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Chapter 8  
Walls and Buried Structures

8.1 Retaining Walls

8.1.1 General

A retaining wall is a structure built to provide lateral support for a mass of earth or other material where a grade separation is required. Retaining walls depend either on their own weight, their own weight plus the additional weight of laterally supported material, or on a tieback system for their stability. Additional information is provided in Chapter 15 of the WSDOT M 46-03.

Standard designs for reinforced concrete cantilevered retaining walls, noise barrier walls (precast concrete, cast-in-place concrete, or masonry), and geosynthetic walls are shown in the Standard Plans. The Region Design PE Offices are responsible for preparing the PS&E for retaining walls for which standard designs are available, in accordance with the WSDOT Design Manual M 22-01. However, the Bridge and Structures Office may prepare PS&E for such standard type retaining walls if such retaining walls are directly related to other bridge structures being designed by the Bridge and Structures Office.

Structural earth wall (SE) systems meeting established WSDOT design and performance criteria have been listed as “pre-approved” by the Bridge and Structures Office and the Materials Laboratory Geotechnical Branch. The PS&E for “pre-approved” structural earth wall systems shall be coordinated by the Region Design PE Office with the Bridge and Structures Office, and the Materials Laboratory Geotechnical Branch, in accordance with WSDOT Design Manual M 22-01.

The PS&E for minor non-structural retaining walls, such as rock walls, gravity block walls, and gabion walls, are prepared by the Region Design PE Offices in accordance with the WSDOT Design Manual M 22-01, and any other design input from the Region Materials Office, Materials Laboratory Geotechnical Branch or Geotechnical Engineer.

All other retaining walls not covered by the Standard Plans such as soil nail walls, soldier pile walls, soldier pile tieback walls and all walls beyond the scope of the designs tabulated in the Standard Plans, are designed by the Bridge and Structures Office according to the design parameters provided by the Geotechnical Engineer.

The Hydraulics Branch of the Design Office should be consulted for walls that are subject to floodwater or are located in a flood plain. The State Bridge and Structures Architect should review the architectural features and visual impact of the walls during the Preliminary Design stage. The designer is also directed to the retaining walls chapter in the WSDOT Design Manual M 22-01 and Chapter 15 of the WSDOT Geotechnical Design Manual M 46-03, which provide valuable information on the design of retaining walls.
8.1.2 Common Types of Retaining Walls

The majority of retaining walls used by WSDOT are one of the following five types:


2. Geosynthetic Walls (Temporary and Permanent) - *Standard Plans* D-3 and *Standard Specifications* Section 6-14.


4. Soldier Pile Walls and Soldier Pile Tieback Walls - *Standard Specifications* Sections 6-16 and 6-17.


Other wall systems, such as secant pile or cylinder pile walls, may be used based on the recommendation of the Geotechnical Engineer. These walls shall be designed in accordance with the current AASHTO LRFD Bridge Design Specifications.

A. Pre-approved Proprietary Walls – A wall specified to be supplied from a single source (patented, trademark, or copyright) is a proprietary wall. Walls are generally pre-approved for heights up to 33 ft. The Materials Laboratory Geotechnical Division will make the determination as to which pre-approved proprietary wall system is appropriate on a case-by-case basis. The following is a description of the most common types of proprietary walls:

1. **Structural Earth Walls (SE)** – A structural earth wall is a flexible system consisting of concrete face panels or modular blocks that are held rigidly into place with reinforcing steel strips, steel mesh, welded wire, or geogrid extending into a select backfill mass. These walls will allow for some settlement and are best used for fill sections. The walls have two principal elements:
   - Backfill or wall mass: a granular soil with good internal friction (i.e. gravel borrow).
   - Facing: precast concrete panels, precast concrete blocks, or welded wire (with or without vegetation).

Design heights in excess of 33 feet shall be approved by the Materials Laboratory Geotechnical Division. If approval is granted, the designer shall contact the individual structural earth wall manufacturers for design of these walls before the project is bid so details can be included in the Plans. See Appendix 8.1-A2 for details that need to be provided in the Plans for manufacturer designed walls.

A list of current pre-approved proprietary wall systems is provided in Appendix 15-D of the WSDOT *Geotechnical Design Manual* M 46-03. For additional information see the retaining walls chapter in the WSDOT *Design Manual* M 22-01 and Chapter 15 of the WSDOT *Geotechnical Design Manual* M 46-03. For the SEW shop drawing review procedure see Chapter 15 of the WSDOT *Geotechnical Design Manual* M 46-03.
2. **Other Proprietary Walls** – Other proprietary wall systems such as crib walls, bin walls, or precast cantilever walls, can offer cost reductions, reduce construction time, and provide special aesthetic features under certain project specific conditions.

A list of current pre-approved proprietary wall systems and their height limitations is provided in Appendix 15-D of the WSDOT *Geotechnical Design Manual* M 46-03. The Region shall refer to the retaining walls chapter in the WSDOT *Design Manual* M 22-01 for guidelines on the selection of wall types. The Materials Laboratory Geotechnical Division and the Bridge and Structures Office Preliminary Plans Unit must approve the concept prior to development of the PS&E.

B. **Geosynthetic Wrapped Face Walls** – Geosynthetic walls use geosynthetics for the soil reinforcement and part of the wall facing. Use of geosynthetic walls as permanent structures requires the placement of a cast-in-place, precast or shotcrete facing. Details for construction are shown in Standard Plan D-3.09, D-3.10 and D-3.11.

C. **Standard Reinforced Concrete Cantilever Walls** – Reinforced concrete cantilever walls consist of a base slab footing from which a vertical stem wall extends. These walls are suitable for heights up to 35 feet. Details for construction and the maximum bearing pressure in the soil are given in the Standard Plan D-10.10 to D-10.45.

A major disadvantage of these walls is the low tolerance to post-construction settlement, which may require use of deep foundations (shafts or piling) to provide adequate support.

D. **Soldier Pile Walls and Soldier Pile Tieback Walls** – Soldier Pile Walls utilize wide flange steel members, such as W or HP shapes. The piles are usually spaced 6 to 10 feet apart. The main horizontal members are timber lagging, precast concrete lagging or cast in place concrete fascia panels which are designed to transfer the soil loads to the piles. For additional information see WSDOT *Geotechnical Design Manual* M 46-03 Chapter 15. See Appendix 8.1-A3 for typical soldier pile wall details.

