9. Internal sliding stability shall be checked at each dry cast concrete block level from the lowest block to the top of wall. Dry cast facing must have sufficient interface shear capacity to transfer lateral loads to the base of the structure without excessive wall translation, bulging, or damage. Interface sliding resistance between dry cast concrete blocks shall be calculated using the corrected wall weight based on the calculated hinge height in accordance with AASHTO Figure and Equation 11.10.6.4.4b-2. Dry cast block interface friction resistance parameters shall be based on product-specific data using NCMA Test Method SRWU-2 (*Determination of Shear Strength between Segmental Concrete Units*) in accordance with Appendix C.2 in NCMA (2002).
10. Calculate bearing resistance in accordance with AASHTO Article 10.6.3.1.
11. Calculate base sliding resistance (external stability) in accordance with AASHTO Article 10.6.3.4. Sliding resistance analysis shall address dry cast units bearing on gravel or on cast-in-place concrete leveling pads. The total vertical force used to calculate sliding resistance shall be corrected based on the corrected height of the dry cast column (hinge height) calculated in accordance with AASHTO Equation 11.10.6.4.4b-2. The calculated hinge height shall not exceed the wall height.
12. In sliding, lateral resistance shall neglect any contribution from passive earth pressure resistance.

15.7.5 Wet Cast Concrete Block Gravity Walls

15.7.5.1 General Considerations

Wet cast concrete block gravity retaining walls consist of a single row or multiple rows of stacked concrete blocks that resist overturning, base sliding, and shear forces through self-weight of the blocks and the retained backfill. Design of wet cast concrete block gravity retaining walls in accordance with the *AASHTO LRFD Bridge Design Specifications* and the *ODOT Geotechnical Design Manual (GDM)*.

15.7.5.2 Geotechnical Investigation

Design of wet cast concrete block gravity retaining walls requires a geotechnical investigation to explore, sample, characterize and test foundation soils and measure groundwater levels. Geotechnical investigation requirements for wall foundation design are outlined in Chapter 3.

15.7.5.3 Wall Selection Criteria

The decision to select a wet cast concrete block gravity retaining wall should be based on project specific criteria. This decision should consider the general wall design and performance requirements contained in Section 15.3 and in the Oregon Standards Specifications for Construction.

15.7.5.4 Wall Height, Footprint and Construction Easement

Wet cast concrete block gravity walls are typically designed to a maximum height of 15 ft. Wet cast concrete block gravity retaining walls typically require an additional lateral construction easement of at least 1.5*H (H = wall height) behind the wall heel to accommodate open-cut construction, drainage installation, backfill placement and compaction behind the wall. A lateral easement restriction and/or the presence of an existing roadway, structure, or utility within the construction limits could require shoring, underpinning and/or right-of-way acquisitions that can affect the construction budget and/or schedule.

15.7.5.5 Design Requirements

1. Wet cast concrete block gravity retaining walls shall include adequate subdrainage, including drainage blankets, chimney drains, and/or perforated collector pipes to relieve hydrostatic pressures and seepage forces on walls in accordance with Section 15.3.18. Additionally, provide adequate surface drainage facilities, including ditches, gutters, curbs and drop inlets, to intercept and direct water towards suitable discharge locations. Follow these guidelines:
2. Wet cast gravity retaining walls shall have backfill slopes no steeper than 1v:2h.
3. Where practical, a minimum 4.0-ft-wide horizontal bench shall be provided in front of wet cast gravity walls.
4. Assess internal stability, external stability (soil bearing resistance, settlement, eccentricity and sliding), and overall (global) slope stability for wet cast concrete block gravity retaining walls in accordance with the *AASHTO LRFD Bridge Design Specifications* and the *ODOT GDM*.
5. Active earth pressures acting on wet cast concrete block gravity retaining walls should be calculated using Coulomb earth pressure theory in accordance with AASHTO Article 3.11.5.3 and Section 15.3.10.
6. Unless otherwise noted, wet cast concrete block gravity wall analysis and design shall assume the following geotechnical properties for the wall backfill:
   - Friction angle of backfill: $\varphi = 34°$;
   - Backfill cohesion: c = 0 psf;
   - Backfill moist unit weight (γ_{wet}) = 125 pcf; and
   - Friction angle of gravel leveling pad fill: $\varphi = 34°$.

7. Internal sliding stability shall be checked at each wet cast concrete block level from the lowest block to the top of wall. Wet cast facing must have sufficient interface shear capacity to transfer lateral loads to the base of the structure without excessive wall translation, bulging, or damage. Interface sliding resistance between wet cast concrete blocks shall be calculated using the corrected wall weight based on the calculated hinge height in accordance with AASHTO Figure and Equation 11.10.6.4.4b-2. Wet cast block interface friction resistance parameters shall be based on product-specific data using NCMA Test Method SRWU-2 (*Determination of Shear Strength between Segmental Concrete Units*) in accordance with Appendix C.2 in NCMA (2002).
8. Calculate bearing resistance in accordance with AASHTO Article 10.6.3.
9. Calculate base sliding resistance (external stability) in accordance with AASHTO Article 10.6.3.4. Sliding resistance analysis shall address wet cast units bearing on gravel or on cast-in-place concrete leveling pads. The total vertical force used to calculate sliding resistance shall be based on the corrected height of the wet cast column (hinge height) calculated in accordance with AASHTO Equation 11.10.6.4.4b-2. The calculated hinge height shall not exceed the wall height.
10. In sliding, lateral resistance shall neglect any contribution from passive earth pressure resistance.

15.8 Non-Gravity (Cantilever) Soldier Pile/Lagging and Sheet Pile Walls

15.8.1 General Considerations

Non-gravity (cantilever) soldier pile/lagging and sheet pile walls are typically used in temporary construction applications, but can also be used as permanent retaining walls. These wall systems are typically limited to a maximum height (H_w) of 15 ft. or less due to inadequate stability, overstress of wall elements, and/or excessive lateral and vertical ground movements behind the wall caused by wall rotation and/or translation (H_w shown in Figure 15-9). Greater wall heights can be achieved using ground anchors or deadmen Section 15.9 and Section 15.10.

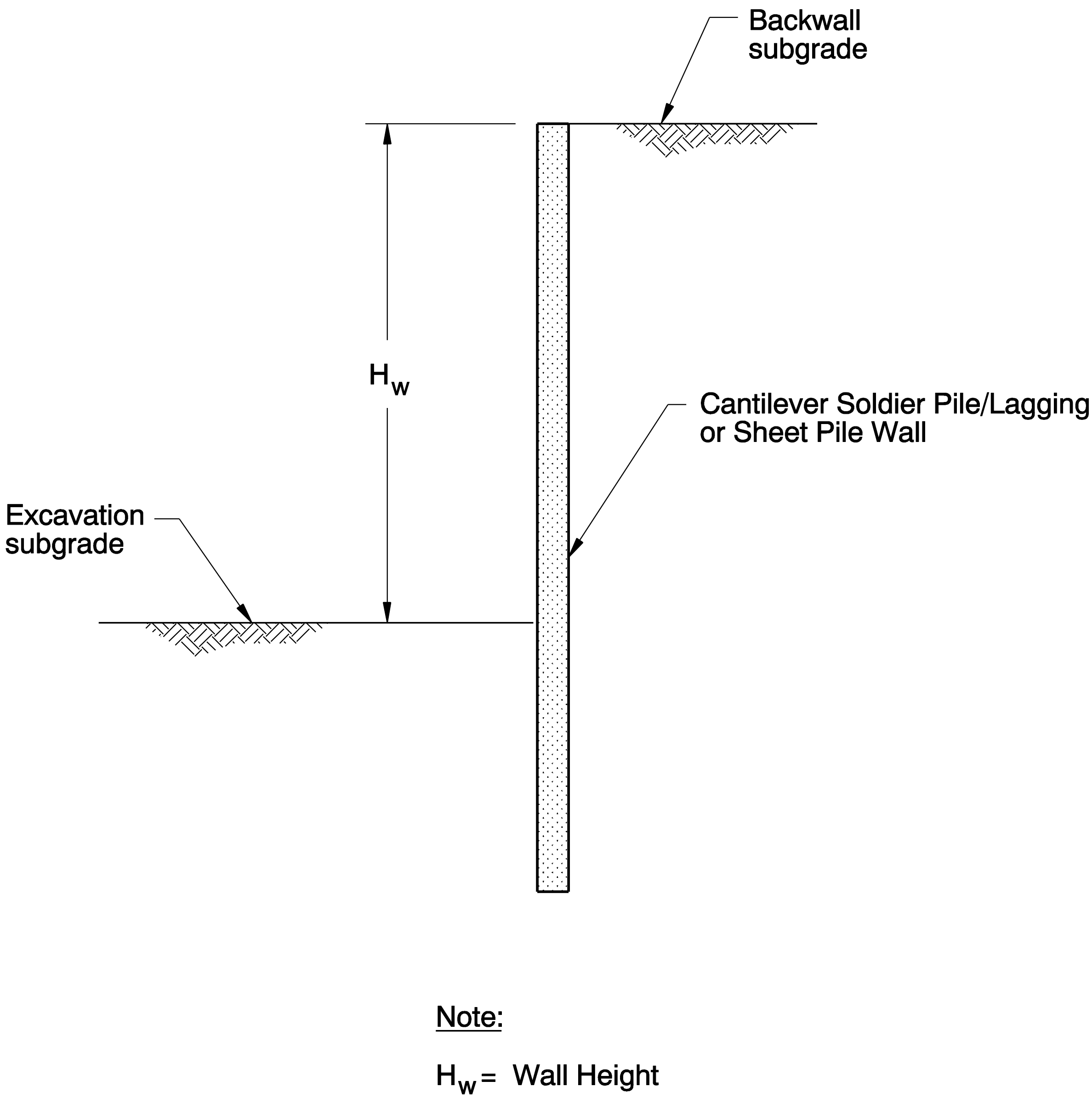


Figure 15-9. Non-Gravity (Cantilever) Soldier Pile and Sheet Pile Wall Heights

15.8.2 Design Requirements

Design of non-gravity (cantilever) soldier pile/lagging and sheet pile walls shall be in accordance with AASHTO Article 11.8.

Design of soldier pile/lagging and sheet pile walls requires a detailed geotechnical investigation to explore, sample, characterize and test the retained soils and the foundation soils along each wall. Geotechnical investigation requirements are outlined in Chapter 3. At a minimum, the geotechnical information required for wall design includes SPT N-values (depth intervals of 5 ft., or less), soil profile, unit weight, natural water content, Atterberg limit, sieve analysis, pH, resistivity, organic content, chloride and sulfate concentrations, shear strength, consolidation parameters, foreslope and backslope inclinations, and groundwater levels.

Corrosion protection for soldier piles, sheet piles, connections, and other wall components should be consistent with the design life of the wall.

15.8.3 Soldier Pile/Lagging Walls

Soldier pile walls shall be designed in accordance with AASHTO Article 11.8*, Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchorage Systems* (FHWA, 1999), the *ODOT Geotechnical Design Manual (GDM),* and the *ODOT Bridge Design and Drafting Manual (BDDM)*.

Soldier pile walls are used for both temporary and permanent applications. Soldier pile walls use wide flange steel members such as W or HP shapes. Built-up, double-channel shapes are also used as soldier piles. The spacing between soldier piles is typically 6 to 10 ft. (center-to-center). Lagging members (timber, reinforced concrete, shotcrete, and/or steel plates) span between the soldier piles to provide soil retention as wall excavation proceeds (top-down construction). Cantilever soldier pile wall heights (H_w in Figure 15-9 in excess of 15 ft. are usually feasible using ground anchors, tiebacks or deadmen anchors.

Soldier pile/lagging walls are frequently used for temporary shoring in cut applications. Impact or vibratory methods may be used to install temporary soldier piles, but installation in drill holes is typically recommended.

Permanent soldier piles (typically HP or wide flange sections) for soldier pile/lagging walls and anchored walls should be installed in drilled holes backfilled with Controlled Low Strength Material or CSLM (Section 00442), grout, and/or concrete.

15.8.4 Sheet Pile Walls

Sheet pile walls shall be designed in accordance with AASHTO Article 11.8 *, Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchorage Systems* (FHWA, 1999), the *ODOT Geotechnical Design Manual (GDM)*, the *USS Sheet Piling Design Manual* (United States Steel, 1984), and the *ODOT Bridge Design and Drafting Manual (BDDM).*

Interlocking Z-type piles are typically used for sheet pile walls. Sheet pile walls are used for both temporary and permanent applications, including excavations, bulkhead walls, cofferdams and trenches. Cantilever sheet pile walls are relatively flexible and may not be well suited for areas with strict ground movement criteria.

Cantilever sheet pile wall heights (H_w) in excess of 15 ft. can be achieved with the use of ground anchors or deadmen. Sheet pile wall embedment can be designed to reduce seepage forces and groundwater inflow into excavations and are well suited for foundation or trench excavations below the groundwater table, or as braced cofferdams below groundwater and in open water. Articulated sheet pile wall connections allow for a wide variety of irregular-shaped walls.

Sheet pile walls should not be used in areas with shallow bedrock or very dense and/or coarse soils (gravel, cobbles, or boulders), or where underground utilities, buried structures, debris or other obstructions may exist. Sheet piles are typically installed using high-energy, vibratory pile hammers that can cause excavation slope failures or create damaging ground settlements and/or vibrations in a wide area around wall construction. Design of sheet pile walls shall include the consideration of construction vibration effects of sheet pile wall installation on adjacent features, including new concrete construction, steeper cuts/fills, underground utilities, shallow foundations, roadways, bridges or other structures.

The steel sheet pile section shall be designed for the anticipated corrosion loss during the design life of the wall.

If groundwater levels differ between the front and back of the wall, design shall consider the effects of the unbalanced, hydrostatic pressure and seepage forces on wall stability, including the potential for backfill piping through interlock joints or other perforations in the sheet pile wall. Design shall consider upward seepage forces that could create a critical seepage gradient (boiling condition) in front of the wall. Boiling conditions typically develop in cohesionless soils (coarse silts and sands) subject to critical seepage gradients caused by a high water head.

15.9 Anchored Soldier Pile/Lagging and Sheet Pile Walls

15.9.1 General Considerations

Soldier pile/lagging and sheet pile walls over 15 ft. in height typically require additional lateral resistance to maintain stability and/or limit wall movements. This lateral resistance can be provided using ground anchors or buried deadmen. For highway applications, anchored sheet pile walls are typically less than 33 ft. in height due to excessive top of wall deflections, excessive sheet pile bending stresses, and high stresses at the wall-anchor connection.

Anchor terminology, minimum anchor length and embedment guidelines are shown in AASHTO Figure 11.9.1-1. Anchor spacing is controlled by many factors including anchor (or deadmen) capacity, temporary (unsupported) cut slope stability, subsurface obstructions in the anchorage zone, and the structural capacity of lagging or facing elements. Performance or proof testing shall be performed on every production anchor in accordance with the requirements in Section 15.10.

Excavation shall not proceed more than 3.0 ft. below the level of ground anchors until the ground anchors have been accepted by the Engineer.

Where backfill is placed behind an anchored wall, either above or around the unbonded length, special designs and construction specifications shall be provided to prevent anchor damage.

15.9.2 Design Requirements

1. Anchored soldier pile/lagging and sheet pile wall designs shall evaluate the anticipated combinations of lateral earth pressures, hydrostatic pressures, and seepage forces, including rapid drawdown during construction dewatering. Walls shall either include a properly designed subdrainage system to drain the retained earth or be designed for hydrostatic pressures and seepage forces in accordance with the *AASHTO LRFD Bridge Design Specifications*, FHWA (1999), and Section 15.3.18.
2. Design anchored walls, constructed top down, using unfactored apparent earth pressure distributions described in AASHTO Article 3.11.5.7.
3. Calculate maximum ordinates of apparent earth pressure for cohesionless soils using Equation 3.11.5.7.1-1 (one row of anchors) and Equation 3.11.5.7.1-2 (multiple anchor levels).
4. Analyze overall (global) slope stability and settlement of non-gravity anchored walls in accordance with the *AASHTO LRFD Bridge Design Specifications* and the requirements of the *ODOT GDM*.
5. The influence of anchored wall movements shall be evaluated for all wall systems, especially walls located near settlement-sensitive structures, including bridge foundations, wing walls, end-panels, traffic signals, pavements, utilities or developments near right-of-way boundaries.