E. **Soil Nail Walls** – The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing steel bars called “nails” into a slope or excavation as construction proceeds from the “top down”. Soil nailing is a technique used to stabilize moving earth, such as a landslide, or as temporary shoring. Soil anchors are used along with the strength of the soil to provide stability. The Geotechnical Engineer designs the soil nail system whereas the Bridge and Structures Office designs the wall fascia. Presently, the FHWA Publication FHWA-IF-03-017 “Geotechnical Engineering Circular No. 7 Soil Nail Walls” is being used for structural design of the fascia. See Appendix 8.1-A4 for typical soil nail wall details.
8.1.3 General Design Considerations

All designs shall follow procedures as outlined in AASHTO LRFD Bridge Design Specifications Chapter 11, the WSDOT Geotechnical Design Manual M 46-03, and this manual. See Appendix 8.1-A1 for a summary of design specification requirements for walls.

All construction shall follow procedures as outlined in the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction, latest edition.

The Geotechnical Engineer will provide the earth pressure diagrams and other geotechnical design requirements for special walls to be designed by the Bridge and Structures Office. Pertinent soil data will also be provided for pre-approved proprietary structural earth walls (SEW), non-standard reinforced concrete retaining walls, and geosynthetic walls.

8.1.4 Design of Reinforced Concrete Cantilever Retaining Walls

A. Standard Reinforced Concrete Cantilever Retaining Walls

The Standard Plan reinforced concrete retaining walls have been designed in accordance with the requirements of the AASHTO LRFD Bridge Design Specifications 4th Edition 2007 and interims through 2008.

1. Western Washington Walls (Types 1 through 4)
   a. The seismic design of these walls has been completed using and effective Peak Ground Acceleration of 0.51g. Extreme Event stability of the wall was based on 100% of the wall inertia force combined with 50% of the seismic earth pressure.
   b. Active Earth pressure distribution was linearly distributed per Section 7.7.4. The corresponding Ka values used for design were 0.24 for wall Types 1 and 2, and 0.36 for Types 3 and 4.
   c. Seismic Earth pressure distribution was uniformly distributed per WSDOT Geotechnical Design Manual M 46-03, Nov. 2008, Section 15.4.2.9, and was supplemented by AASHTO LRFD Bridge Design Specifications (Fig. 11.10.7.1-1). The corresponding Kae values used for design were 0.43 for Types 1 and 2, and 0.94 for Types 3 and 4.
   d. Passive Earth pressure distribution was linearly distributed. The corresponding Kp value used for design was 1.5 for all walls. For Types 1 and 2, passive earth pressure was taken over the depth of the footing. For Types 3 and 4, passive earth pressure was taken over the depth of the footing and the height of the shear key.
   e. The retained fill was assumed to have an angle of internal friction of 36 degrees and a unit weight of 130 pounds per cubic foot. The friction angle for sliding stability was assumed to be 32 degrees.
   f. Load factors and load combinations used per AASHTO LRFD Bridge Design Specifications 3.4.1-1 and 2. Stability analysis performed per AASHTO LRFD Bridge Design Specifications Section 11.6.3 and C11.5.5-1& 2.
g. Wall Types 1 and 2 were designed for traffic barrier collision forces, as specified in AASHTO LRFD Bridge Design Specifications section A13.2 for TL-4. These walls have been designed with this force distributed over the distance between wall section expansion joints (48 feet).

2. Eastern Washington Walls (Types 5 through 8)
   a. The seismic design of these walls has been completed using and effective Peak Ground Acceleration of 0.2g. Extreme Event stability of the wall was based on 100% of the wall inertia force combined with 50% of the seismic earth pressure.
   b. Active Earth pressure distribution was linearly distributed per Section 7.7.4 of this manual. The corresponding Ka values used for design were 0.36 for wall Types 5 and 6, and 0.24 for Types 7 and 8.
   c. Seismic Earth pressure distribution was uniformly distributed per WSDOT Geotechnical Design Manual M 46-03, Nov. 2008, Section 15.4.2.9, and was supplemented by AASHTO LRFD Bridge Design Specifications (Fig. 11.10.7.1-1). The corresponding Kae values used for design were 0.55 for Types 5 and 6, and 0.30 for Types 7 and 8.
   d. Passive Earth pressure distribution was linearly distributed, and was taken over the depth of the footing and the height of the shear key. The corresponding Kp value used for design was 1.5 for all walls.
   e. The retained fill was assumed to have an angle of internal friction of 36 degrees and a unit weight of 130 pounds per cubic foot. The friction angle for sliding stability was assumed to be 32 degrees.
   f. Load factors and load combinations used per AASHTO LRFD Bridge Design Specifications 3.4.1-1& 2. Stability analysis performed per AASHTO LRFD Bridge Design Specifications Section 11.6.3 and C11.5.5-1 & 2.
   g. Wall Types 7 and 8 were designed for traffic barrier collision forces, as specified in AASHTO LRFD Bridge Design Specifications section A13.2 for TL-4. These walls have been designed with this force distributed over the distance between wall section expansion joints (48 feet).

B. Non-Standard Reinforced Concrete Retaining Walls

For retaining walls where a traffic barrier is to be attached to the top of the wall, the AASHTO LRFD Bridge Design Specifications Extreme Event loading for vehicular collision must be analyzed. These loads are tabulated AASHTO LRFD Bridge Design Specifications Table A13.2-1. Although the current yield line analysis assumptions for this loading are not applicable to retaining walls, the transverse collision load (F_t) may be distributed over the longitudinal length (L_t) at the top of barrier. At this point, the load is distributed at a 45 degree angle into the wall. Future updates to the AASHTO LRFD Bridge Design Specifications code will address this issue.
For sliding, the passive resistance in the front of the footing may be considered if the earth is more than 2 feet deep on the top of the footing and does not slope downward away from the wall. The design soil pressure at the toe of the footing shall not exceed the allowable soil bearing capacity supplied by the Geotechnical Engineer. For retaining walls supported by deep foundations (shafts or piles), refer to Sections 7.7.5, 7.8 and 7.9.

### 8.1.5 Design of Cantilever Soldier Pile and Soldier Pile Tieback Walls

#### A. Ground Anchors (Tiebacks)

See AASHTO LRFD Bridge Design Specifications Section 11.9 “Anchored Walls”. The Geotechnical Engineer will determine whether anchors can feasibly be used at a particular site based on the ability to install the anchors and develop anchor capacity. The presence of utilities or other underground facilities, and the ability to attain underground easement rights may also determine whether anchors can be installed.

The anchor may consist of bars, wires, or strands. The choice of appropriate type is usually left to the Contractor but may be specified by the designer if special site conditions exist that preclude the use of certain anchor types. In general, strands and wires have advantages with respect to tensile strength, limited work areas, ease of transportation, and storage. However, bars are more easily protected against corrosion, and are easier to develop stress and transfer load.