6. Settlement of vertical wall elements can cause reduction of anchor loads and should be considered in design. A preliminary estimate of construction-phase ground settlement behind anchored walls can be made using AASHTO Figure C11.9.3.1-1, which does not include settlement caused by heavy construction surcharge loads, dewatering, foundation settlement, or poor construction practice, which must be estimated separately. Mitigation of excessive ground settlement is recommended.

7. The external and internal failure modes shall be analyzed for non-gravity anchored walls using the methodologies and procedures presented in *Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchorage Systems* (FHWA, 1999). Typical internal, external and anchorage failure modes are presented in Figure 15-10. Check stability along potential failure surfaces passing just behind ground anchors or buried deadmen, including failure surfaces that pass through the free length and/or bonded zones of ground anchors in the lower portion of the wall as shown in Figure 15-10.

8. The elevation of the ground anchor closest to the backslope ground surface should be evaluated considering the allowable cantilever deformations of the wall. The uppermost anchor depth should also be selected to minimize the potential for exceeding the passive resistance of the retained soil during anchor proof or performance load testing.

9. Seismic design of anchored soldier pile/lagging and sheet pile walls shall be in accordance with AASHTO LRFD Articles 11.9.6.

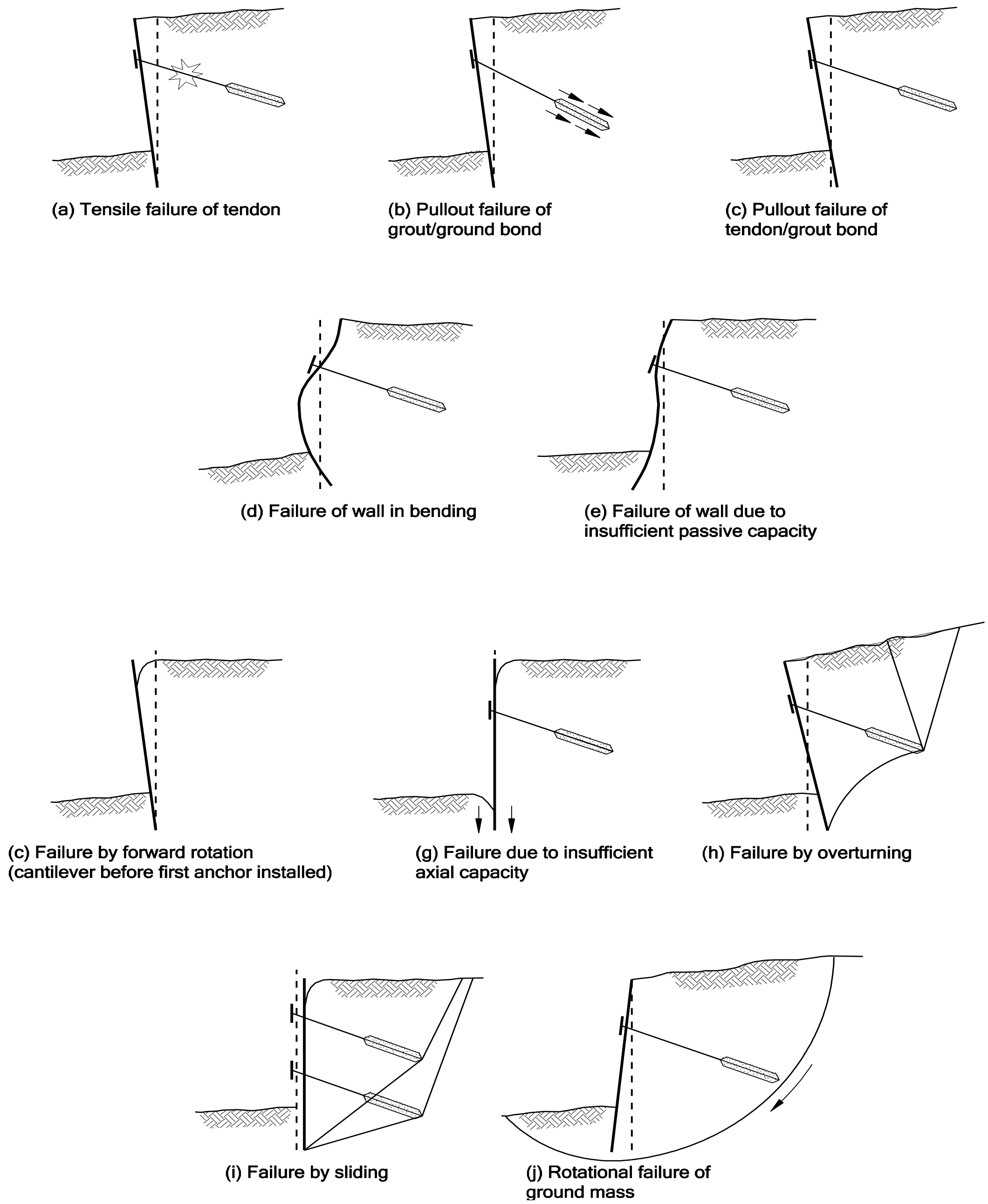


Figure 15-10. Anchored Walls: External, Internal, Global and Facing Failure Modes

15.10 Ground Anchors, Deadmen, and Tie-Rods

15.10.1 General Considerations

Ground anchors are used for permanent and temporary retaining walls and slope or landslide stabilization systems. The design of ground anchors shall be in accordance with the following:

- *AASHTO LRFD Bridge Design Specifications*;
- The *ODOT GDM*; and
- *Geotechnical Engineering Circular No. 4 - Ground Anchors and Anchorage Systems* (FHWA, 1999).

Design of ground anchors requires a detailed geotechnical investigation to explore, sample, characterize and test soil and rock conditions within and around the ground anchorage zone (ground anchors and deadmen). Additional geotechnical borings should be completed to explore soil and/or bedrock conditions within the bond zone of the anchors. The geotechnical investigation shall determine the depth, limits and failure surface geometry of any existing or potential sliding plane, slope failure, or landslide within, above, below, and/or adjacent to the anchors and deadmen.

Geotechnical investigation requirements are outlined in Chapter 3. At a minimum, the geotechnical information required for ground anchorage design includes SPT N-values (depth intervals of 5 ft., or less), soil profile, unit weight, natural water content, Atterberg limit, soil corrosively tests (e.g., pH, resistivity, organic content, chloride and sulfate concentrations), sieve analysis, shear strength, consolidation parameters, foreslope and backslope inclinations, and groundwater levels.

Conventional straight shaft, gravity-grouted ground anchors (bar tendons) are typically used. Ground anchors develop tensile (pullout) capacity from tendon-grout-ground bond stress along the anchor bond zone. Anchor capacity shall be determined based on the soil and rock conditions along the bonded anchor zone.

Highway retaining wall permanent ground anchors shall be designed for a minimum design life of 75 years. Bridge retaining wall permanent ground anchors shall be designed to have a design life consistent with the design life of the bridge—but not less than 75 years.

15.10.2 Anchor Location and Geometry

The geotechnical engineer shall define the no-load zone for anchors in accordance with *Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchorage Systems* (FHWA, 1999) and AASHTO Article 11.9. The boundaries of the no-load zone limits shall be increased to include the failure surface of any existing or potential sliding plane, slope failure, or landslide. The unbonded anchor length shall extend a minimum distance of 5 ft. or 0.2*H (H = design height shown in AASHTO Figure 11.9.1-1), whichever is greater, beyond the defined no load zone. Additionally, ground anchors should be located behind the failure surface associated with the seismic active earth pressure thrust (P_{AE}) - determined in accordance with AASHTO Article 11.9.6.

Conventional gravity-grouted ground anchors shall have a minimum overburden depth of 15 ft. at the midpoint of the anchor bond zone. Ground anchors are typically installed at angles of 15 to 30° below the horizontal. Steeper anchor inclinations (45° max.) may be required to avoid underground utilities, adjacent foundations, right-of-way restraints, or unsuitable soil or rock layers. In general gravity-grouted anchors should be installed as close to horizontal as possible, but not less than 10°.

15.10.3 Ground Anchor Design

Estimate the preliminary ground anchor bond resistance using the presumptive bond stress values in AASHTO Tables C11.9.4.2-1, -2, and -3, which address cohesive soils, cohesionless soils, and rock respectively. Designers should also consider the recommendations in *Geotechnical Engineering Circular No. 4 - Ground Anchors and Anchorage Systems* (FHWA, 1999) when selecting an anchor bond resistance. However, it is recommended that anchor bond stress be estimated from local ground anchor pullout test data, if available. The ground anchor bond stress is based on factors such as the consistency, density or strength of the soil and rock materials encountered within the ground anchorage zone, anchor overburden pressure, groundwater levels (hydrostatic pressures), and the anticipated ground anchor installation method and grouting pressure.

Lateral earth pressure loads on anchored walls shall be designed using the apparent earth pressure diagrams in AASHTO Article 3.11.5.7

15.10.4 Corrosion Protection

Protection of the metallic components of the tendon against corrosion is necessary to assure adequate long-term performance of the ground anchor. Three levels of corrosion protection are commonly specified: Class I or Class II for all permanent ground anchor tendons; and Class III (no protection) for temporary ground anchors with “nonaggressive” corrosion conditions. Class I, II and III corrosion systems are described and shown in *Geotechnical Engineering Circular No. 4 - Ground Anchors and Anchorage Systems FHWA* (1999). Select, design, and detail ground anchor corrosion protection in accordance with the requirements of FHWA (1999).

15.10.5 Anchor Load Testing

All production ground anchors shall be proof tested, except for anchors that are subject to performance tests. A minimum of 5 percent of the total number of wall anchors shall be performance tested. Required ground anchor testing and the resulting test data shall be witnessed and recorded by the Engineer.

Specify the sequence and manner of ground anchor stressing to prevent local overstress of the wale, sheet pile, and/or their connection device. Anchors shall be stressed in a uniform manner to prevent overstress.

15.10.6 Ground Anchor Proof Testing Schedule

The following loading schedule shall be used for ground anchor proof tests:

Table 15-6. Ground Anchor Proof Testing Schedule

Test Load	Hold Time
AL	1 min.
0.25 FDL	1 min.
0.50 FDL	1 min.
0.75 FDL	1 min.
1.00 FDL	**10 min.**
AL	1 min.
Lock-Off	

The maximum proof test load shall be held for at least 10 minutes with anchor movements measured and recorded at 1, 2, 3, 4, 5, 6, and 10 minutes. If anchor movements between one minute and ten minutes exceeds 0.04 inches, the maximum test load shall be held for an additional 50 minutes. If the load hold time is extended, anchor movements shall be measured and recorded at 15, 20, 25, 30, 45, and 60 minutes. If an anchor fails in creep, retesting will not be allowed.

A proof tested ground anchor with a 10-minute load hold is acceptable if the following criteria are met:

1. Ground anchor carries the maximum test load with less than 0.04 inches of movement between 1 and 10 minutes; and
2. Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the tendon-unbonded length.

A proof tested ground anchor with a 60-minute load hold is acceptable if the following criteria are met:

1. Ground anchor carries the maximum test load with a creep rate that does not exceed 0.08 inches/log cycle of time and is a linear or decreasing creep rate.
2. Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the tendon-unbonded length.

15.10.7 Ground Anchor Performance Testing Schedule

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

Table 15-7. Test Load Cycle

1	**2**	**3**	**4**	**5**
AL	AL	AL	AL	AL
0.25FDL	0.25FDL	0.25FDL	0.25FDL	Lock-Off
	0.50FDL	0.50FDL	0.50FDL	
		0.75FDL	0.75FDL	
			1.00FDL	

The anchor load shall be raised from one load increment to another immediately after a deflection reading. The maximum test load (4th load cycle) in a performance test shall be held for ten minutes. If the anchor movement between one minute and ten minutes exceeds 0.04 in., the maximum test load shall be held for an additionally 50 minutes. If the load hold is extended, the anchor movement shall be recorded at 20, 30, 40, 50, and 60 minutes. If an anchor fails in creep, retesting will not be allowed.

A performance tested ground anchor with a 10-minute load hold is acceptable if the following criteria are met:

1. Ground anchor carries the maximum test load with less than 0.04 inches of movement between 1 and 10 minutes; and
2. Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the tendon-unbonded length.

15.10.8 Deadmen or Anchor Blocks

Design deadmen or anchor blocks using passive earth pressure resistance and active earth pressure loads in accordance with the AASHTO LRFD Bridge Design Specifications, *Foundations and Earth Structures, NAVFAC DM7.02* (U.S. Navy, 1986), the *USS Sheet Piling Design Manual* (United States Steel, 1984), and the requirements of the *ODOT GDM*. The deadmen location shall have sufficient embedment within the passive earth pressure zone, beyond the wall active earth pressure zone, as described in Section 4, Figures 20 and 21 in *NAVFAC DM7.02* (U.S. Navy, 1986). Figures 20 and 21 have been reproduced as Figure 15-11 and Figure 15-12, respectively.

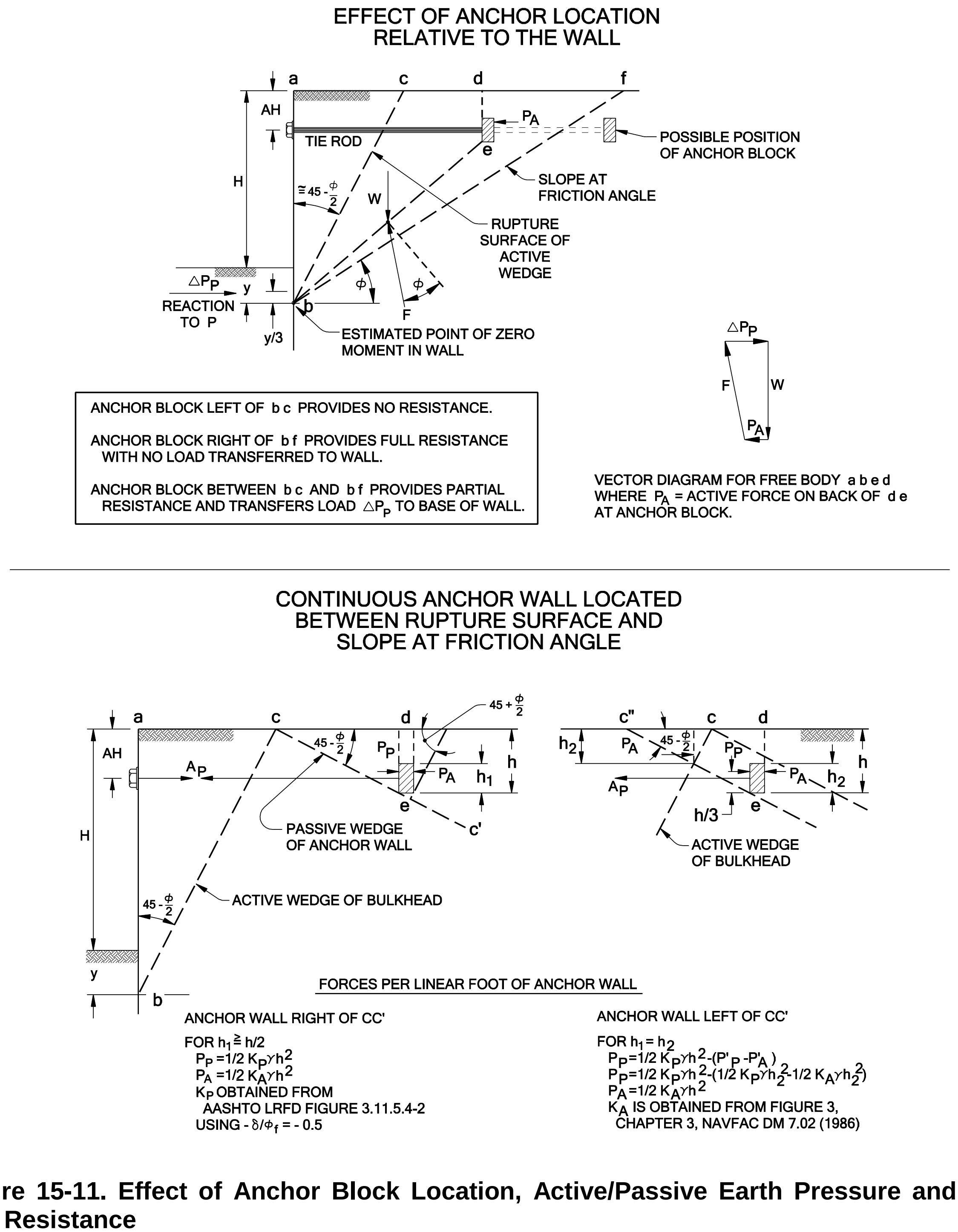
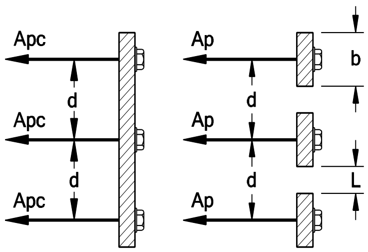


Figure 15-11. Effect of Anchor Block Location, Active/Passive Earth Pressure and Tie-Rod Resistance

EFFECT OF DEPTH AND SPACING OF ANCHOR BLOCKS



Section A-A Continuous Wall

Section A-A Individual Anchors

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

1. CONTINUOUS WALL:
   ULTIMATE $A_{pc}/d = P_P - P_A$ WHERE A_{pc}/d IS ANCHOR RESISTANCE AND P_P, P_A TAKEN PER LINEAL FOOT OF WALL.
2. INDIVIDUAL ANCHORS:
   IF $d > b + h$, ULTIMATE $A_p = b(P_P - P_A) + 2P_0 \tan\phi$, WHERE P_0 = RESULTANT FORCE OF SOIL AT REST ON VERTICAL AREA cde OR C" de.
   IF $d = h+b$, A_p/d IS 70% OF A_{pc}/d FOR CONTINUOUS WALL.
   L FOR THIS CONDITION IS L' AND L' = h.
   IF $d < h + b$, $A_p/d + A_{pc}/d = A_{pc}/d - \frac{L'}{L}(0.3\,A_{pc}/d)$, L' = h.

ANCHOR RESISTANCE FOR $h_1 < \frac{h}{2}$
   ULTIMATE A_p/d OR A_{pc}/d EQUALS BEARING CAPACITY OF STRIP FOOTING OF WIDTH h_1 AND SURCHARGE LOAD $\gamma(h - \frac{h}{2}1)$, SEE FIGURE 1, CHAPTER 4, NAVFAC DM 7.02 (1986)
   USE FRICTION ANGLE ϕ': WHERE $\tan\phi' = 0.6 \tan\phi$.

GENERAL REQUIREMENTS:

1. ALLOWABLE VALUE OF A_p AND A_{pc} = ULTIMATE VALUE /2, FACTOR OF SAFETY OF 2 AGAINST FAILURE.
2. VALUES OF K_A AND K_p ARE FOR COHESIONLESS MATERIALS. IF BACKFILL HAS BOTH ϕ AND c STRENGTHS, COMPUTE ACTIVE AND PASSIVE FORCES ACCORDING TO FIGURES 7 AND 9, Chapter 3, NAVFAC DM7.02 (1986). FINE GRAINED SOILS OF MEDIUM TO HIGH PLASTICITY SHOULD NOT BE USED AT THE ANCHORAGE.
3. SOILS WITHIN PASSIVE WEDGE OF ANCHORAGE SHALL BE COMPACTED TO AT LEAST 100 PERCENT OF RELATIVE MAXIMUM DENSITY PER AASHTO T99.
4. TIE ROD IS DESIGNED FOR ALLOWABLE A_p OF A_{pc}. TIE ROD CONNECTIONS TO WALL AND ANCHORAGE ARE DESIGNED FOR 1.2 (ALLOWABLE A_p OF A_{pc}).
5. TIE ROD CONNECTION TO ANCHORAGE IS MADE AT THE LOCATION OF THE RESULTANT EARTH PRESSURES ACTING ON THE VERTICAL FACE OF THE ANCHORAGE.

Figure 15-12. Effect of Anchor Block Spacing on Tie-Rod Resistance, Continuous and Individual Anchor Blocks

15.10.9 Tie-Rods

Tie-rods shall be designed in accordance with the *AASHTO LRFD Bridge Design Specifications*, *Foundations and Earth Structures, NAVFAC DM7.02* (U.S. Navy, 1986), the *USS Sheet Piling Design Manual* (United States Steel, 1984), and the requirements of the *ODOT GDM*.

Anchored sheet pile wall failures have occurred in the tie-rod as a result of damage from excessive differential settlement along the tie-rod, especially at the connection to the wall face. The tie-rod shall be isolated from the adverse effects of excessive settlement of the wall and/or backfill, including excessive bending, shear or tension in the tie-rod. Perform ground improvement to reduce post-construction foundation settlement to reduce settlement magnitudes if isolation of the tie-rod is not feasible.

Specify the sequence of tie-rod stressing to prevent local overstress of the wale, sheet pile, and/or their connection device. Corrosion protection of the tie-rod, wale and their connection device is necessary to assure adequate long-term wall performance.

15.11 Soil Nail Walls

15.11.1 General Considerations

Soil nail walls consist of passive reinforcement of the ground behind an excavation face by drilling and installing closely spaced rows of grouted steel bars (i.e., soil nails). The soil nails are subsequently covered with a reinforced-shotcrete layer (temporary facing) used to stabilize the exposed excavation face, support the subdrainage system (i.e., composite strip drain, collector and drainage pipes), and distribute the soil nail bearing plate load over a larger area. A permanent facing layer, meeting both structural and aesthetic requirements, is constructed directly on the temporary facing.

The principal components of a typical soil nail wall system are presented in Figure 4.1 of *Geotechnical Engineering Circular No. 7 - Soil Nail Walls* (FHWA, 2003). Soil nail walls are typically used to stabilize excavations where top-down construction, without the effects of drilling or pile installation (impact hammer or vibratory methods), is a significant advantage compared to other retaining wall systems.

Conventional soil nail wall systems are best suited for sites with dense to very dense, granular soil with some apparent cohesion (sands and gravels), stiff to hard, fine-grained soil (silts and clays) of relatively low plasticity (PI<15), or weak, weathered massive rock with no adversely-oriented planes of weakness. Soil nail wall construction requires that open excavations stand unsupported long enough to allow soil nail drilling and grouting, subdrainage installation, reinforcement, and temporary shotcrete placement.

Design of soil nail wall systems requires a detailed geotechnical investigation to explore, sample, characterize and test soil and rock conditions within and around the soil nail reinforced zone behind each wall. The geotechnical investigation shall determine the depth, limits and failure surface geometry of any existing or potential shear failure surface, slope failure, or landslide within or near the soil nail reinforced zone.

Geotechnical investigation requirements are outlined in Chapter 3. At a minimum, the geotechnical information required for soil nail design includes SPT N-values (depth intervals of 5ft, or less), soil profile, groundwater levels, unit weight, natural water content, Atterberg limit, soil electrochemical properties (e.g., pH, resistivity, organic content, chloride and sulfate concentrations), sieve analysis, shear strength, consolidation parameters, foreslope and backslope inclinations, and groundwater levels. Additionally, shallow test pit(s) should be advanced along the line of the wall face to evaluate excavation stability and stand-up time for temporary excavations required for soil nail wall construction. The test pits shall remain open for at least 24 hours and shall be monitored for sloughing, caving, and groundwater seepage. The depth of the test pits shall be at least twice the anticipated vertical nail spacing with a trench bottom length of at least 1.5 times the trench excavation depth.

15.11.2 Wall Footprint and Soil Nail Easement

The soil nail design length, spacing and inclination shall be based on site-specific soil and rock conditions in the soil nail reinforced zone, geometric constraints, and stability requirements. Soil nails shall be at least 12-ft in length, or 60 percent of the wall height, whichever is greater. Uniform soil nail lengths are typically used when back wall deformations are not a concern for the project, such as when soil nails are supported in competent ground and/or structures are not present within the zone of influence behind the wall. Wall deformations can be effectively controlled by using longer soil nails in the upper portions of the wall. Preliminary soil nail design typically assumes a minimum soil nail length of 70 percent of the wall height, which is frequently increased due to factors such as wall heights greater than 33 ft., large surcharge loads, overall (global) stability, seismic loads, and/or strict wall deformation requirements.

The horizontal and vertical spacing of soil nails are typically the same: between 4 and 6½ft for conventional drilled soil nail wall systems. The maximum soil nail spacing meeting design requirements shall be used to improve wall constructability. Soil 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. Soil nail rows should be linear to the greatest extent possible—so each individual nail location elevation can be easily interpolated from a reference nail(s). Nails along the top row shall have at least 1 foot of soil cover over the nail drill hole during installation. Soil nails are installed at angles of 10–30 degrees below the horizontal. To prevent voids in the grout, soil nails shall not be installed at inclination less than 10 degrees. Steeper anchor inclinations may be required to avoid underground utilities, adjacent foundations, right-of-way restraints, or unsuitable soil or rock layers.

The soil nail wall face batter typically varies between 0 and 10 degrees.

15.11.3 Design Requirements

1. The *AASHTO LRFD Bridge Design Specifications* do not currently provide design standards for soil nail walls. Until an AASHTO design standard is available, it is recommended that soil nail walls be designed using the methodology in *Geotechnical Engineering Circular No. 7 – Soil Nail Walls* (FHWA, 2003).
2. The external, internal, and facing connection failure modes shall be analyzed for soil nail walls using the methodologies and procedures presented in Sections 5.1 through 5.6 of *Geotechnical Engineering Circular No. 7 – Soil Nail Walls* (FHWA, 2003). Horizontal and vertical soil nail wall deformations (static and seismic) shall be checked using the methods provided in Section 5.7 of the same reference.
3. The soil nail wall system must be safe against all potential failure modes. Typical external, internal and facing failure modes presented in Figure 5.3 of Section 5.9 (FHWA, 2003) have been reproduced as Figure 15-13.
4. There is no standard laboratory strength testing procedure to accurately measure the bond resistance of a grouted soil nail. Nominal (ultimate) soil nail bond stress values are typically estimated using the values presented in Table 3.1 in *Geotechnical Engineering Circular No. 7*

– *Soil Nail Walls* (FHWA, 2003). Given the uncertainties in accurately estimating soil nail bond strength, it is recommended that pre-production soil nail tests (verification tests) be required to verify the bond strengths included in the construction specifications.

5. Highway Retaining Wall permanent soil nail walls are designed to have a minimum Design Life of 75 years. Bridge Retaining Wall permanent soil nail walls shall be designed to have a minimum design life consistent with the bridge, but not less than 75 years.

6. Design soil nail walls for seismic design forces in accordance with *AASHTO LRFD, Geotechnical Engineering Circular No. 7 – Soil Nail Walls* (FHWA, 2003), Section 15.3.13, and the following recommendations:

   - Soil nail wall analyses (Extreme I limit state loads) shall confirm the wall can resist forces due to static and seismic earth pressures (including water head load if applicable) and inertial forces of the wall without structural failure or excessive sliding, movement, or rotation of the soil nail wall.

   - Stability analyses of soil nail walls shall be designed using the most current versions of Gold Nail (version 3.11), the Caltrans SnailzWin 3.10 (version 6.01) or Snail programs.

   - External and compound stability analyses of soil nail walls shall be performed using a state-of-the-practice slope stability computer program, such as the most current versions of Slope/W® (Geo-Slope International), and ReSSA® (ADAMA Engineering, Inc.) as described in Section 15.3.13 and AASHTO LRFD.

   - The design horizontal acceleration coefficient (k_h) of soil nail walls shall be in accordance with AASHTO Article 11.6.5.

7. The soil nail wall system must be safe against any potential temporary critical wall stability condition that may exist during construction. For example, the soil nail wall must be safe against all modes of failure from temporary, unreinforced, near-vertical excavations required to install additional nailed lifts. The factor of safety for temporary soil nail wall stability, including internal, external, and compound\global stability, shall not be less than 1.3.

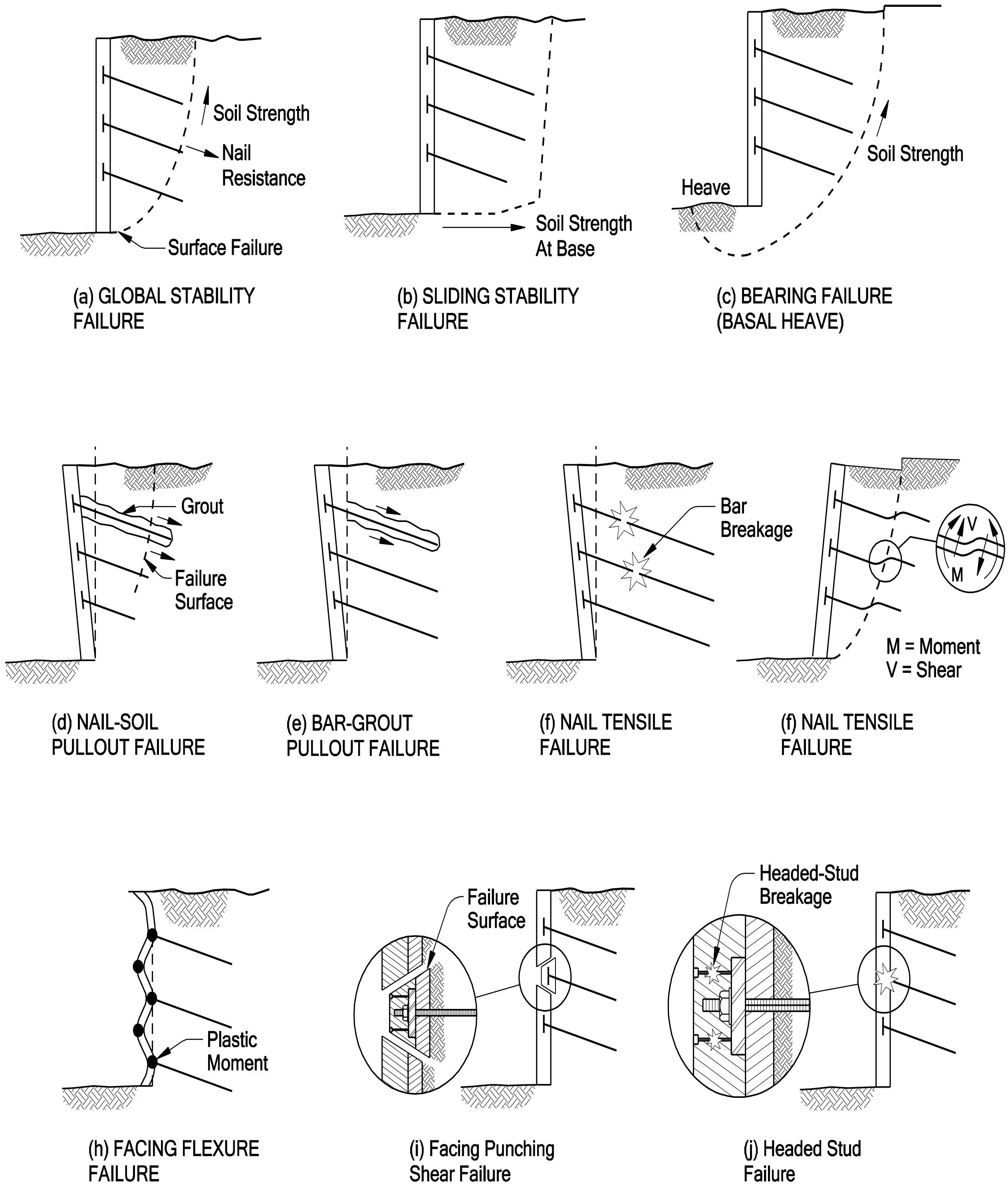


Figure 15-13. Soil Nail Walls: External, Internal, Global, and Facing Failure Mode

15.11.4 Facing

A permanent wall facing is required for all permanent soil nail walls. In addition to meeting aesthetic requirements and providing adequate corrosion protect to the steel soil nail, design facing for all facing connection failure modes, including but not limited to those indicated in Figure 15-13.

The soil nail wall face batter typically varies between 0 and 10 degrees.