The geotechnical report will provide a reliable estimate of the feasible factored design load of the anchor, recommended anchor installation angles (typically 10° to 45°), no-load zone dimensions, and any other special requirements for wall stability for each project.

Both the “tributary area method” and the “hinge method” as outlined in AASHTO LRFD Bridge Design Specifications Section C11.9.5.1 are considered acceptable design procedures to determine the horizontal anchor design force. The capacity of each anchor shall be verified by testing. Testing shall be done during the anchor installation (See Standard Specifications Section 6-17.3(8) and WSDOT Geotechnical Design Manual M 46-03).

1. The horizontal anchor spacing typically follows the pile spacing of 6 to 10 feet. The vertical anchor spacing is typically 8 to 12 feet. A minimum spacing of 4 feet in both directions is not recommended because it can cause a loss of effectiveness due to disturbance of the anchors during installation.

2. For permanent ground anchors, the anchor DESIGN LOAD, T, shall be according to AASHTO LRFD Bridge Design Specifications. For temporary ground anchors, the anchor DESIGN LOAD, T, may ignore extreme event load cases.

3. The lock-off load is 60 percent of the controlling factored design load for temporary and permanent walls (see WSDOT Geotechnical Design Manual M 46-03 Chapter 15).
Permanent ground anchors shall have double corrosion protection consisting of an encapsulation-protected tendon bond length as specified in the WSDOT General Special Provisions. Typical permanent ground anchor details are provided in the Appendix 8.1.

Temporary ground anchors may have either double corrosion protection consisting of an encapsulation-protected tendon bond length or simple corrosion protection consisting of grout-protected tendon bond length.

B. **Design of Soldier Pile**

The soldier piles shall be designed for shear, bending, and axial stresses according to the latest AASHTO LRFD Bridge Design Specifications and WSDOT Geotechnical Design Manual M 46-03 design criteria. The bending moment shall be based on the elastic section modulus “S” for the entire length of the pile for all Load combinations.

1. **Application of Lateral Loads**
   
   a. Lateral loads are assumed to act over one pile spacing above the base of excavation in front of the wall. These lateral loads result from horizontal earth pressure, live load surcharge, seismic earth pressure, or any other applicable load.

   b. Lateral loads are assumed to act over the shaft diameter below the base of excavation in front of the wall. These lateral loads result from horizontal earth pressure, seismic earth pressure or any other applicable load.

   c. Passive earth pressure usually acts over three times the shaft diameter or one times the pile spacing, whichever is smaller.

2. **Determining Depth of Pile Embedment**

   The depth of embedment of soldier piles shall be the maximum embedment as determined from the following:

   a. 10 feet

   b. As recommended by the Geotechnical Engineer of Record

   c. As required for skin friction resistance and end bearing resistance.

   d. As required to satisfy horizontal force equilibrium and moment equilibrium about the bottom of the soldier pile for cantilever soldier piles without permanent ground anchors.

   e. As required to satisfy moment equilibrium of lateral force about the bottom of the soldier pile for soldier piles with permanent ground anchors.

3. **Soldier Pile Shaft Backfill**

   Specify controlled density fill (CDF, 145 pcf) for the full height of the soldier pile shaft when shafts are anticipated to be excavated and concrete placed in the dry.

   Specify pumpable lean concrete for the full height of the soldier pile shaft when shafts are anticipated to be excavated and concrete placed in the wet.
C. **Design of Lagging** – Lagging for soldier pile walls, with and without permanent ground anchors, may be comprised of timber, precast concrete, or steel. The expected service life of timber lagging is 20 years which is less than the 75 year service life of structures designed in accordance with AASHTO LRFD Bridge Design Specifications.

The Geotechnical Engineer will specify when lagging shall be designed for an additional 250 psf surcharge due to temporary construction load or traffic surcharge. The lateral pressure transferred from a moment slab shall be considered in the design of soldier pile walls and laggings.

1. **Temporary Timber Lagging**

   Temporary lagging is based on a maximum 36 month service life before a permanent fascia is applied over the lagging. The wall Design Engineer shall review the Geotechnical Recommendations or consult with the Geotechnical Engineer regarding whether the lagging may be considered as temporary as defined in Standard Specifications Section 6-16.3(6). Temporary timber lagging shall be designed by the contractor in accordance with Section 6-16.3(6)B of the Standard Specifications.

2. **Permanent Lagging**

   Permanent lagging shall be designed for 100% of the lateral load that could occur during the life of the wall in accordance with AASHTO LRFD Bridge Design Specifications Sections 11.8.5.2 and 11.8.6 for simple spans without soil arching. A reduction factor to account for soil arching effects may be used if permitted by the Geotechnical Engineer.

   Timber lagging shall be designed in accordance with AASHTO LRFD Bridge Design Specifications Section 8.6. The size effect factor ($CF_b$) should be considered 1.0, unless a specific size is shown in the wall plans. The wet service factor ($CM_b$) should be considered 0.85 for a saturated condition at some point during the life of the lagging. The load applied to lagging should be applied at the critical depth. The design should include the option for the contractor to step the size of lagging over the height of tall walls, defined as walls over 15 feet in exposed face height.

   Timber lagging designed as a permanent structural element shall consist of treated Douglas Fir-Larch, grade No. 2 or better. Hem-fir wood species, due to the inadequate durability in wet condition, shall not be used for permanent timber lagging. Permanent lagging is intended to last the design life cycle (75 years) of the wall. Timber lagging does not have this life cycle capacity but can be used when both of the following are applicable:

   a. The wall will be replaced within a 20 year period or a permanent fascia will be added to contain the lateral loads within that time period.

       And,

   b. The lagging is visible for inspections during this life cycle.
D. Design of Fascia Panels

Cast-in-place concrete fascia panels shall be designed as a permanent load carrying member in accordance with AASHTO LRFD Bridge Design Specifications Section 11.8.5.2. For walls without permanent ground anchors the minimum structural thickness of the fascia panels shall be 9 inches. For walls with permanent ground anchors the minimum structural thickness of the fascia panels shall be 14 inches. Architectural treatment of concrete fascia panels shall be indicated in the plans.

Concrete strength shall not be less than 4,000 psi at 28 days. The wall is to extend 2 feet minimum below the finish ground line adjacent to the wall.

When concrete fascia panels are placed on soldier piles, a generalized detail of lagging with strongback (see Appendix 8.1-A3-5) shall be shown in the plans. This information will assist the contractor in designing formwork that does not overstress the piles while concrete is being placed.