15.11.5 Corrosion Protection

Corrosion protection is required for all permanent soil nail wall systems. Protection of the metallic components of the soil nail wall against corrosion after construction is necessary to assure adequate long-term wall durability.

Two levels of corrosion protection (*Class I* and *Class II*) are commonly specified for soil nail walls depending on the wall design life (e.g., temporary or permanent wall system), structure criticality, and electrochemical properties of the site soils. *Class I* corrosion protection consists of either an epoxy-coated bar or a grout-coated bar inside PVC sheathing; both encapsulated in an outer grout layer ("double-corrosion" protection system). *Class I* corrosion protection is required for all permanent soil nail walls. *Class II* corrosion protection consists of bare bar encapsulated in an outer grout layer – typically used for non-permanent (temporary), soil nail walls.

Class I and *Class II* corrosion systems are described in detail and shown in *Geotechnical Engineering Circular No. 7 – Soil Nail Walls* (FHWA, 2003).

The level of corrosion protection required should be determined on a project-specific basis based on factors such as wall design life, structure criticality and the electrochemical properties of the supporting soil and rock materials. Criteria for classification of the supporting soil and rock materials as "aggressive" or "non-aggressive" are provided in Table 3.9 of *Geotechnical Engineering Circular No. 7 – Soil Nail Walls* (FHWA, 2003). This classification shall be used in *Appendix C.5* of FHWA (2003) to determine if *Class I* or *Class II* corrosion protection is required.

15.11.6 Load Testing

Soil nails are field tested to verify nail design loads can be supported without excessive movement and with an adequate margin of safety. Perform both verification and proof testing of designated test nails.

Perform preproduction verification tests on sacrificial test nails at locations shown on the plans and/or described in the Special Provisions. Preproduction verification testing shall be performed prior to installation of production soil nails to verify the Contractor's installation methods, proposed drill hole diameter and pullout resistance. Perform a minimum of two verification tests in each principal soil or rock unit providing soil nail support and for each different drilling/grouting method proposed to be used, at each wall location. Verification test soil nails will be sacrificial and not incorporated as production nails. Creep tests are performed as part of the verification tests.

Verification test nails shall have both bonded and unbonded lengths. Prior to testing only the bonded length of the test nail shall be grouted. The unbonded length of the test soil nail shall be at least 3.0 ft. The bonded length of the test nail shall be determined based on the production nail bar grade and size such that the allowable bar structural load is not exceeded during testing and shall not be less than 10 feet. Verification test nails shall be incrementally loaded to a maximum test load of 200 percent of the Design Load (DL) in accordance with the loading schedule in Section 15.11.7. The soil nail bar shall be proportioned such that the maximum stress at 200 percent of the design load (2.00 DL) does not exceed 80 percent of the yield strength of the steel.

Soil nail capacity is sensitive to the Contractor's drilling, installation, and grouting methods and changes in soil and rock support conditions. Therefore, additional soil nail verification testing is required at any time the Contractor changes construction equipment or methods, or if there is a change in soil or rock support conditions.

15.11.7 Soil Nail Verification Test Schedule

Perform verification tests on soil nails at locations selected by the Engineer. Verification tests on soil nails shall be installed using the same equipment, methods, nail inclinations, nail lengths, and hole diameters as the production nails. Required soil nail test data shall be recorded by the Engineer - including the bonded and unbonded lengths for each tested soil nail.

The following schedule shall be used for verification tests:

Table 15-8. Soil Nail Verification Tests

Test Load	Hold Time
AL	1 Min.
0.25 DL	10 Min.
0.50 DL	10 Min.
0.75 DL	10 Min.
1.00 DL	10 Min.
1.25 DL	10 Min.
1.50 DL	**60 Min.**
1.75 DL	10 Min.
2.00 DL	**10 Min.**
AL	1 Min.

The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load (DL). Dial gauges should be set to "zero" after the alignment load has been applied. The test load shall be applied in increments of 25 percent of the design load to 2.00 DL. Measurements of soil nail movement shall be obtained at each load increment. The test nail movement shall be measured and recorded to the nearest 0.001 in. with respect to an independent fixed reference point. Each test load increment shall be held for at least 10 minutes. All load increments shall be maintained within 5 percent of the intended load.

The verification test soil nail shall be monitored for creep at the 1.50 DL load increment. Nail movements during the creep portion of the test shall be measured and recorded at 1, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes. If the soil nail fails in creep, retesting will not be allowed.

A verification tested soil nail with a 60-minute load hold is acceptable if the following criteria are met:

1. Soil nail carries the maximum test load with a creep rate that does not exceed 0.08 inches/log cycle of time and is a linear or decreasing creep rate; and
2. Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the tendon-unbonded length.
3. A pullout failure does not occur. Pullout failure is defined as the load as which attempts to further increase the test load result in continued pullout movement of the test nail.

15.11.8 Soil Nail Proof Test Schedule

Perform proof tests on production soil nails at locations selected by the Engineer. Successful proof testing shall be demonstrated on at least 5 percent of production soil nails in each nail row or a minimum of one per row. Required soil nail test data shall be recorded by the Engineer - including the bonded and unbonded lengths for each tested soil nail.

Production proof test nails shall have both bonded and temporary unbonded lengths. Prior to testing, only the bonded length of the test nail shall be grouted. The temporary unbonded length of the test nail during proof testing shall be at least 3 ft. The bonded length of the test nail during proof testing shall be at least 10 ft., except production proof test nails shorter than 12 ft may be constructed with less than the minimum 10 ft bond length.

Proof tests shall be performed by incrementally loading the proof test nail to a maximum test load of 150 percent of the design load (DL) in accordance with the schedule below. The test nail movement shall be measured and recorded to the nearest 0.001 in. with respect to an independent fixed reference point.

The following shall be used for proof tests:

Table 15-9. Soil Nail Proof Test Schedule

Test Load	Hold Time
AL	1 Min.
0.25 DL	5 Min.
0.50 DL	5 Min.
0.75 DL	5 Min.
1.00 DL	5 Min.
1.25 DL	5 Min.
1.50 DL	10 Min.

The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load (DL). Dial gauges should be set to "zero" after the alignment load has been applied. The test load shall be applied in increments of 25 percent of the design load to 1.50 DL. Measurements of soil nail movement shall be obtained at each load increment. The maximum load in the proof test shall be held for at least 10 minutes. All load increments shall be maintained within 5 percent of the intended load.

Depending on the following performance criteria, either 10- or 60-minute creep tests shall be performed at the maximum test load (1.50 DL). The creep period shall start as soon as the maximum test load is applied and the nail movement shall be measured and recorded at 1, 2, 3, 5, 6, and 10 minutes. If nail movement exceeds 0.04 in. between 1- and 10-minute readings, the maximum test load shall be maintained an additional 50 minutes and movements shall be recorded at 20, 30, 50, and 60 minutes. If a soil nail fails in creep, retesting will not be allowed.

A proof tested nail is acceptable if the following criteria are met:

- The nail carries the maximum load with less than 0.04 inches of movement between 1 and 10 minutes, unless the load hold time extended to 60 minutes in which case the nail would be acceptable if the creep rate does not exceed 0,08 inches per log cycle of time;

- The total movement at the maximum load exceeded 80 percent of the theoretical elastic elongation of the non-bonded length; and
- The creep rate is not increasing with time during the hold period.
- A pullout failure does not occur. Pullout failure is defined as the load as which attempts to further increase the test load result in continued pullout movement of the test nail.

15.12 Tangent/Secant Pile Walls

Tangent/secant pile walls shall be designed as non-gravity (cantilever) or anchored retaining walls in accordance with ODOT Section 15.8, except as noted in this section. Selection, design, and construction criteria for tangent/secant pile walls are provided in *Geotechnical Engineering Circular No. 2 - Earth Retaining Systems*, FHWA (1997) and *Geotechnical Engineering Circular No. 8 - Design and Construction of Continuous Flight Augers*, FHWA (2007).

Tangent/secant pile walls consist of rows of cast-in-place, reinforced concrete drilled shafts (typically 24- to 48-in. diameter) that are tangentially touching (tangent piles) or overlapping (secant piles) to create a continuous retaining wall. Greater wall heights can be achieved using ground anchors (tiebacks). Tangent/secant pile walls are typically used in permanent excavation applications. Tangent/secant pile wall construction is a relatively noise-free and vibration-free alternative to sheet pile and soldier pile wall installations.

Tangent/secant pile walls with ground anchors are very stiff wall systems that can reduce ground movements to a strict tolerance. Anchored walls have been successfully used for underpinning building foundations and other settlement-sensitive structures near excavations. Tangent/secant pile walls also create an effective groundwater seepage barrier and have cofferdam applications. Walls shall either be designed to drain the retained earth or be designed for hydrostatic pressures in accordance with AASHTO Articles 3.11.3 and 11.6.6 of the *AASHTO LRFD Bridge Design Specifications*.

15.13 Slurry/Diaphragm Walls

Slurry/diaphragm walls shall be designed as non-gravity (cantilever) or anchored retaining walls as indicated in Section 15.9, except as noted in this section. Selection, design, and construction criteria for slurry/diaphragm walls are provided in *Geotechnical Engineering Circular No. 2 - Earth Retaining Systems*, FHWA (1997).

Slurry/diaphragm walls are typically used for permanent applications and consist of cast-in-place, reinforced concrete panels constructed in a trench-using mineral or polymer slurry to maintain trench stability. The walls are well suited for sites where flexible sheet pile walls would have potential installation problems due to high penetration resistance in very dense and/or coarse soils (gravel, cobbles, or boulders). Slurry/diaphragm walls have a very high section modulus and are well suited for applications with strict wall movement criteria. The walls also provide a highly effective seepage barrier that allows for rapid excavation dewatering and long-term, watertight construction. Other advantages include relatively high vertical and lateral load capacities and minimal construction vibration effects. New trench cutting equipment has headroom requirements of less than 20 ft.

Slurry/diaphragm walls should include a properly designed subdrainage system or be designed as a watertight structure with hydrostatic pressures (AASHTO Articles 3.11.3 and 11.6.6 of the *AASHTO LRFD Bridge Design Specifications*). Since slurry/diaphragm walls can have a very high section modulus, consider wall movement magnitudes to reach active earth pressures conditions (Table C3.11.1-1 in AASHTO Article 3.11.1). Design for at-rest earth pressures (AASHTO Article 3.11.5.2) if wall movement is restrained.

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Appendix 15-A:

General Requirements for Proprietary Retaining Wall Systems

15-A.1 Overview

Proprietary retaining wall systems shall be designed and supported by the wall Manufacturer in accordance with the requirements of ODOT project plans, ODOT specifications, preapproved Manufacturer details, and this Manual (see Section 15.2.1.2 for the definition of proprietary retaining wall system).

Proprietary retaining wall systems shall be preapproved by the Agency before being considered for use on Agency projects. Note that preapproval of a Manufacturer's retaining wall system does not imply preapproval of any other system from the Manufacturer. The Agency preapproves specific retaining wall systems and does not approve the Manufacturer.

Preapproval shall not be regarded as project specific design acceptance. The Manufacturer must also submit a project specific design according to Agency requirements. Submittal requirements for project specific designs are specified in the contract documents.

Submit proprietary retaining wall systems for preapproval according to Appendix 15-B.

Preapproval will be based on an extensive technical audit by the Agency. This review examines system theory, Manufacturer design methods, details, materials, QA/QC plan, and construction methods. Constructability, Manufacturer support, and system performance on previous projects will also be considered.

The example calculations required in Appendix 15-B.3 are intended to demonstrate that the proposed proprietary retaining wall system is capable of satisfying loading requirements for the proposed uses, and that the Manufacturer's design methods are in accordance with Agency requirements. The detail drawings required in Appendix 15-DB. are intended to demonstrate that manufacturer plans and details adequately address typical wall construction requirements.

Proprietary retaining wall systems are pre-approved by category. There are three retaining wall preapproval categories, corresponding to the three retaining wall definitions see Section 15.2.1.1:

- Bridge retaining walls;
- Highway retaining walls; and
- Minor retaining walls

The Conditions of Preapproval and Preapproved Manufacturer Details for specific preapproved proprietary retaining wall systems may limit the use of preapproved proprietary retaining wall systems. See Appendix 15-D for specific Conditions of Preapproval and Preapproved Manufacturer Details for each preapproved proprietary retaining wall system.

15-A.2 Design and Construction Requirements:

Proprietary retaining wall systems shall meet the requirements of *AASHTO LRFD Bridge Design Specifications*, as modified by the *ODOT GDM*, and the *Oregon Standard Specifications for Construction*.

15-A.3 Responsibilities:

This section establishes responsibilities for both ODOT and the proprietary retaining wall system Manufacturer.

15-A.3.1 Agency Responsibilities:

15-A.3.1.1 Agency Standards and Practices Responsibilities

- ODOT Geotechnical Design Manual.
- Oregon Standard Specifications for Construction.
- Preapproval of Proprietary Retaining Wall systems.

15-A.3.1.2 Agency Design Responsibilities

- Select proprietary retaining wall systems that are appropriate for the project, and list them in the Project Special Provisions.
- Perform retaining wall overall (global) stability analysis (including preliminary compound stability analysis) and provide minimum requirements for overall and compound stability (i.e. minimum dimensions for overall and compound stability) in the Special Provisions. For MSE walls, provide the minimum soil reinforcement length from the Special Provisions.
- Perform preliminary external stability analysis (sliding, eccentricity, bearing), and provide minimum requirements for external stability (i.e., minimum dimensions for external stability) in the project plans and/or special provisions.
- Perform retaining wall settlement analysis for the Service Limit State and provide nominal and factored settlement limited bearing resistance and settlement estimates in the project plans and/or special provisions.
- Perform retaining wall bearing resistance analysis for the Strength and Extreme Event Limit States and provide nominal and factored bearing resistances in the project plans and/or special provisions.
- Perform retaining wall drainage analysis and provide drainage design in the project plans and/or special provisions.
- Perform liquefaction analysis and provide liquefaction mitigation design for the retaining wall in the project plans and/or special provisions when applicable.
- Provide scour prevention design in the project plans and/or specification when applicable.
- Provide geotechnical properties and design values needed by the Manufacturer for design of the proprietary retaining wall system from the Special Provisions.
- Provide minimum required embedment depths for the retaining wall in the project plans.

- Provide special notes in the project plans and/or special provisions as applicable.
- Provide geotechnical /foundation data sheet in project plans.
- Provide a Final Geotechnical Report for the retaining wall to the Project Manager for use by the Manufacturer of the proprietary retaining wall system.
- Select acceptable preapproved proprietary retaining wall systems and list them in the project special provisions as “Options” or “Alternates.”
- Provide a wall-loading diagram or loading table with sufficient detail for the Manufacturer of the proprietary retaining wall system to design the wall.
- Prepare control plans see Section 15.2.8.1.
- Prepare Special Provisions.

15-A.3.1.3 Agency Construction Assistance Responsibilities

- Review Manufacturer working drawings and calculations for conformance with contract documents, Conditions of Preapproval in Appendix 15-D, and preapproved Manufacturer details in Appendix 15-D and the GDM. Also verify that all previous design assumptions are still valid for the specific proprietary retaining wall system proposed by the contractor.
- Construction consultation.

15-A.3.2 Proprietary Retaining Wall System Manufacturer Responsibilities:

- Obtain preapproval for the proprietary retaining wall system from the ODOT Retaining Structures Program before bidding on projects.
- Submit annual system updates (optional) (see Appendix 15-A.7).
- Design the proprietary retaining wall system to satisfy internal stability, external stability (bearing, sliding, and overturning), and compound stability under all applicable limit states. The design shall be in accordance with the project plans and specifications, the *ODOT GDM*, the Conditions of Preapproval for the specific proprietary retaining wall system in Appendix 15-D, and the preapproved Manufacturer details in Appendix 15-D.
- Submit stamped working drawings and stamped calculations, according to the contract documents, for Agency review.
- Provide proprietary product (materials).
- Provide technical assistance in accordance with the contract documents.
- Satisfy all other applicable Agency requirements.

15-A.4 Preapproval Process and Submittal Requirements for Proprietary Retaining Wall Systems

The conditions of Preapproval for each preapproved proprietary retaining wall system are included in Appendix 15-D. Conditions of Preapproval are developed during the detailed technical audit of proprietary retaining wall systems.