Precast concrete fascia panels shall be designed to carry 100% of the load that could occur during the life of the wall. When timber lagging (including pressure treated lumber) is designed to be placed behind a precast element, conventional design practice is to assume that lagging will eventually fail and the load will be transferred to the precast panel. If another type of permanent lagging is used behind the precast fascia panel, then the design of the fascia panel will be controlled by internal and external forces other than lateral pressures from the soil (weight, temperature, Seismic, Wind, etc.). The connections for precast panels to soldier piles shall be designed for all applicable loads and the designer should consider rigidity, longevity (to resist cyclic loading, corrosion, etc.), and load transfer.

See Section 5.1.1 of this manual for use of shotcrete in lieu of cast-in-place conventional concrete for soldier pile fascia panels.

8.1.6 Design of Structural Earth Walls

A. Pre-approved Proprietary Structural Earth Walls

Structural earth (SE) wall systems meeting established WSDOT design and performance criteria have been listed as “pre approved” by the Bridge and Structures Office and the Materials Laboratory Geotechnical Branch. A list of current pre approved proprietary wall systems and their limitations is provided in Appendix 15 D of the Geotechnical Design Manual M 46-03. For the SE wall shop drawing review procedure, see Chapter 15 of the Geotechnical Design Manual M 46-03.

B. Non Pre-approved Proprietary Structural Earth Walls

Structural earth walls that exceed the limitations as provided in Appendix 15-D of the Geotechnical Design Manual M 46-03 are considered to be non pre-approved. Use of non pre-approved structural earth walls shall require the approval of the State Geotechnical Engineer and the State Bridge and Structures Engineer.
8.1.7 **Design of Standard Plan Geosynthetic Walls**

Details for construction are given in the *Standard Plans Manual M 21 0*, Section D.

The width “w” of the precast panels as defined in Standard Plan D-3.11 is to be shown on the plan sheets and should be selected considering the architectural requirements for the wall.

8.1.8 **Design of Soil Nail Walls**

Soil nail walls shall be designed in accordance with the FHWA Publication FHWA IF 03 017 “Geotechnical Engineering Circular No. 7 Soil Nail Walls” March 2003. The seismic design parameters shall be determined in accordance with the most current edition of the AASHTO Guide Specifications for LRFD Seismic Bridge Design. Typical soil nail wall details are provided in Appendix 8.1.

8.1.9 **Miscellaneous Items**

A. **Architectural Treatment**

   Approval by the State Bridge and Structures Architect is required on all retaining wall aesthetics, including finishes, materials, and configuration

B. **Scour**

   The foundation for all walls constructed along rivers and streams shall be evaluated during design by the Hydraulics Engineer for scour in accordance with AASHTO LRFD Bridge Design Specifications Section 2.6.4.4.2. The wall foundation shall be located at least 2 feet below the scour depth in accordance with the *Geotechnical Design Manual M 46-03* Section 15.4.5.

C. **Fall Protection**

   For retaining walls with exposed wall heights of 10 feet or more, fall protection shall be provided in accordance with WAC 296-155-24510 or WAC 296-155-505 and as described in the *Design Manual M 22-01*, Chapter 730.

D. **Drainage**

   Drainage features shall be detailed in the Plans.

   Permanent drainage systems shall be provided to prevent hydrostatic pressures developing behind the wall. A cut that slopes toward the proposed wall will invariably encounter natural subsurface drainage. Vertical chimney drains or prefabricated drainage mats can be used for normal situations to collect and transport drainage to a weep hole or pipe located at the base of the wall. Installing horizontal drains to intercept the flow at a distance well behind the wall may control concentrated areas of subsurface drainage (see WSDOT *Geotechnical Design Manual M 46-03* Chapter 15).

   All reinforced concrete retaining walls shall have 3-inch diameter weepholes located 6 inches above final ground line and spaced about 12 feet apart. In case the vertical distance between the top of the footing and final ground line is greater than 10 feet, additional weepholes shall be provided 6 inches above the top of the footing. No weepholes are necessary in cantilever wingwalls.
Weepholes can get clogged up or freeze up, and the water pressure behind the wall may start to increase. In order to keep the water pressure from building, it is important to have well draining gravel backfill and underdrains. Appropriate details must be shown in the Plans.

No underdrain pipe or gravel backfill for drains is necessary behind cantilever wingwalls. A 3 foot minimum vertical layer of gravel backfill shall be placed behind the cantilever wingwalls and shown in the Plans. Backfill material shall be included with the civil quantities (not the bridge quantities). If it is necessary to excavate existing material for the backfill, then this excavation shall be a part of the bridge quantities for “Structure Excavation Class A Incl. Haul”.

E. Joints

Odd panels for all types of walls shall normally be made up at the ends of the walls.

Every joint in the wall shall provide for expansion. For cast-in-place construction, a minimum of ½ inch premolded filler should be specified in the joints. A compressible back-up strip of closed-cell foam polyethylene or butyl rubber with a sealant on the front face is used for precast concrete walls.

For cantilevered and gravity walls constructed without a traffic barrier attached to the top, expansion joint spacing should be a maximum of 24 feet on centers. For cantilevered and gravity walls constructed with a traffic barrier attached to the top, expansion joint spacing should be at 48 feet on centers or that determined for adequate distribution of the traffic collision loading.

For counterfort walls, expansion joint spacing should be a maximum of 32 feet on centers.

For soldier pile and soldier pile tieback walls with concrete fascia panels, expansion joint spacing should be 24 to 32 feet on centers.

For precast units, the length of the unit depends on the height and weight of each unit. No joints other than construction joints shall be used in footings except at bridge abutments and where substructure changes such as spread footing to pile footing occur. In these cases, the footing shall be interrupted by a ½ inch premolded expansion joint through both the footing and the wall. The maximum spacing of construction joints in the footing shall be 120 feet. The footing construction joints should have a 6-inch minimum offset from the expansion joints in the wall.
F. Detailing of Standard Reinforced Concrete Retaining Walls

1. In general, the “H” dimension shown in the retaining wall Plans should be in foot increments. Use the actual design “H” reduced to the next lower even foot for dimensions up to 3 inches higher than the even foot.

   Examples:  
   - Actual height = 15′-3”↑, show “H” = 15’ on design plans  
   - Actual height > 15′-3”↑, show “H” = 16’ on design plans

   For walls that are not of a uniform height, “H” should be shown for each segment of the wall between expansion joints or at some other convenient location. On walls with a steep slope or vertical curve, it may be desirable to show 2 or 3 different “H” dimensions within a particular segment. The horizontal distance should be shown between changes in the “H” dimensions.