15-A.5 Responsibility for Preapproval;

Preapproval of proprietary retaining wall systems is the responsibility of the ODOT Retaining Structures Program in special cases proprietary retaining wall systems may also be preapproved on a project specific basis by the local Region Tech Center. All project specific preapprovals of proprietary retaining wall systems shall be in accordance with the ODOT GDM and must be reported to the ODOT Retaining Structures Program.

15-A.6 Conditions of Preapproval for Specific Proprietary Retaining Wall Systems

The Conditions of Preapproval include, but are not limited to:

- Preapproved Manufacture detail drawings shown in Appendix 15-D;
  - See the Conditions of Preapproval for Agency comments and requirements regarding the proprietary retaining wall systems;
  - Details not shown on the preapproved Manufacturer detail drawings are not considered preapproved;
- General comments about the system;
- Categories preapproved (Bridge, Highway, Minor);
- Preapproval effective date;
- Preapproval maximum wall height; and
- Specific requirements intended to point out and correct Manufacturer practices that do not meet ODOT requirements. The ODOT EOR for the retaining wall system., and Agency personnel performing construction inspection and other Agency QA/QC functions shall consider the Conditions of Preapproval to be mandatory requirements.

15-A.7 System Updates (Optional)

Manufacturers may submit annual updates for retaining wall systems during January starting 2013. System updates are required to change the limits of Agency retaining wall systems preapproval.

System updates shall provide the following information:

- Manufacturer name;
- Retaining Wall System Name(s);
- Contact Person name and signature;
- Contact phone;
- Contact Address;
- Contact email;
- Description of proposed changes to reapproved design and construction method, or confirmation that preapproved design and construction methods have not changed; and

- Description of changes to formulation of the preapproved system, or confirmation that formulation of the preapproved system has not changed.

Send the annual update to:

Oregon Department of Transportation
Geo-Environmental Section
Engineering and Asset Management Unit
4040 Fairview Industrial Dr. SE, MS 6
Salem, OR 97302
Phone: 503.986.3252 Fax: 503.986.3249

15-A.8 Disqualification and Requalification

Disqualification

The Retaining Structures Program reserves the right to disqualify proprietary retaining wall systems (remove from “preapproved” status) for:

- Non-conformance with preapproved design and construction methods;
- Non-conformance with Agency requirements; and
- Documented history of poor field performance.

Requalification

The Retaining Structures Program will re-evaluate a product that has been disqualified (removed from “preapproved” status) only after submission of a formal request along with acceptable evidence that the problems causing the disqualification have been resolved.

Appendix 15-B

Preapproval Process and Submittal Requirements for Proprietary Retaining Wall Systems:

As noted in Section 15.1.1, "Preapproved" status of proprietary retaining wall systems that were preapproved prior to January 5, 2009 will expire on July 1, 2010. Proprietary retaining wall system preapprovals obtained prior to January 5, 2009 were based on the *ODOT Retaining Structures Manual*, and do not meet current requirements of the ODOT GDM. Proprietary retaining wall systems must obtain "preapproved" status or "preapproved-temporary" status based on the ODOT GDM, and be listed in Appendix 15-D, to be considered on Agency projects with bid dates occurring after July 1, 2010.

To apply for ODOT GDM based preapproval of retaining wall systems that were not preapproved by the Agency prior to January 5, 2009 ("new systems"), submit an application for preapproval according to Appendix 15-B.2.1. To apply for ODOT GDM based preapproval of retaining wall systems that were preapproved by the Agency prior to January 5, 2009 ("existing systems"), submit an application for preapproval according to Appendix 15-B.2.2.

Preapproval of both "new systems" and "existing systems" will be based on a detailed system review in accordance with the ODOT GDM. Once preapproved, proprietary retaining wall systems will be listed as "preapproved" in Appendix 15-D. During system review, (after Agency acceptance of their preapproval application), "existing systems" will also be listed as "preapproved-temporary."

The "conditions of preapproval" for walls with either "preapproved" or "preapproved-temporary" status will be listed in Appendix 15-D. The "conditions of preapproval" for "existing systems" with "preapproved-temporary" status will be consistent with prior Agency preapproval.

"Preapproved-temporary" status will only remain effective until the Agency determines that an adequate number of proprietary retaining wall systems have undergone a detailed system review, and been preapproved in accordance with the ODOT GDM, at which time all proprietary retaining wall systems with temporary preapproval will be removed from the list of preapproved retaining wall systems until they obtain "preapproved" status from the agency.

15-B.1 Preapproval Process:

Step A: Manufacturer Submits Application

Conditions for acceptance of applications:

- The application for preapproval must be for a single proprietary retaining wall system. A single wall system may include only one wall type (wall types are listed in Section 15.2.2). A single MSE wall system may include only one batter, one facing type, and one facing connection type.
- Applicant must own the proprietary retaining wall system or act as the sole representative of the proprietary retaining wall system owner for the purpose of obtaining Agency preapproval. Applicant must also provide system design and support.

Applications shall be submitted according to Appendix 15-B.2.1 for "new systems", or according to Appendix 15-B.2.2 for "existing systems".

Manufacturers may submit applications to the Agency at the address shown below.

Oregon Department of Transportation
ODOT Geo Environmental
Engineering and Asset Management Unit
4040 Fairview Industrial Dry SE, MS #6
Salem, OR 97302
Phone 503-986-3252 Fax 503-986-3252

Step B: Agency Reviews Application

Written acknowledgement is sent to the applicant upon receipt of application.

Agency reviews the application, and does one of the following:

- Agency sends written notice to the Manufacturer that the application is accepted and provides any supplemental information and/or direction required for the manufacturer to prepare detailed system information for the proprietary retaining wall system described in the application.
- Agency sends written notice to the Manufacturer stating that the application has not been accepted, along with an explanation of why the application has not been accepted. The notice may request more information or clarification about the proprietary retaining wall system described in the application.

Step C: Manufacturer Submits Detailed Information

After the Agency accepts the manufacturer application the Manufacturer submit five sets of the detailed information required in Appendix 15-B.3 (for MSE walls) Appendix 15-B.4 (for prefabricated modular walls).

To help ODOT understand the functioning and performance of the technology and thereby facilitate the technical audit, applicants are urged to spend the time necessary to provide clear, complete and detailed responses. Missing or incomplete information will delay the Agency technical audit.

A response on all items that could possibly apply to the system or its elements and components, even those where evaluation procedures have not been fully established would be of interest to ODOT. Any omissions should be noted and explained.

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

Prior to beginning the technical audit (Step D), the Agency will verify completeness of the submittal. The technical audit will not be started until the submittal is complete.

Step D: Agency Performs System Technical Audit

ODOT performs a technical audit of the manufacturer submittals. Preapproval will be based on the compliance with ODOT GDM requirements. Additional system information may be required from the system manufacturer during the system technical audit if needed to complete the technical audit.

Step E: Agency Issues Findings

Based on the findings of the Agency technical audit, the wall system will be preapproved in one or more of the three categories (Bridge, Highway, Minor), or it will be rejected. Rejected systems will be provided an explanation of items warranting the finding. If preapproved, the findings will be sent to the Manufacturer and posted in Appendix 15-D.

15-B.2 Application for Preapproval of Retaining Wall System:

15-B.2.1 "New System" Application:

This option is only available for proprietary retaining wall systems that did not have Agency "preapproved" status prior to January 5, 2009. This application process places the system in the queue for full preapproval. "New systems" are not eligible for temporary approval.

Provide answers to the following questions, as applicable. Answers should follow the order of the questions, and each answer should reference the question number.

A. Applicant Identification
   1. Company name
   2. Name and title of authorized representative
   3. Street Address
   4. Email Address
   5. Phone
   6. Fax
   7. Signature and date

B. Product Identification
   1. Product or trade name (only one system per application)
   2. Description. As part of the description, identify the retaining wall system type from the list in Section 15.2.4.2, and describe the system. Indicate batter of the wall face.
   3. Indicate which of the following categories of preapproval is being requested (see Appendix 15-A General Requirements for Proprietary Retaining Wall Systems).
      a. Bridge Retaining Wall System (also indicate proposed maximum wall height).
      b. Highway Retaining Wall System (also indicate proposed maximum wall height).
      c. Minor Retaining Wall System.
   4. Indicate whether preapproval for tiered wall applications is being requested.

C. Performance Criteria and History
   1. Please write a brief history of the product's development, introduction, and acceptance to date. Where applicable, include a description of any predecessor products.
   2. Summarize any tests/evaluations already performed on the product, including the place, date, and result of testing. Attach copies of any available reports.
   3. Are there any issues other than functional performance that might be of significant interest or concern to a potential user (e.g. environmental acceptability, safety performance)?

D. Proprietary Rights

1. Does the product involve proprietary technology?

2. Is the product patented, copyrighted, or otherwise protected?

3. If proprietary or patented technology is involved, please provide a summary description of the proprietary/protected features. Also indicate the date patented and the date the patent expires.

4. If there is any specific information regarding your firm, the product, your application for preapproval, or any other matter that you wish to be treated as strictly confidential, please describe by categories or subject of confidential data how you would like the Agency to treat this data. Also, where appropriate, please describe any measures or safeguards that have been applied (or could be applied) to protect the confidentiality of the data.

E. Organizational Structure

1. Please provide a brief description of the size, organizational structure, and technical resources of your company.

F. HITEC

1. Please provide *Highway Innovative Technology Evaluation Center (HITEC) Technical Evaluation Report* for the retaining wall system, if available.

15-B.2.2 "Existing System Application":

This option is only available for proprietary retaining wall systems that had Agency "preapproved" status prior to January 5, 2009. This application process places the system in the queue for full preapproval. "Existing systems" will also be listed as "preapproved-temporary" upon Agency acceptance of their preapproval application.

Please provide answers to the following questions, as applicable. Answers should follow the order of the questions, and each answer should reference the question number.

A. Applicant Identification

1. Company name

2. Name and title of authorized representative

3. Address

4. Phone

5. Fax

6. Signature and date

B. Product Identification

1. Product or trade name (only one system per application)

2. Description. As part of the description, identify the retaining wall system type from the list in Section 15.2.4.2 and describe the system. Indicate batter of the wall face.

3. Indicate which of the following categories of preapproval is being requested (see Appendix 15-A General Requirements for Proprietary Retaining Wall Systems).

   a. Bridge Retaining Wall System (also indicate proposed maximum wall height)

b. Highway Retaining Wall System (also indicate proposed maximum wall height)

c. Minor Retaining Wall System

4. Indicate whether preapproval for tiered wall applications is being requested.

C. Acknowledgement of ODOT GDM Implementation

1. In your application for preapproval, include a statement acknowledging that proprietary retaining wall systems with "preapproved-temporary" status must meet all requirements of the ODOT Geotechnical Design Manual (GDM).

D. Performance Criteria and History

1. Please indicate the ODOT "index number" of the retaining wall system used in the ODOT Retaining Structures Manual, and date of preapproval.

2. Please indicate any changes that have been made to the retaining wall system since preapproval.

3. Please describe and explain any performance problems that have occurred with the retaining wall system.

E. Proprietary Rights

1. Does the product involve proprietary technology?

2. Is the product patented, copyrighted, or otherwise protected?

3. If proprietary or patented technology is involved, please provide a summary description of the proprietary/protected features. Also indicate the date patented and the date the patent expires.

4. If there is any specific information regarding your firm, the product, your application for preapproval, or any other matter that you wish to be treated as strictly confidential, please describe by categories or subject of confidential data how you would like the Agency to treat this data. Also, where appropriate, please describe any measures or safeguards that have been applied (or could be applied) to protect the confidentiality of the data.

F. Organizational Structure

1. Please provide a brief description of the size, organizational structure, and technical resources of your company.

G. HITEC

1. Please provide *Highway Innovative Technology Evaluation Center (HITEC) Technical Evaluation Report* for the retaining wall system, if available.

15-B.3 Submittal Requirements for Proprietary MSE Retaining Wall Systems

Instructions:

To expedite the evaluation of the MSE Retaining Wall system, applicants must furnish information as indicated in the Checklist. The Checklist items should be referenced to assure that the submittal package includes all of the listed information. The submittal package should be organized according to the numbered items in the Checklist. The completed Checklist should be included with the submitted package.

Part One:

Identify material specification designations that govern the materials that are used in furnishing the wall system elements and components. Provide product literature that describes the wall system, its elements and components and adequately addresses the checklist items. Identify precast concrete facilities that have experience with fabricating the concrete elements and components of the wall system.

1.1 Concrete Facing Unit

Yes	No	N/A	
___	___	___	(a) Standard dimensions and tolerances
___	___	___	(b) Joint sizes
___	___	___	(c) Concrete strength
___	___	___	(d) Wet cast concrete percent air (range)
___	___	___	(e) Moisture absorption (percent by weight)
___	___	___	(f) Scaling resistance
___	___	___	(g) Freeze thaw durability
___	___	___	(h) Facing unit to facing unit shear resistance
___	___	___	(i) Bearing pads (joints)
___	___	___	(j) Spacers (pins, etc.)
___	___	___	(k) Joint filter requirements: geotextile or graded granular
___	___	___	(l) Aesthetic choices (texture, relief, color, graffiti treatment)
___	___	___	(m) Other facing materials

1.2 Earth reinforcement

1.2.1 Metallic

Yes	No	N/A	
___	___	___	(a) Type identified (welded wire, steel bars, etc.)
___	___	___	(b) Ultimate and yield strength of steel
___	___	___	(c) Minimum galvanization thickness
___	___	___	(d) Corrosion resistance test data

1.2.2 Geosynthetic

Yes	No	N/A	
___	___	___	(a) Polymer type and grade
___	___	___	(b) HDPE: resin type, class, grade and category
___	___	___	(c) Minimum intrinsic viscosity correlated to number of average molecular weight and maximum carboxyl end groups
___	___	___	(d) Weight per unit area
___	___	___	(e) Minimum average roll value for ultimate strength
___	___	___	(f) Creep reduction factor for 75 and 100 year design life, including effect of temperatures
___	___	___	(g) Durability reduction factor (chemical, hydrolysis, oxidation)
___	___	___	(h) Additional durability reduction factor for high biologically active environments
___	___	___	(i) Installation damage reduction factor for range of backfill (select backfill, course aggregate)
___	___	___	(j) UV resistance

1.3 Facing Connection(s)

Yes	No	N/A	
___	___	___	(a) Mode (structural, frictional or combined)
___	___	___	(b) Connection strength as a percentage of reinforcement strength at various confining pressures for each reinforcement product and connection type submitted
___	___	___	(c) Composition of devices, dimensions, tolerances
___	___	___	(d) Full scale connection test method/results

1.4 Range of Backfill

Yes	No	N/A	
___	___	___	(a) Soil classification, gradation, unit weight, friction angle for reinforcement method
___	___	___	(b) Soil classification, gradation, unit weight, friction angle for facing type

(Note: Backfill must meet AGENCY requirements.)

1.5 Leveling Pad

Yes	No	N/A	
___	___	___	(a) Cast-in-place
___	___	___	(b) Precast
___	___	___	(c) Granular

1.6 Drainage Elements

Yes	No	N/A	
___	___	___	(a) Modular Block Core and Drainage Backfill
___	___	___	(b) Pipe Drainage Backfill

1.7 Coping

Yes	No	N/A	
___	___	___	(a) Precast
___	___	___	(b) Precast attachment method/details
___	___	___	(c) Cast-in-place

1.8 Traffic Barrier

Yes	No	N/A	
___	___	___	(a) Precast
___	___	___	(b) Cast-in-place

1.9 Connections to Appurtenances

Yes	No	N/A	
___	___	___	(a) Precast

Part Two: Design

Clearly identify that the design conforms to the *AASHTO LRFD Bridge Design Specifications* and the GDM. Identify design assumptions and procedures with specific references (e.g., design code sections) for each of the listed items.

2.1 *AASHTO LRFD* Provisions

Yes	No	N/A	
___	___	___	(a) Sliding
___	___	___	(b) Overturning (including vehicle collision)
___	___	___	(c) Bearing resistance
___	___	___	(d) Compound stability
___	___	___	(e) Seismic
___	___	___	(f) Movement at service limit state
___	___	___	(g) Passive resistance and sliding
___	___	___	(h) Safety against structural failure
___	___	___	(i) Drainage

2.2 Performance Criteria

Yes	No	N/A	
___	___	___	(a) Erection tolerances
___	___	___	(b) Horizontal/vertical deflection limits

2.3 Drawings

Provide representative drawings showing all standard details along with any alternate details, as required in Appendix 15-B.6.