   The value for “H” shall be shown in a block in the center of the panel or segment. See Example, Figure 9.4.4-1.

2. Follow the example format shown in Figure 8.1.4-1.


4. Wall dimensions shall be determined by the designer using the Standard Plans.

5. Do not show any details given in the Standard Plans.


7. Do not detail reinforcing steel, unless it deviates from the Standard Plans.

8. For pile footings, use the example format with revised footing sizes, detail any additional steel, and show pile locations. Similar plan details are required for footings supported by shafts.
8.2 Noise Barrier Walls

8.2.1 General

Design of noise barrier walls shall be based on the requirements and guidance cited herein and in the current AASHTO LRFD Bridge Design Specifications, AASHTO Guide Specifications for LRFD Seismic Bridge Design, AASHTO LRFD Bridge Construction Specifications, WSDOT General & Bridge Special Provisions and the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction unless otherwise cited herein.

Details for construction of the Standard Plan Noise Barrier Walls may be found in Standard Plan D-2.04 through D-2.68 and Standard Specifications Section 6-12.

Noise barrier walls are primarily used in urban or residential areas to mitigate noise or to hide views of the roadway. Common types, as shown in the Standard Plans, include cast-in-place concrete panels (with or without traffic barrier), precast concrete panels (with or without traffic barrier), and masonry blocks. The State Bridge and Structures Architect should be consulted for wall type selection.

8.2.2 Loads

Noise barrier walls and their components shall be designed for all applicable loads defined in the current AASHTO LRFD Bridge Design Specifications, Chapter 3.

Wind loads and on noise barriers shall be as specified in Chapter 3.

Seismic load shall be as follows:

\[
\text{Seismic Dead Load} = A_s \times f \times D
\]

(8.2-1)

In which:

\[
A_s \times f \geq 0.10
\]

(8.2-2)

Where:

- \(A_s\) = Peak seismic ground acceleration coefficient modified by short-period site factor
- \(D\) = Dead load of the noise barrier and its components
- \(f\) = Dead load coefficient as specified in Table 8.2-1.

<table>
<thead>
<tr>
<th>Location</th>
<th>(f)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monolithic connection</td>
<td>1.0</td>
</tr>
<tr>
<td>Monolithic connection on bridges</td>
<td>2.5</td>
</tr>
<tr>
<td>Connection of precast wall to bridge barrier</td>
<td>8.0</td>
</tr>
<tr>
<td>Connection of precast wall to retaining wall or moment slab barrier</td>
<td>5.0</td>
</tr>
</tbody>
</table>

Dead Load Coefficient, \(f\)

Table 8.2-1
8.2.3 Design

A. Standard Plan Noise Barrier Walls

1. Noise Barrier Walls detailed in Standard Plans D-2.04 through D-2.34, D-2.42 through D-2.44, D-2.48 through D-2.68 have been designed in accordance with the following criteria.
   b. The seismic design was based on a PGA of 0.35g which corresponds to a peak bedrock acceleration of 0.3g with an amplification factor of 1.18 for stiff soil.
   c. The WSDOT Design Manual M 22 01, Chapter 740 tabulates the design wind speeds and various exposure conditions used to determine the appropriate wall type.
   d. The design parameters used in the standard plan noise wall foundation design are summarized in the WSDOT Geotechnical Design Manual M 46-03, Chapter 17.

2. Noise Barrier Walls detailed in Standard Plans D-2.36 and D-2.46 have been designed in accordance with the requirements of the AASHTO LRFD Bridge Design Specifications, 6th Edition 2012 and interims through 2013, and the requirements and guidance cited herein:
   a. Load factors and load combinations for the design of all structural elements are in accordance with AASHTO LRFD Tables 3.4.1-1 and 3.4.1-2.
   b. Seismic design is in accordance with AASHTO LRFD Article 3.10.2.1-General Procedure, considering site classes B, C, D, and E and the following:
      i. Peak seismic ground acceleration coefficient on Rock (Site Class B).
         1. PGA = 0.45g for Western Washington
         2. PGA = 0.19g for Eastern Washington
      ii. Horizontal response spectral acceleration coefficient at 0.2-sec period on rock (Site Class B).
         1. $S_s = 1.00$ for Western Washington
         2. $S_s = 0.43$ for Eastern Washington
      iii. Horizontal response spectral acceleration coefficient at 1.0-sec period on rock (Site Class B).
         1. $S_1 = 0.33$ for Western Washington
         2. $S_1 = 0.15$ for Eastern Washington
      iv. Modal analysis was performed for the first four periods. The elastic seismic response coefficient $C_{sm}$ was computed for each modal period in accordance with AASHTO LRFD Article 3.10.4.2 and all four $C_{sm}$ coefficients were combined through the SRSS method.
v. The resultant seismic force is considered to act at a height of 0.71H above the top of the shaft, where H is the total height measured from the top of the panel to the top of the shaft.

c. Wind loads are computed in accordance with AASHTO LRFD Article 15.8.2 considering surface conditions characterized as “Sparse Suburban”. The 50 year return period maximum wind velocity, as determined from AASHTO LRFD Figure 15.8.2-1, is 100 mph for Western Washington and 80 mph for Eastern Washington.

d. Drilled shaft foundations is designed for earth pressure distributions as shown in AASHTO LRFD Figure 3.11.5.10-1 considering the following:

i. Shaft depth, D1
   1. 2H:1V fore-slope and a flat backslope
   2. Angle of internal friction = 32 degrees
   3. Soil unit weight = 125pcf
   4. Corresponding $K_p = 1.5$
   5. Corresponding $K_a = 0.28$

ii. Shaft depth, D2
   1. 2H:1V fore-slope and a flat backslope
   2. Angle of internal friction = 38 degrees
   3. Soil unit weight = 125pcf
   4. Corresponding $K_p = 2.3$
   5. Corresponding $K_a = 0.22$

iii. The passive earth pressure distribution was assumed to start at the finished grade. However, the uppermost two feet of passive earth pressure was neglected, resulting in a trapezoidal passive earth pressure distribution.

iv. In accordance with AASHTO LRFD Table 11.5.7-1 and Article 11.5.8, the resistance factor applied to the passive earth pressure is as follows:
   1. For the Strength Limit State, the resistance factor is taken as 0.75.
   2. For the Extreme Event Limit State, the resistance factor is taken as 1.0.

e. Barrier is designed for minimum Test Level 4 (TL-4) vehicular collision loads in accordance to AASHTO LRFD Article 13, and shafts are designed for an equivalent static load of 10 kips.

f. Barrier shown in the Standard Plan could be either precast or cast-in-place, and the barrier shape could be Type F (shown), single slope or other TL-3 and TL-4 barrier systems.
B. Non-Standard Noise Barrier Walls

Noise barrier walls containing design parameters which exceed those used in the standard noise barrier wall design are considered to be non-standard.

All noise barrier walls which will be mounted on existing structures, supported by existing structures, or constructed as part of a new structure are considered to be non-standard and shall be evaluated by the Bridge and Structures Office and the Geotechnical Office.

1. Noise Barrier Walls on Bridges and Retaining Walls
   a. For noise barrier walls located on bridges, the total height, as measured from the top of bridge deck to the top of the noise barrier wall, shall be limited to 8’-0”.
   b. For noise barrier walls located on retaining walls, the total height, as measured from the top of roadway to the top of the noise barrier wall, shall be limited to 14’-0”.
   c. Cast-in-place noise barrier walls constructed with self-consolidating concrete and precast concrete noise barrier walls and shall conform to the following requirements.
      - Minimum thickness of the wall stem shall be 7 inches.
      - Minimum concrete clear cover on each face shall be 1½ inches.
      - Both vertical and horizontal reinforcement shall be placed in two parallel layers.
   d. Cast-in-place noise barrier walls constructed with conventional concrete shall conform to the following requirements.
      - Minimum thickness of the wall stem shall be 8 inches.
      - Minimum concrete clear cover on each face shall be 1½ inches.
      - Both vertical and horizontal reinforcement shall be placed in two parallel layers.
      - Minimum clear distance between parallel layers of reinforcement shall be 2 ½ inches.
8.3 Miscellaneous Buried Structures

8.3.1 General

Miscellaneous buried structures consist of metal pipe, structural plate pipe, long-span structural plate, deep corrugated plate, reinforced concrete pipe, cast-in-place reinforced concrete and precast concrete arch, box and elliptical structures, thermoplastic pipe, and fiberglass pipe. All buried structures shall comply with the current edition of the AASHTO LRFD Specifications for minimum service life of 75 years.

The 26 feet span limit is based on the WSDOT past practice for shipping, handling and installation of buried concrete structures.

General NBI requirements are that the Bridge Preservation Office has a load rating on file for every structure with span length greater than 20 feet.

A. General Requirements

Buried structure systems considered herein are: metal pipe, structural plate pipe, long-span structural plate, deep corrugated plate, reinforced concrete pipe, cast-in-place reinforced concrete and precast concrete arch, box and elliptical structures, thermoplastic pipe, and fiberglass pipe. Due to lack of satisfactory performance, buried box culverts constructed of aluminum or steel structural plate shall not be used for any types of buried structures regardless of span lengths or usage.

The span length for buried structures regardless of type and materials is limited to 26 feet. On a case-by-case basis, spans beyond 26 feet may be used upon approval of the State Hydraulics, Geotechnical, and Bridge Design Engineers, provided sufficient supporting project specific justification and documentation regarding past performance and compliance with the AASHTO LRFD minimum service life of 75 years is provided. Performance monitoring instrumentations may be required for buried metal structures with spans beyond 26 feet. The span length for buried structures is the widest opening measured along centerline roadway.

For Design-Build projects all buried structures are required to be sealed and signed by licensed Hydraulics, Geotechnical and Structural Engineers.

B. Design Requirements

The design of buried structures shall be in accordance with the requirement of current edition of AASHTO LRFD Bridge Design Specifications Section 12, unless otherwise required in the project-specific criteria. The decrease in live load effect due to increase in fill depth shall be considered in both design and load rating of buried structures. The requirement of BDM Section 3.5 for inclusion of live load in Extreme Event-I load combination is applicable. Foundations for buried structures shall be designed and detailed in accordance with Bridge Design and Geotechnical Manuals and shall include the effects of potential scour.

Wingwalls and headwalls for buried structures shall be designed in accordance with the current versions of WSDOT Geotechnical Design Manual M 46-03, AASHTO LRFD Bridge Design Specifications, Chapter 11, and the BDM.
C. **Seismic Design Requirements**

The seismic design need not be considered for buried structures with span lengths of 20 feet or less. Structures with span lengths as defined above. Buried structures greater than 20 feet shall be designed for seismic effects. Seismic design of buried structures shall confirm to Chapter 13 Seismic Considerations in FHWA publication FHWA-NHI-10-034, Technical Manual for Design and Construction of Road Tunnels – Civil Elements. The seismic effects of transient racking/ovaling deformations on culverts and pipe structures must be considered in addition to the normal load effects from dead loads of structural components, vertical and horizontal earth and water loads, and live load surcharges. The AASHTO LRFD Bridge Design Specifications Section 12.6.1 exemption from seismic loading shall not apply.

The ground motion attenuation as specified below shall be considered used for seismic design of buried structures.

<table>
<thead>
<tr>
<th>Depth to Top of Buried Structure, feet</th>
<th>Ratio of Ground Motion at Buried Structure Depth to Motion at Ground Surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 20</td>
<td>1.0</td>
</tr>
<tr>
<td>20 to 50</td>
<td>0.9</td>
</tr>
<tr>
<td>50 to 100</td>
<td>0.8</td>
</tr>
<tr>
<td>&gt;100</td>
<td>0.7</td>
</tr>
</tbody>
</table>

**Ground Motion Attenuation with Depth**

*Table 1*

For buried structures, with span lengths more than 20 feet, the seismic effects of potential unstable ground conditions (e.g., liquefaction, liquefaction induced settlement, landslides, and fault dis-placements) on the function of the buried structures shall be considered, except liquefaction need not be considered if the liquefaction, landslides, or fault displacements do not cause life safety hazards. As a guideline, if the depth of fill on top of the structure is more than one-half the clear span along the skew, liquefaction induced settlement or local instability are not likely to cause life safety hazards.

The above provisions are the minimum seismic design requirements for conventional buried structures. Additional provisions may be specified, on a case-by-case basis, to achieve higher seismic performance criteria for essential or critical buried structures. Where such additional re-quirements are specified, they shall be site or project specific and are tailored to a particular structure type.