Yes No N/A
___ ___ ___ (a) Details

2.4 Specifications

Provide sample specifications for:

Yes No N/A
___ ___ ___ (a) Wall system component materials

2.5 Calculations

Provide detailed calculations for the example problems in Appendix 15-B.5. Explain all assumptions and calculations. Example problem calculations, including computer assisted analyses, shall be sealed and performed under the responsible charge of a Professional Engineer licensed in the State of Oregon.

Yes No N/A
___ ___ ___ (a) Calculations

2.6 Computer Support

If a computer program is used to support vendor MSE wall designs, it shall be the latest version and latest update of MSEW (Adama Engineering, Inc.).

Yes No N/A
___ ___ ___ (a) Computer programs used

Part Three: Construction

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

3.1 Fabrication of Facing Units

Yes No N/A
___ ___ ___ (a) Curing methods
___ ___ ___ (b) Concrete surface finish requirements

3.2 Field Construction Manual

Provide a documented field construction manual describing in detail and with illustrations as necessary the step-by-step construction sequence, including requirements for:

Yes	No	N/A	
___	___	___	(a) Foundation preparation
___	___	___	(b) Special tools required
___	___	___	(c) Leveling pad
___	___	___	(d) Facing erection
___	___	___	(e) Facing batter for alignment
___	___	___	(f) Steps to maintain horizontal and vertical alignment
___	___	___	(g) Retained and backfill placement/compaction
___	___	___	(h) All equipment requirements

3.3 Contractor or Subcontractor Prequalification Requirements

List any contractor or subcontractor prequalification's.

Yes	No	N/A	
___	___	___	(a) Contractor prequalification's

Part Four: Performance

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

4.1 Project Performance History

Provide a well-documented history of performance (with photos, where available), including:

Yes	No	N/A	
___	___	___	(a) Oldest
___	___	___	(b) Highest
___	___	___	(c) Projects experiencing maximum measured settlement (total and differential) measurements of lateral movement/tilt
___	___	___	(d) Demonstrated aesthetics
___	___	___	(e) Maintenance history

15-B.4 Submittal Requirements for Proprietary Prefabricated Modular Retaining Wall Systems:

Instructions:

To expedite the evaluation of the Prefabricated Modular Retaining Wall system, applicants must furnish information as indicated in the Checklist. The Checklist items should be referenced to assure that the submittal package includes all of the listed information. The submittal package should be organized according to the numbered items in the Checklist. The completed Checklist should be included with the submitted package.

Part One:

Identify material specification designations that govern the materials that are used in furnishing the wall system elements and components. Provide product literature or other documentation that describes the wall system, its elements and components and adequately addresses the checklist items. Identify precast concrete facilities that have experience with fabricating the concrete elements and components of the wall system.

1.1 Concrete Facing Unit

Yes	No	N/A	
___	___	___	(a) Standard dimensions and tolerances
___	___	___	(b) Joint sizes
___	___	___	(c) Concrete strength
___	___	___	(d) Wet cast concrete % air (range)
___	___	___	(e) Moisture absorption (percent by weight)
___	___	___	(f) Scaling resistance
___	___	___	(g) Freeze thaw durability
___	___	___	(h) Facing unit to facing unit shear resistance
___	___	___	(i) Bearing pads (joints)
___	___	___	(j) Spacers (pins, etc.)
___	___	___	(k) Joint filter requirements: geotextile or graded granular
___	___	___	(l) Aesthetic choices (texture, relief, color, graffiti treatment)
___	___	___	(m) Other facing materials

1.2 Leveling Pad

Yes	No	N/A	
___	___	___	(a) Cast-in-place
___	___	___	(b) Precast
___	___	___	(c) Granular

1.3 Drainage Elements

Yes	No	N/A	
___	___	___	(a) Weep holes
___	___	___	(b) Base
___	___	___	(c) Backfill
___	___	___	(d) Surface

1.4 Coping

Yes	No	N/A	
___	___	___	(a) Precast
___	___	___	(b) Precast attachment method/details
___	___	___	(c) Cast-in-place

1.5 Traffic Barrier

Yes	No	N/A	
___	___	___	(a) Precast
___	___	___	(b) Cast-in-place

1.6 Connections to Appurtenances

Yes	No	N/A	
___	___	___	(a) Precast
___	___	___	(b) Precast attachment method/details
___	___	___	(c) Cast-in-place

Part Two: Design

Clearly identify that the design conforms to the AASHTO LRFD Bridge Design Specifications. Identify design assumptions and procedures with specific references (e.g., design code sections) for each of the listed items.

2.1 AASHTO LRFD Provisions

Yes	No	N/A	
___	___	___	(a) Sliding
___	___	___	(b) Overturning (including vehicle collision)
___	___	___	(c) Bearing resistance
___	___	___	(d) Overall stability
___	___	___	(e) Seismic
___	___	___	(f) Movement at service limit state
___	___	___	(g) Passive resistance and sliding
___	___	___	(h) Safety against structural failure
___	___	___	(i) Drainage

2.2 Performance Criteria

Yes	No	N/A	
___	___	___	(a) Erection tolerances
___	___	___	(b) Horizontal/vertical deflection limits

2.3 Drawings

Provide representative drawings showing all standard details along with any alternate details, as required in Appendix 15-B.6.

Yes	No	N/A	
___	___	___	(a) Details

2.4 Specifications

Provide sample specifications for:

Yes	No	N/A	
___	___	___	(a) Wall system component materials

2.5 Calculations

Provide detailed calculations for the example problems in Appendix 15-B.5. Explain all assumptions and calculations. Example problem calculations, including computer assisted analyses, shall be sealed and performed under the responsible charge of a Professional Engineer licensed in the State of Oregon.

Yes	No	N/A	
___	___	___	(a) Calculations

2.6 Computer Support

If a computer program is used for design of Agency projects, provide hand calculations for the required example problems demonstrating the reasonableness of computer results.

Yes	No	N/A	
___	___	___	(a) Computer program used

Part Three: Construction

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

3.1 Fabrication of Facing Units

Yes	No	N/A	
___	___	___	(a) Curing methods
___	___	___	(b) Concrete surface finish requirements

3.2 Field Construction Manual

Provide a documented field construction manual describing in detail and with illustrations as necessary the step-by-step construction sequence, including requirements for:

Yes	No	N/A	
___	___	___	(a) Foundation preparation
___	___	___	(b) Special tools required
___	___	___	(c) Leveling pad
___	___	___	(d) Facing erection
___	___	___	(e) Facing batter for alignment
___	___	___	(f) Steps to maintain horizontal and vertical alignment
___	___	___	(g) Retained and backfill placement/compaction
___	___	___	(h) Erosion mitigation
___	___	___	(i) All equipment requirements

3.3 Contractor or Subcontractor Prequalification Requirements

List any contractor or subcontractor prequalification's.

Yes	No	N/A	
___	___	___	(a) Contractor prequalification's

Part Four: Performance

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

4.1 Project Performance History

Provide a well-documented history of performance (with photos, where available), including:

Yes	No	N/A	
___	___	___	(a) Oldest
___	___	___	(b) Highest
___	___	___	(c) Projects experiencing maximum measured settlement (total and differential) measurements of lateral movement/tilt
___	___	___	(d) Demonstrated aesthetics possibilities
___	___	___	(e) Maintenance history

15-B.5 Proprietary Retaining Wall System Example Problems (by Preapproval Category):

Introduction:

This appendix includes example problems that are referenced in Appendix 15-B.3 and Appendix 15-B.4.

Submit all calculations in LRFD format, in accordance with the *AASHTO LRFD Bridge Design Specifications*, as modified by the *ODOT GDM,* unless specified otherwise. Investigate all applicable limit states (load combinations), with load factors selected to produce the total extreme force effects. Loads stated in the example problems are all unfactored loads (unless noted otherwise), and require the Manufacturer to apply appropriate load factors.

Calculations and computer output for each example problem shall include or be accompanied by a design narrative. The design narrative shall define all variables, state and justify all design assumptions and interpretations, describe all design steps performed, show the results of each design step, and show that the results satisfy all applicable design requirements. Include references to all applicable GDM, AASHTO, and FHWA sections. Also provide dimensioned plans, details, and sectional views showing the retaining wall design for each example problem. Once a proprietary retaining wall system is preapproved, the solved example problems will become the standard for preparation of all project specific proprietary retaining wall submittals, as well as for Agency review of the Manufacturer submittals.

The example problems show an MSE retaining wall system. If the proposed proprietary retaining wall system is not an MSE retaining wall system, the Manufacturer should substitute the proposed wall type, in accordance with the requirements of the example problems.

Unless specified otherwise in the example problems, the Manufacturer shall select the wall height to be used in the example problem calculations (subject to *AASHTO* and *ODOT GDM* requirements). The wall height used in the example problems, if preapproved by ODOT, will become the maximum wall height allowed for the specific retaining wall system on Agency projects.

Required example problems (by category):

- Highway retaining walls: Submit calculations for Retaining Wall Example Problems #1 and #2. Also submit Example Problem #4 when requesting preapproval for tiered Highway retaining wall applications.
- Bridge retaining walls: In addition to the calculations required for Highway retaining walls, submit calculations for Example Problem #3. Since the GDM does not allow the use of prefabricated modular bridge retaining walls, do not submit Example Problem #3 for proprietary prefabricated modular walls.
- Minor retaining walls: Submit calculations for Retaining Wall Example Problem #5 only. Proprietary minor retaining wall systems shall be one of the following retaining wall types:
  - Dry cast concrete block prefabricated modular retaining wall system (Type 2E);
  - Wet cast concrete block prefabricated modular retaining wall system (Type 2F); or
  - Gabion prefabricated modular retaining wall system (Type 2D).

Retaining Wall Example Problem #1:

- See Figure 15-14 Problem # 1
- Wall is parallel to roadway
- Wall height: Maximum wall height for which preapproval is requested
- Design life: 75 years
- Backslope: Level
- Foreslope: Level
- EH Lateral earth pressure: Yes
- ES Earth surcharge load: No
- EV Vertical pressure from dead load of earth fill: Yes
- DC Component Dead Loads: Yes
- DW Dead load of future wearing surface: Assume DW = 50 psf
- EQ Earthquake loading:
  - Assume peak ground acceleration coefficient “PGA” = 0.22 g.
  - Assume “Site Class” is “D.”
  - Site adjusted seismic coefficient “A_s” = 0.30 g.
  - Assume the Agency EOR has determined that the M-O method is applicable but that a reduction to A_s is not applicable (i.e., $k_h = A_s$).
  - Assume the load factor on live load equals 0.50 for the Extreme Event I limit state.
- CT Vehicular collision force for MSE walls. For this example problem:
  - Assume Type “F” (32 in.) traffic barrier coping with self-supporting moment slab (ODOT Standard Drawing BR760).
  - Design MSE walls to ensure soil reinforcements do not rupture or pullout due to vehicle impact loads on traffic railing in accordance with Section 15.6.10.

- The vehicular collision force shall be as specified in AASHTO LRFD 11.10.10.2 and the ODOT GDM.
  - Assume no load is transferred directly from the traffic barrier coping/moment slab to the wall facing.
- CT Vehicular collision force for prefabricated modular walls. For this example problem:
  - Assume Type 2A Guardrail (see ODOT Standard Drawing RD400) with 5.0 ft. of embedment, and located at least three feet clear from the back of the wall.
  - The vehicular collision force shall be as specified in AASHTO LRFD 11.10.10.2 and the ODOT GDM.
- Assume drained conditions for the reinforced soil, retained soil, and foundation soil.
- Assume reinforced backfill soil friction angle (Φ_2) = 34° (for MSE walls)
- Assume reinforced backfill cohesion (C_2) = 0 psf (for MSE walls)
- Assume reinforced backfill unit weight (γ_2) = 130 pcf (for MSE walls)
- Assume retained soil friction angle (Φ_1) = 32°
- Assume retained soil cohesion (C_1) = 0 psf
- Assume retained soil unit weight (γ_1) = 120 pcf
- Assume foundation soil friction angle (Φ_3) = 30°
- Assume foundation soil cohesion (C_3) = 0 psf
- Assume foundation soil unit weight (γ_3) = 120 pcf
- Assume wall embedment = H/20 or 2.0 ft. (whichever is greater)
- Assume bearing resistance and settlement of foundation soils is acceptable for all limit states
- Assume overall stability does not govern reinforcement length (for MSE walls)

Problem #1

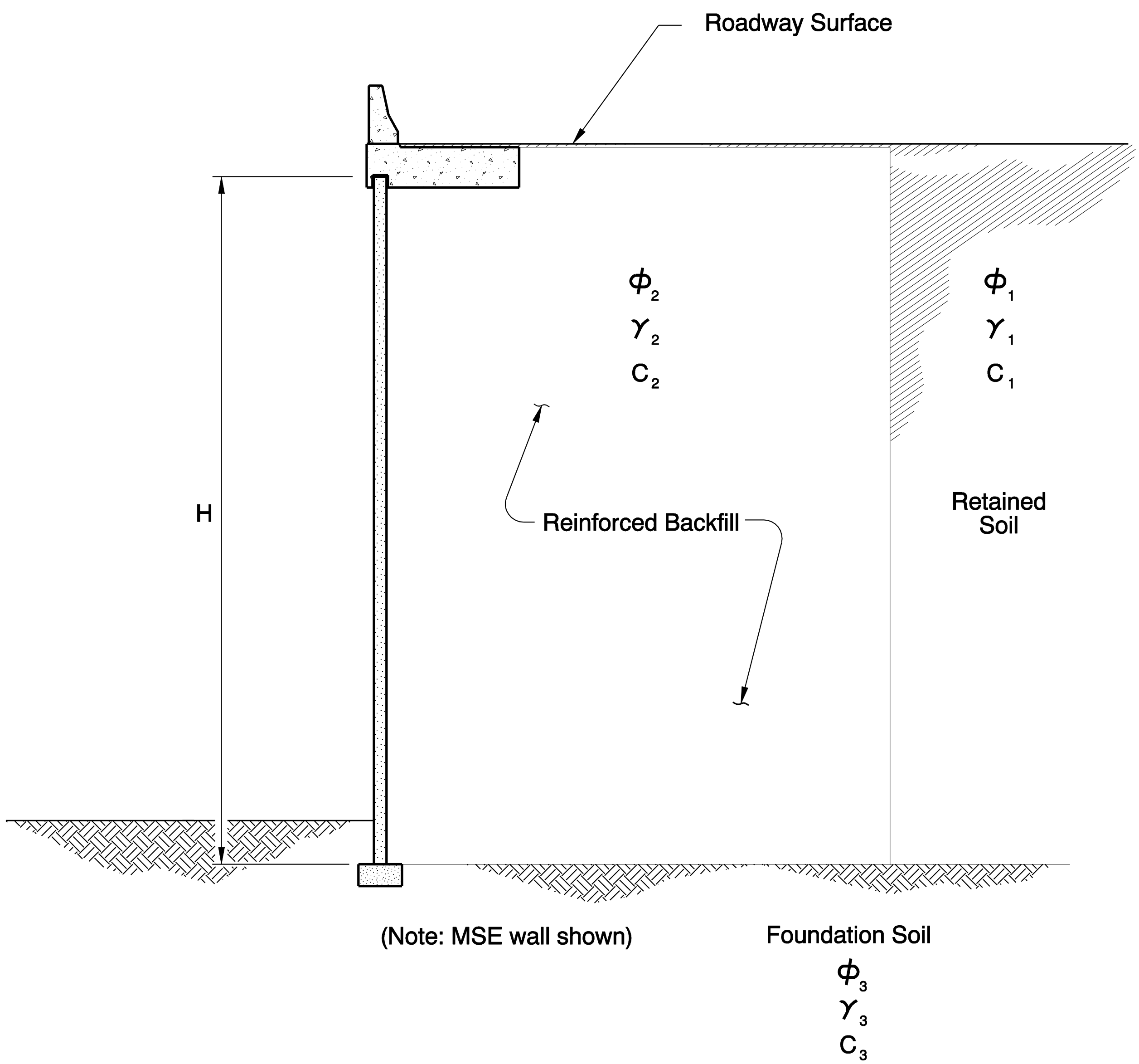

Figure 15-14. Problem # 1.