D. **Submittal Requirements**

The design calculations and detailed shop drawings of buried structures shall be submitted to the Bridge and Structures Office for review and approval. The submittal shall include load rating for all buried structures with span length beyond 20 feet. The load rating shall be in accordance with. The submittal shall include the geotechnical design parameters, hydraulic analysis, including scour depth, installation procedures, backfill materials, and compacting sequences. The adequacy of the buried structure for the required depth of fill shall be provided in the submittal. Final as-built plans shall be submitted to the Bridge and Structures Office for records.
The above provisions apply to buried structures composed of precast or cast-in-place concrete walls supporting the roadway embankment. Precast walls that are being supplied as part of the culvert systems could be preapproved if they meet the WSDOT design and detailing requirement, and the preapproval procedures.

### 8.3.2 Design

A. **Box Culverts** – Box culverts are four-sided rigid frame structures and are either made from cast-in-place (CIP) reinforced concrete or precast concrete. In the past, standardized box culvert plan details were shown in the WSDOT Standard Plans, under the responsibility of the Hydraulics Branch. These former Standard Plans have been deleted and are no longer available.

The precast concrete fabricators for the precast reinforced concrete box and split box culverts are responsible for the structural design and the preparation of shop plans. Precast reinforced concrete box and split box culverts, constructed in accordance with the current WSDOT General Special Provisions (GSP’s) for these structures, shall be designed under AASHTO LRFD Bridge Specifications. The bridge designer reviewing the project will be responsible for reviewing the fabricator’s design calculations and details with consultation from the Construction Support Unit. Under the current GSP, precast reinforced concrete box and split box culverts are limited to spans of 26 feet or less. However, in special cases it may be necessary to allow longer spans, with the specific approval of the Bridge and Structures Office.

B. **Precast Reinforced Concrete Three-Sided Structures** – Precast reinforced concrete three-sided structures are patented or trademarked rigid frame structures made from precast concrete. Some fabricators of these systems are: Utility Vault Company, Central Pre-Mix Prestress Company, and Bridge Tek, LLC. These systems require a CIP concrete or precast footing that must provide sufficient resistance to the horizontal reaction or thrust at the base of the vertical legs.

The precast concrete fabricators are responsible for the structural design and the preparation of shop plans. Precast reinforced concrete three sided structures, constructed in accordance with the current WSDOT General Special Provisions (GSP’s) for these structures, shall be designed under AASHTO LRFD Bridge Specifications. The bridge designer reviewing the project will be responsible for reviewing the fabricator’s design calculations and details with consultation from the Construction Support Unit. Under the current GSP, precast reinforced concrete three sided structures are limited to spans of 26 feet or less. However, in special cases it may be necessary to allow longer spans, with the specific approval of the Bridge and Structures Office.

C. **Detention Vaults** – Detention vaults are used for stormwater storage and are to be watertight. These structures can be open at the top like a swimming pool, or completely enclosed and buried below ground. Detention vaults shall be designed by the AASHTO LRFD Bridge Design Specification and the following: Seismic design effects shall satisfy the requirements of ACI 350.3-06 “Seismic Design of Liquid-Containing Concrete Structures.” Requirements for Joints and jointing shall satisfy the requirements of ACI 350-06. Two references for tank design are

The geotechnical field investigations and recommendations shall comply with the requirements given in 8.16 of the WSDOT M 46-03. In addition to earth pressures, water tables, seismic design, and uplift, special consideration should be given to ensure differential settlement either does not occur or is included in the calculations for forces, crack control and water stops.

Buoyant forces from high ground water conditions should be investigated for permanent as well as construction load cases so the vault does not float. Controlling loading conditions may include: backfilling an empty vault, filling the vault with stormwater before it is backfilled, or seasonal maintenance that requires draining the vault when there is a high water table. In all Limit States, the buoyancy force (WA) load factor shall be taken as $\gamma_{WA} = 1.25$ in AASHTO LRFD Table 3.4.1-1. In the Strength Limit State, the load factors that resist buoyancy ($\gamma_{DC}, \gamma_{DW}, \gamma_{ES}$, etc.) shall be their minimum values, per AASHTO LRFD Table 3.4.1-2 and the entire vault shall be considered empty. During the vault construction, the water table shall be taken as the seal vent elevation or the top of the vault, if open at the top. In this case the load factors that resist buoyancy shall be their minimum values, except where specified as a construction load, per AASHTO LRFD Section 3.4.2. In certain situations tie-downs may be required to resist buoyancy forces. The resisting force ($R_n$) and resistance factors ($\phi$) for tie-downs shall be provided by the Geotechnical Engineers. The buoyancy check shall be as follows:

For Buoyancy without tie-downs:

$\left( \frac{R_{RES}}{R_{UPLIFT}} \right) \geq 1.0$

For Buoyancy with tie-downs:

$\left( \frac{R_{RES}}{[R_{UPLIFT} + \phi R_n]} \right) \geq 1.0$

Where:

$R_{RES} = \left| \gamma_{DC} DC + \gamma_{DW} DW + \gamma_{ES} ES + \gamma_i Q_1 \right|$

$R_{UPLIFT} = \left| \gamma_{WA} WA \right|$

ACI 350-06 has stricter criteria for cover and spacing of joints than the AASHTO LRFD Specifications. Cover is not to be less than 2 inches (ACI 7.7.1), no metal or other material is to be within 1½ inches from the formed surface, and the maximum bar spacing shall not exceed 12 inches (ACI 7.6.5). Crack control criteria is per AASHTO LRFD 5.7.3.4 with $\gamma_e = 0.5$ (in order to maintain a crack width of 0.0085 inches, per the commentary of 5.7.3.4).

Joints in the vault’s top slab, bottom slab and walls shall allow dissipation of temperature and shrinkage stresses, thereby reducing cracking. The amount of temperature and shrinkage reinforcement is a function of reinforcing steel grade "and length between joints (ACI Table 7.12.2-1). All joints shall have a shear key and a continuous and integral PVC waterstop with a 4-inch minimum width. The purpose of the waterstop is to prevent water infiltration and exfiltration. Joints having welded shear connectors with grouted keyways shall use details from WSDOT Precast Prestressed Slab Details or approved equivalent, with weld ties
spaced at 4'-0" on center. Modifications to the above joints shall be justified with calculations. Calculations shall be provided for all grouted shear connections. The width of precast panels shall be increased to minimize the number of joints between precast units.

For cast-in-place walls in contact with liquid that are over 10’ in height, the minimum wall thickness is 12". This minimum thickness is generally good practice for all external walls, regardless of height, to allow for 2 inches of cover as well as space for concrete placement and vibration.

After the forms are placed, the void left from the form ties shall be coned shaped, at least 1 inch in diameter and 1½ inches deep, to allow proper patching.

Detention vaults that need to be located within the prism supporting the roadway are required to meet the following maintenance criteria. A by-pass piping system is required. Each cell in the vault shall hold no more than 6,000 gallons of water to facilitate maintenance and cleanout operations. Baffles shall be water tight. Access hatches shall be spaced no more than 50 feet apart. There shall be an access near both the inlet and the outfall. These two accesses shall allow for visual inspection of the inlet and outfall elements, in such a manner that a person standing on the ladder, out of any standing water, will be in reach of any grab handles, grates or screens. All other access hatches shall be over sump areas. All access hatches shall be a minimum 30 inch in diameter, have ladders that extend to the vault floor, and shall be designed to resist HS-20 wheel loads with applicable impact factors as described below.

Detention vaults that need to be located in the roadway shall be oriented so that the access hatches are located outside the traveled lanes. Lane closures are usually required next to each access hatch for maintenance and inspection, even when the hatches are in 12'-0" wide shoulders.

A 16 kip wheel load having the dynamic load allowance for deck joints, in AASHTO LRFD Table 3.6.2.1-1, shall be applied at the top of access hatches and risers. The load path of this impact force shall be shown in the calculations.

Minimum vault dimensions shall be 4'-0" wide and 7'-0" tall; inside dimensions.

Original signed plans of all closed top detention vaults with access shall be forwarded to the Bridge Plans Engineer in the Bridge Project Unit (see Section 12.4.10.B of this manual). This ensures that the Bridge Preservation Office will have the necessary inventory information for inspection requirements. A set of plans must be submitted to both the WSDOT Hydraulics Office and the Regional WSDOT Maintenance Office for plans approval.

D. Metal Pipe Arches – Soil pH should be investigated prior to selecting this type of structure. Metal Pipe arches are not generally recommended under high volume highways or under large fills.

Pipe arch systems are similar to precast reinforced concrete three sided structures in that these are generally proprietary systems provided by several manufacturers, and that their design includes interaction with the surrounding soil. Pipe arch systems shall be designed in accordance with the AASHTO Standard Specifications for Highway Bridges, and applicable ACI design and ASTM material specifications.
E. Tunnels – Tunnels are unique structures in that the surrounding ground material is the structural material that carries most of the ground load. Therefore, geology has even more importance in tunnel construction than with above ground bridge structures. In short, geotechnical site investigation is the most important process in planning, design and construction of a tunnel. These structures are designed in accordance with the *AASHTO LRFD Bridge Design Specifications*.

Tunnels are not a conventional structure, and estimation of costs is more variable as size and length increase. Ventilation, safety access, fire suppression facilities, warning signs, lighting, emergency egress, drainage, operation and maintenance are extremely critical issues associated with the design of tunnels and will require the expertise of geologists, tunnel experts and mechanical engineers.

For motor vehicle fire protection, a standard has been produced by the National Fire Protection Association. This document, *NFPA 502 – Standard for Road Tunnels, Bridges, and Other Limited Access Highways*, uses tunnel length to dictate minimum fire protection requirements:

- 300 feet or less: no fire protection requirements
- 300 to 800 feet: minor fire protection requirements
- 800 feet or more: major fire protection requirements

Some recent WSDOT tunnel projects are:

**I-90 Mt. Baker Ridge Tunnel Bore**  
Contract: 3105  
Bridge No: 90/24N

This 1500 foot long tunnel is part of the major improvement of Interstate 90. Work was started in 1983 and completed in 1988. The net interior diameter of the bored portion, which is sized for vehicular traffic on two levels with a bike/pedestrian corridor on the third level, is 63.5 feet. The project is the world’s largest diameter tunnel in soft ground, which is predominantly stiff clay. Construction by a stacked-drift method resulted in minimal distortion of the liner and insignificant disturbance at the ground surface above.

**Jet I-5 SR 526 E-N Tunnel Ramp**  
Contract: 4372  
Bridge No: 526/22E-N

This 465 foot long tunnel, an example of the cut and cover method, was constructed in 1995. The interior dimensions were sized for a 25 foot wide one lane ramp roadway with a vertical height of 18 feet. The tunnel was constructed in three stages. 3 and 4 foot diameter shafts for the walls were placed first, a 2 foot thick cast-in-place top slab was placed second and then the tunnel was excavated, lined and finished.

**I-5 Sleater-Kinney Bike/Ped. Tunnel**  
Contract: 6031  
Bridge No: 5/335P

This 122 foot long bike and pedestrian tunnel was constructed in 2002 to link an existing path along I-5 under busy Sleater-Kinney Road. The project consisted of precast prestressed slab units and soldier pile walls. Construction was staged to minimize traffic disruptions.
8.3.3 References

### Summary of Design

#### Appendix 8.1-A1 Specification Requirements for Walls

<table>
<thead>
<tr>
<th>Wall Types</th>
<th>Design Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pre-Approved Proprietary Structural Earth Walls</strong></td>
<td>Design shall be based on current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications for projects initiated after October 1, 2010, WSDOT Geotechnical Design Manual (GDM) and WSDOT Bridge Design Manual (BDM).</td>
</tr>
<tr>
<td><strong>General</strong></td>
<td>AASHTO LRFD Bridge Design Specifications 1000 year Seismic Acceleration map.</td>
</tr>
<tr>
<td><strong>Seismic</strong></td>
<td>AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load, unless otherwise specified in the Contract Plans or Contract Special Provisions.</td>
</tr>
<tr>
<td><strong>Traffic Barrier</strong></td>
<td>AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load, unless otherwise specified in the Contract Plans or Contract Special Provisions.</td>
</tr>
<tr>
<td><strong>Non-Preapproved Proprietary Structural Earth Walls</strong></td>
<td>Design shall be based on current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM.</td>
</tr>
<tr>
<td><strong>General</strong></td>
<td>For Standard Plan Geosynthetic walls use Standard Plan D-3.15, D-3.16, or D-3.17 for traffic barriers. Special design barriers to be constructed on Standard Plan or Non-Standard Geosynthetic Walls shall be designed in accordance with the WSDOT Bridge Design Manual and the AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load.</td>
</tr>
<tr>
<td><strong>Seismic</strong></td>
<td>AASHTO LRFD Bridge Design Specifications 1000 year Seismic Acceleration map.</td>
</tr>
<tr>
<td><strong>Traffic Barrier</strong></td>
<td>WSDOT Bridge Design Manual and the AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load. Load from top of barrier is distributed over 48 ft at the base of the wall.</td>
</tr>
<tr>
<td><strong>General</strong></td>
<td>Design shall be based on current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM.</td>