Retaining Wall System Example Problem #2:

- See Figure 15-15 Problem # 2**.**
- Wall is parallel to roadway
- Wall height: Assume the maximum wall height for which preapproval is requested
- Design life: 75 years
- Backslope (β): 1v:2h (assume length of slope is 100 ft.)
- Foreslope: Level
- EH Lateral earth pressure: Yes
- ES Earth surcharge load: No
- EV Vertical pressure from dead load of earth fill: Yes
- DC Component Dead Loads: Yes
- DW Dead load of future wearing surface: No
- LS Live load surcharge: No
- EQ Earthquake load:
  - Assume peak ground acceleration coefficient "PGA" = 0.22 g.
  - Assume "Site Class" is "D".
  - Site adjusted seismic coefficient "A_s" = "k_{max}" = 0.30 g.
  - Assume the Agency EOR has determined the M-O method is not applicable.
  - Assume the total seismic thrust coefficient (K_{AE}) was obtained using the GLE method in accordance with FHWA, 2009.
  - Assume the total seismic thrust coefficient (K_{AE}) equals 1.00. No reduction to K_{AE} is applicable. Use K_{AE} to calculate the total seismic thrust (P_{AE}), which includes both the active (static) thrust and the dynamic (seismic) thrust.
  - Calculate P_{AE} based on the height $H_2 = H+[(0.5H*Tan(\beta))/(1-0.5*Tan(\beta)]$, where H is shown in Figure 15-15.
  - Assume the total seismic thrust (P_{AE}) is applied at a height of $H_2/2$ at the same inclination of the backslope (1v:2h).
- CT Vehicular collision force (on barrier): No
- Traffic Barrier at top of wall on barrier coping: No
- Standard cast in place concrete coping at top of wall: Yes
- Assume drained conditions in reinforced soil, retained soil, and foundation soil
- Assume reinforced backfill soil friction angle (Φ_2) = 34° (for MSE walls)
- Assume reinforced backfill cohesion (C_2) = 0 psf (for MSE walls)
- Assume reinforced backfill unit weight (γ_2) = 130 pcf (for MSE walls)
- Assume retained soil and cover friction angle (Φ_1) = 32°

- Assume retained soil and cover cohesion (C_1) = 0 psf
- Assume retained soil and cover unit weight (γ_1) = 120 pcf
- Assume foundation soil friction angle (Φ_3) = 30°
- Assume foundation soil cohesion (C_3) = 0 psf
- Assume foundation soil unit weight (γ_3) = 120 pcf
- Assume wall embedment = H/20 or 2.0 ft. (whichever is greater)
- Assume soil bearing resistance and settlement of foundation soils is acceptable for all limit states
- Check compound stability but assume overall stability is acceptable
- Assume overall stability does not govern reinforcement length (for MSE walls)

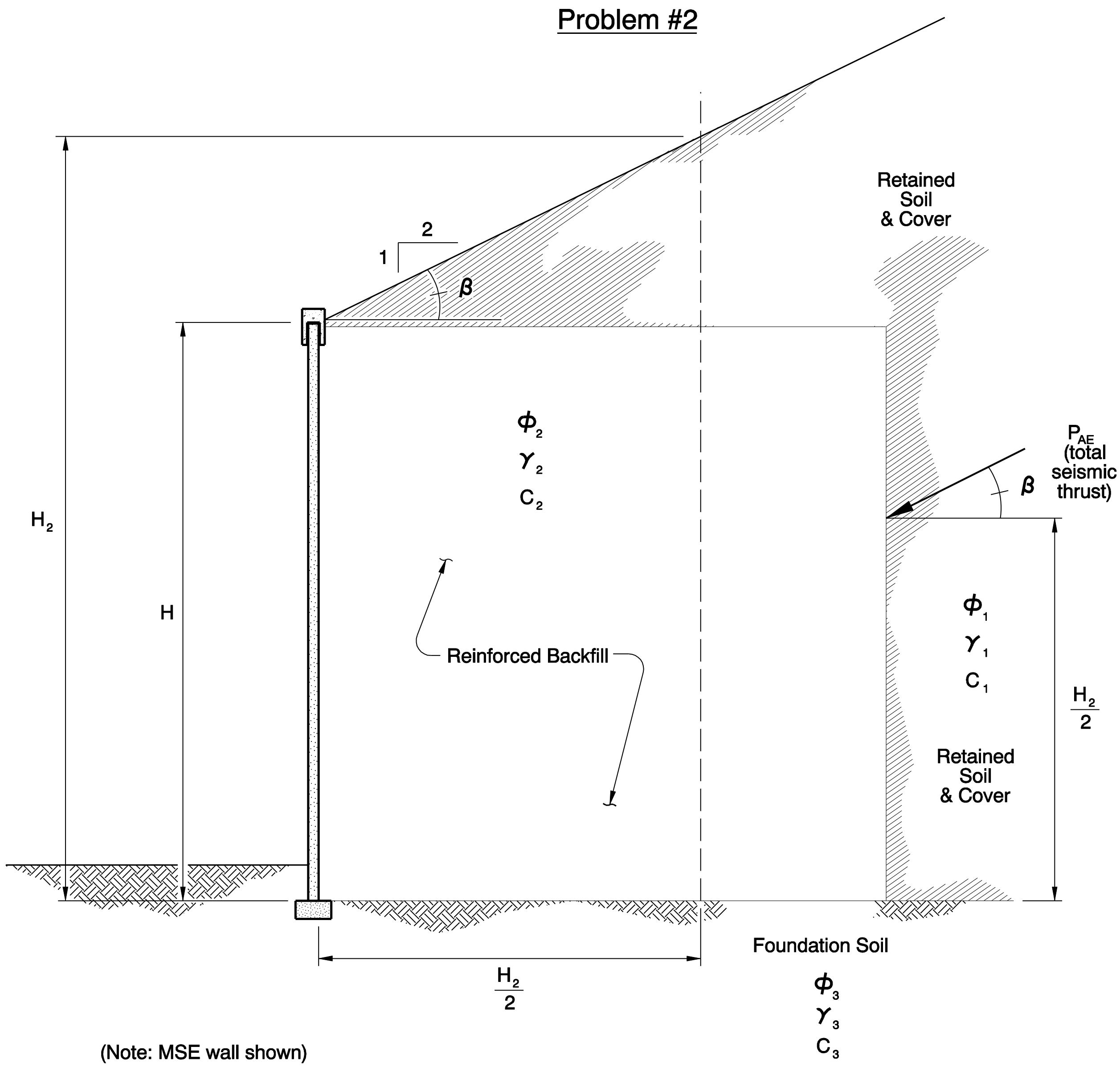

Figure 15-15. Problem # 2.

Retaining Wall System Example Problem #3:

- See Figure 15-16 Problem # 3.
- Wall is transverse to upper roadway (not "U" or "L" shaped).
- Assume wall height "H" = 22.0 ft.
- Design life: 75 years
- Backslope: Level
- Foreslope: Level
- EH Lateral earth pressure: Yes
- ES Earth surcharge load: Unfactored bridge reactions on spread footing are as follows:
  - o Pv (dead) = 3.50 k/(ft. of wall)
  - o Pv (live) = 3.50 k/(ft. of wall)
  - o Ph. (seismic, normal to wall) = 1.00 k/(ft. of wall)
- EV Vertical pressure from dead load of earth fill: Yes
- DC Component Dead Loads: Yes
- LS Live load surcharge on bridge approach: Yes
- EQ Earthquake load
  - o Assume peak ground acceleration coefficient "PGA" = 0.22 g.
  - o Assume "Site Class" is "D".
  - o (Site adjusted seismic coefficient "A_s" = 0.30 g).
  - o Assume the Agency EOR has determined that the M-O method is applicable but that a reduction to A_s is not applicable (i.e., $k_h = A_s$).
  - o Assume the load factor on live load equals 0.50 for the Extreme Event I limit state.
- Assume drained conditions in reinforced soil, retained soil, and foundation soil
- Assume reinforced backfill soil friction angle (Φ_2) = 34° (for MSE walls)
- Assume reinforced backfill cohesion (C_2) = 0 psf (for MSE walls)
- Assume reinforced backfill unit weight (γ_2) = 130 pcf (for MSE walls)
- Assume retained soil friction angle (Φ_1) = 32°
- Assume retained soil cohesion (C_1) = 0 psf
- Assume retained soil unit weight (γ_1) = 120 pcf
- Assume foundation soil friction angle (Φ_3) = 30°
- Assume foundation soil cohesion (C_3) = 0 psf
- Assume foundation soil unit weight (γ_3) = 120 pcf
- Assume wall embedment = H/10 or 2.0 ft. (whichever is greater)

- Assume the bearing resistance and settlement of foundation soil is acceptable for all limit states
- Check compound stability but assume overall stability is acceptable
- Assume overall stability does not govern reinforcement length (for MSE walls)

Problem #3

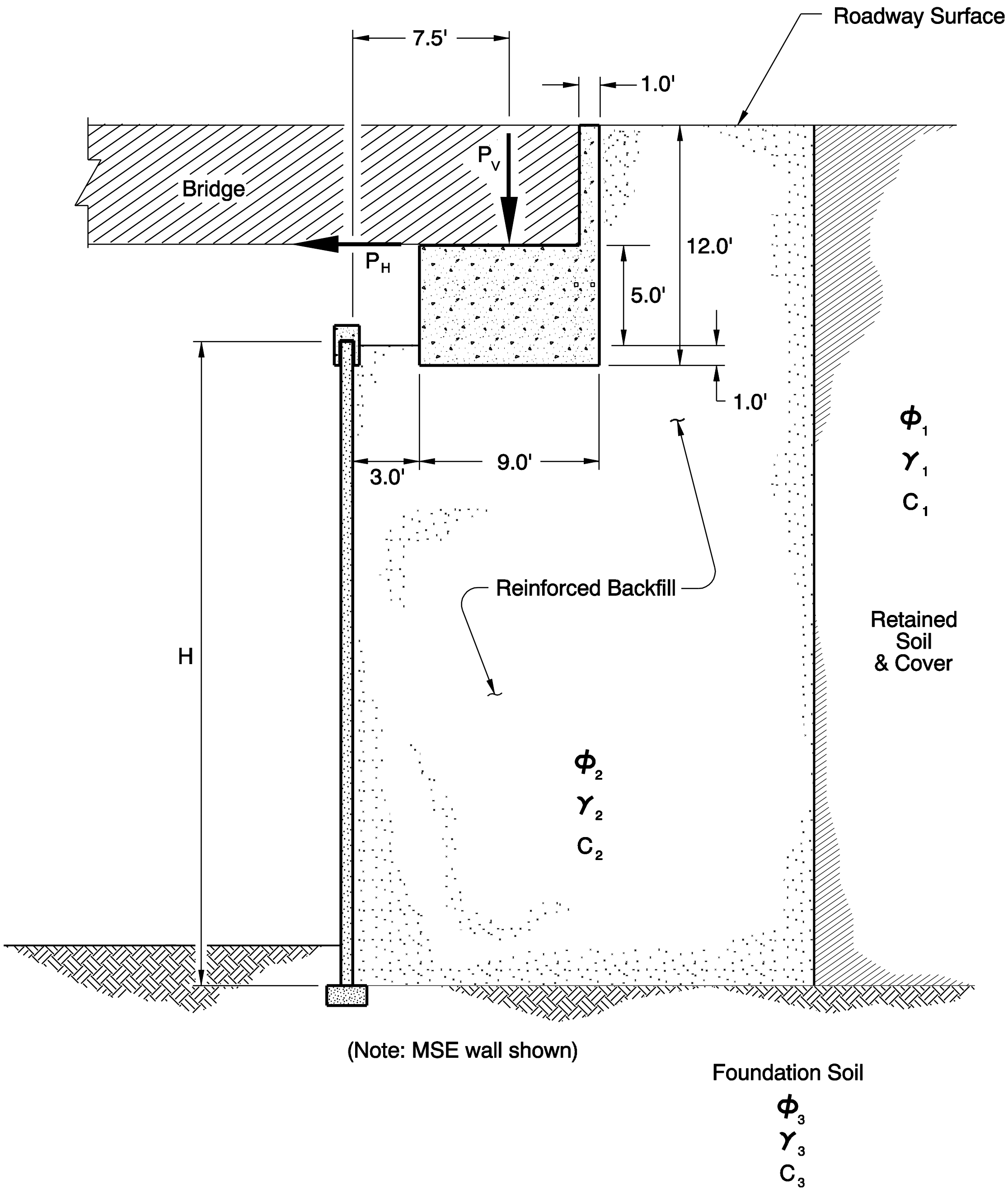

Figure 15-16. Problem # 3.

Retaining Wall System Example Problem #4

- See Figure 15-17 Problem #4.
- Walls are parallel to roadway
- Wall heights:
  - o Assume total wall height "H" is the maximum wall heights for which preapproval is requested.
  - o Assume H1 = H2 = H/2
- Design life: 75 years
- Backslope (upper wall): Level
- Backslope (lower wall): Level
- Foreslope (lower wall): Level
- EH Lateral earth pressure: Yes
- ES Earth surcharge load: Upper wall on lower wall
- EV Vertical pressure from dead load of earth fill: Yes
- DC Component Dead Loads: Yes
- DW Dead load of future wearing surface: Assume DW = 50 psf
- LS Live load surcharge: Yes
- EQ Earthquake load:
  - o Assume peak ground acceleration coefficient "PGA" = 0.22 g.
  - o Assume "Site Class" is "D".
  - o Site adjusted seismic coefficient "A_s" = 0.30 g.
  - o Assume the Agency EOR has determined that the M-O method is applicable but that a reduction to A_s is not applicable (i.e., $k_h = A_s$).
  - o Assume the load factor on live load equals 0.0 for the Extreme Event I limit state.
- CT Vehicular collision force for MSE walls - For this example problem:
  - o Assume Type "F" (32 in.) traffic barrier coping with self-supporting moment slab (ODOT Standard Drawing BR760).
  - o Design MSE walls to ensure soil reinforcements do not rupture or pullout due to vehicle impact loads on traffic railing in accordance with Section 15.6.10.
  - o The vehicular collision force shall be as specified in AASHTO LRFD 11.10.10.2 and the ODOT GDM.
  - o Assume no load is transferred directly from the traffic barrier coping/moment slab to the wall facing.
- CT Vehicular collision force for prefabricated modular walls. For this example problem
  - o Assume Type 2A Guardrail (see ODOT Standard Drawing RD400) with 5.0 ft. of embedment, located at least 3.0 ft. clear from the back of the wall.

  - The vehicular collision force shall be as specified in AASHTO LRFD Section 11.10.10.2 and the ODOT GDM.
- Standard cast in place concrete coping at top of walls: No
- Assume drained conditions for reinforced soil, retained soil, and foundation soil
- Assume reinforced backfill soil friction angle (Φ_2) = 34° (for MSE walls)
- Assume reinforced backfill cohesion (C_2) = 0 psf (for MSE walls)
- Assume reinforced backfill unit weight (γ_2) = 130 pcf (for MSE walls)
- Assume retained soil friction angle (Φ_1) = 32°
- Assume retained soil cohesion (C_1) = 0 psf
- Assume retained soil unit weight (γ_1) = 120 pcf
- Assume foundation soil friction angle (Φ_3) = 30°
- Assume foundation soil cohesion (C_3) = 0 psf
- Assume foundation soil unit weight (γ_3) = 120 pcf
- Assume lower wall embedment = H/20 or 2 ft., whichever is greater
- Assume upper wall embedment = 1.0 ft.
- Assume foundation soil bearing resistance and settlement are acceptable at all applicable limit states.
- Check compound stability, but assume overall stability is acceptable
- Assume overall stability does not govern reinforcement length (for MSE walls)

Problem #4

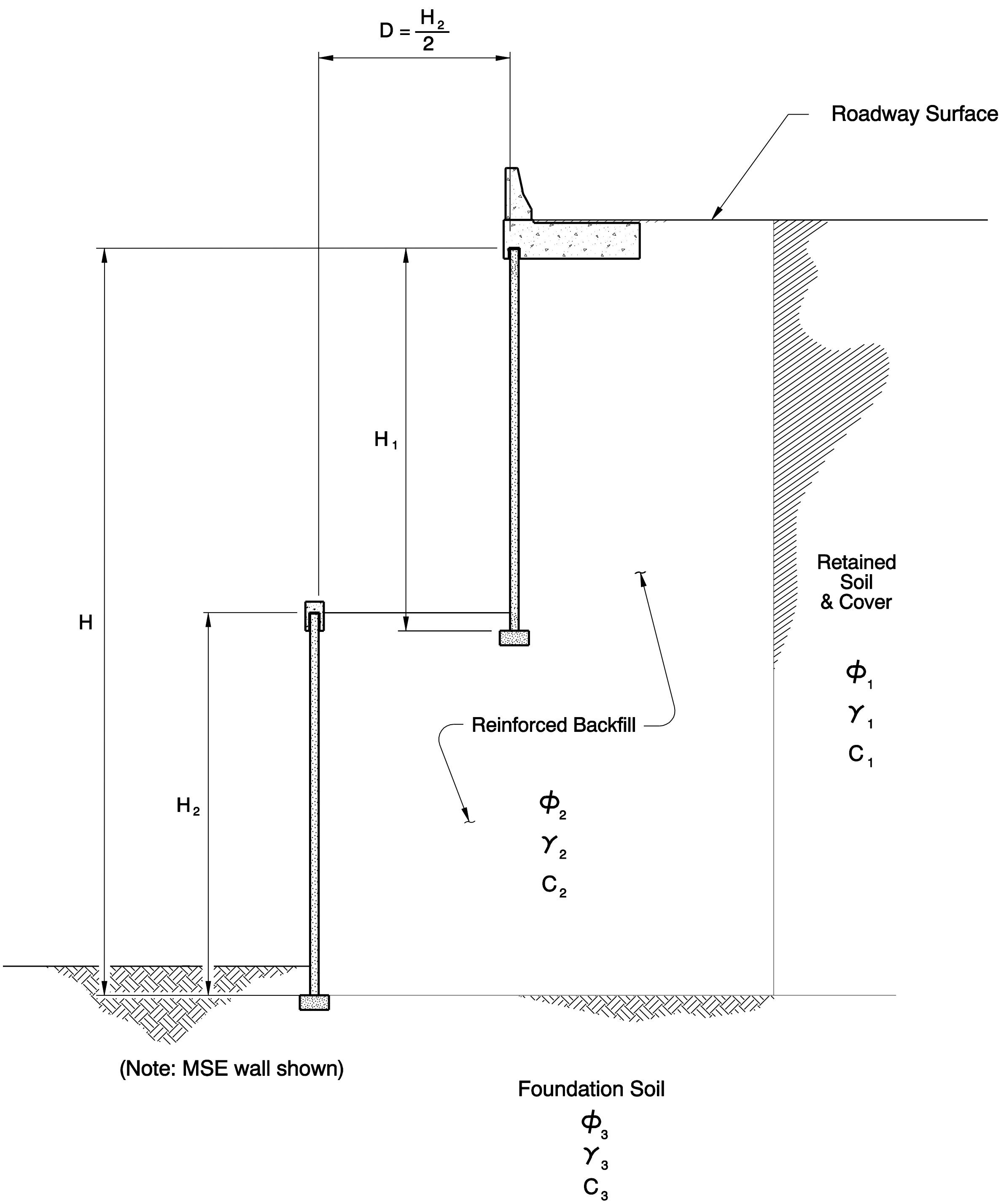

Figure 15-17. Problem # 4.

Retaining Wall System Example Problem #5:

- See Figure 15-18 Problem #5**.**
- See Section 15.3.23.
- Wall is parallel to roadway
- Wall height: 4.0 ft.
- Design life: 75 years
- Backslope: Level
- Foreslope: Level
- EH Lateral earth pressure: Yes
- ES Earth surcharge load: No
- EV Vertical pressure from dead load of earth fill: Yes
- EQ Earthquake load: No
- DC Component Dead Loads: Yes
- DW Dead load of future wearing surface: No
- LS Live load surcharge: No
- CT Vehicular collision force: No
- Assume drained conditions in retained and foundation soil
- Assume gravel leveling pad angle of internal friction is equal to 34°
- Assume the foundation soil (below the leveling pad) is noncohesive soil with angle of internal friction equal to 30°
- Assume retained soil friction angle (Φ_1) = 34°
- Assume retained soil cohesion (C_1) = 0 psf
- Assume retained soil unit weight (γ_1) = 120 pcf
- Assume wall embedment: 0.5 ft.
- Assume foundation soil bearing resistance and settlement are acceptable for all limit states
- Assume overall stability is acceptable
- Assume the active earth pressure coefficient (ka) = 0.31
- Base coefficient of friction = 0.45 when designing sliding stability

<u>Problem #5</u>

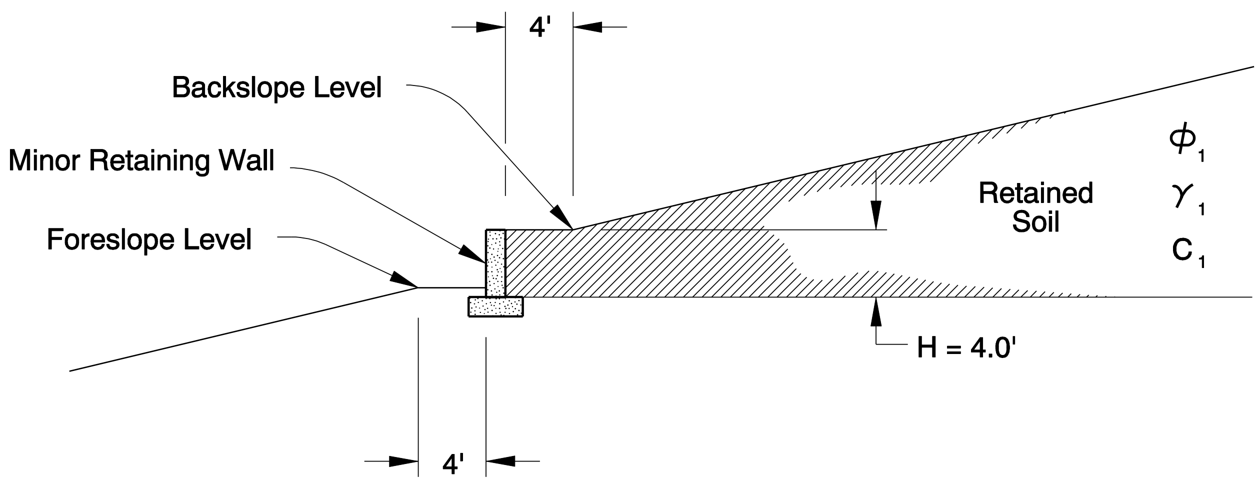

Figure 15-18. Problem # 5.

15-B.6 Requirements for Proprietary Retaining Wall System Detail Drawings (by Preapproval Category):

Provide the following drawings for proprietary retaining wall systems (as a minimum):

15-B.6.1 Bridge Retaining Wall Systems

- Wall elements
- Connection details
- Details at bridge abutment
- Appurtenance connection details
- Obstruction avoidance details
- Corrosion protection details
- Basic wall construction details
- Roadway drainage inlet details
- Drainage swale at top of wall
- Typical drainage details behind wall
- Culverts through wall

- Sidewalk at top of wall
- Pedestrian rail at top of wall
- Fencing at top of wall
- Traffic barrier at top of wall
- Guardrail at top of wall
- Standard coping
- Barrier coping
- Leveling pad or other base details
- Backfill reinforcement details (MSE walls)

15-B.6.2 Highway Retaining Wall Systems

- Wall elements
- Connection details
- Appurtenance connection details
- Obstruction avoidance details
- Corrosion protection details
- Basic wall construction details
- Roadway drainage inlet details
- Drainage swale at top of wall
- Typical drainage details behind wall
- Culverts through wall
- Sidewalk at top of wall
- Pedestrian rail at top of wall
- Fencing at top of wall
- Traffic barrier at top of wall
- Guardrail at top of wall
- Standard coping
- Barrier coping
- Sidewalk coping
- Leveling pad or other base details
- Backfill reinforcement details (MSE walls)

15-B.6.3 Minor Retaining Wall Systems

- Basic wall construction details
- Typical drainage details at heel of wall

Appendix 15-C

Guidelines for Review of Proprietary Retaining Wall System Working Drawings and Calculations

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

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

15-C.1 Geotechnical Design Issues

The following design issues should have already been addressed by the Geotechnical Engineer of Record in the development of the contract requirements:

- Design parameters are appropriate for the site soil/rock conditions
- Wall is stable for overall stability and compound stability (service and extreme event limit states)
- Settlement is within acceptable limits for the specific wall type(s) allowed by the contract (service limit state)
- The design for any mitigating measures to provide adequate bearing resistance, overall stability, compound stability, to address seismic hazards such as liquefaction consistent with the policies provided in Chapter 6 of the ODOT GDM, and to keep settlement within acceptable tolerances for the allowed wall is fully addressed (service, strength and extreme event limit states)
- The design for drainage of the wall, both behind and within the wall, has been completed and is implemented to insure long-term drainage

15-C.2 External Stability Design

15-C.2.1 Structure Geometry

Are the structure dimensions, design cross-sections, and any other requirements affecting the design of the wall consistent with the contract requirements? As a minimum, check wall length, top elevation (both coping and barrier, if present), finished ground line elevation in front of wall, horizontal curve data, and locations and size of all obstructions (e.g., utilities, drainage structures, sign foundations, etc.) in the reinforced backfill, if any are present.

15-C.2.2 Design Procedure

Has the correct design procedure been used, including the correct earth pressures, earth pressure coefficients, and any other input parameters specified in the contract, both for static and seismic design?

15-C.2.3 Load Combinations

Have appropriate load combinations for each limit state been selected?

15-C.2.4 Load Factors

Have the correct load factors been selected, both in terms of magnitude and for those load factors that have maximum and minimum values, has the right combination of maximum and minimum values been selected?

15-C.2.5 Live Load

Has live load been treated correctly regarding magnitude and location (over reinforced zone for bearing, behind reinforced zone for sliding and overturning)?

15-C.2.6 Seismic

Have the correct PGA, A_s, k_h, and k_v, been used for seismic design?

15-C.2.7 Resistance Factors

Have the correct resistance factors been selected for each limit state, and is the wall stable against sliding?

15-C.2.8 Soil Properties

Have the correct soil properties been used in the analyses (reinforced zone properties and retained fill properties)?

15-C.2.9 External Loads

Have the required external loads been applied in the analysis (external foundation loads, soil surcharge loads, etc.)?

15-C.2.10 Wall Widths

Have minimum specified wall widths (i.e., *AASHTO LRFD* specified minimum reinforcement lengths, *ODOT GDM, Chapter 15* specified minimum reinforcement lengths, and minimum reinforcement lengths specified to insure overall stability), in addition to those required for external and internal stability, been met in the final wall design?

15-C.2.11 Wall Embedment

Does the wall embedment meet the minimum embedment criteria specified?

15-C.2.12 Bearing Stresses

Are the maximum factored bearing stresses less than or equal to the factored bearing resistance for the structure for all limit states (service, strength, and extreme event)?

15-C.2.13 Computer Output Checks

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

15-C.2.14 Special Design Requirements

Have all the special design requirements specified in the contract that are in addition to the ODOT GDM and AASHTO LRFD Specification requirements been implemented in the Manufacturer's design?

15-C.2.15 Design Documents and Plan Details

Have the design documents and plan details been certified in accordance with the contract?

15-C.3 Internal Stability Design

15-C.3.1 Design Procedure

Has the correct design procedure been used, including the correct earth pressures and earth pressure coefficients?

15-C.3.2 Load Combinations

Have the appropriate load combinations for each limit state been selected?

15-C.3.3 Load Factors

Have the correct load factors been selected?

15-C.3.4 Live Load

Have live load been treated correctly regarding magnitude and location (over reinforced zone for bearing, behind reinforced zone for sliding and overturning)?

15-C.3.5 External Surcharge Loads

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

15-C.3.6 Seismic

Have the correct seismic parameters been used for seismic design for internal stability?

15-C.3.7 Resistance Factors

Have the correct resistance factors been selected for design for each limit state?

15-C.3.8 Reinforcement and Connector Properties

Have the correct reinforcement and connector properties been used?

- For steel reinforcement, have the steel reinforcement dimensions and spacing been identified?
- For steel reinforcement, has it been designed for corrosion using the correct corrosion rates, correct design life (75 years, unless specified otherwise in the contract documents)?
- Have the steel reinforcement connections to the facing been designed for corrosion, and has appropriate separation between the soil reinforcement and the facing concrete reinforcement been done so that a corrosion cell cannot occur, per the *AASHTO LRFD Specifications*?
- For geosynthetic reinforcement products selected, are the long-term design nominal strengths, T_{al}, used for design consistent with the values of T_{al} provided in the *ODOT Qualified Products List (QPL)*?
- Is the use of soil reinforcement - facing connection design parameters consistent with the connection plan details provided? For steel reinforced systems, such details include the shear resistance of the connection pins or bolts, bolt hole sizes, etc. For geosynthetic reinforced systems, such details include the type of connection, and since the connection strength is specific to the reinforcement product (i.e., product material, strength, and type) – facing unit (i.e., material type and strength, and detailed facing unit geometry) combination, and the specific type of connector used, including material type and connector geometry, as well as how it fits with the facing unit. Check to make sure that the reinforcement – facing connection has been previously approved and that the approved design properties have been used.
- If a coverage ratio, R_c, of less than 1.0 is used for the reinforcement, and its connection to the facing, has the facing been checked to see that it is structurally adequate to carry the earth load between reinforcement connection points without bulging of facing units, facing unit distress, or overstressing of the connection between the facing and the soil reinforcement?
- Are the facing material properties used by the wall supplier consistent with what is required to produce a facing system that has the required design life and that is durable in light of the environmental conditions anticipated? Have these properties been backed up with appropriate supporting test data? Is the facing used by the supplier consistent with the aesthetic requirements for the project?

15-C.3.9 Limit States

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

- Reinforcement resistance in reinforced backfill (strength and extreme event)
- Reinforcement resistance at connection with facing (strength and extreme event)
- Reinforcement pullout (strength and extreme event)

15-C.3.10 Obstructions

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

15-C.3.11 Computer Output

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

15-C.3.12 Specific Requirements

Have the specific requirements, material properties, and plan details relating to internal stability specified in the sections that follow been used?

15-C.3.13 Structural Design and Detail Review

Note that for structural wall facings for MSE walls, design of prefabricated modular walls, and design of other structural wall systems, a structural design and detail review should be conducted in accordance with the *AASHTO LRFD Specifications*.

- Compare preapproved wall details to the shop drawing regarding the concrete facing panel dimensions, concrete cover, rebar size, orientation and location. This also applies to any other structural elements of the wall (e.g., steel stiffeners for welded wire facings, concrete elements and components of modular walls whether reinforced or not, etc.).
- Do the geometry and dimensions of any traffic barriers or coping shown on shop drawings match with what is required by contract drawings (may need to check other portions of contract plans for verification? Has the structural design and sizing of the barrier/reaction slab been done consistently with the AASHTO specifications? Are the barrier details constructible?
- Do notes in the shop drawings state the date of manufacture, production lot number, and piece mark be marked clearly on the rear face of each panel (if required by special the contract provisions)?

15-C.4 Wall Construction Sequence Requirements

Wall construction sequence and requirements provided in shop drawings should follow the guidelines defined in the next sections.

15-C.4.1 Construction Sequence

Make sure construction sequence and notes provided in the shop drawings do not conflict with the contract specifications (e.g., minimum lift thickness, compaction requirements, construction sequence and details, etc.). Any conflicts should be pointed out in the shop drawing review comments.

15-C.4.2 Preapproved Details and Contract Requirements

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

Appendix 15-D

Preapproved Proprietary Retaining Wall Systems

Appendix 15-D is now located on the ODOT Retaining Structures web page: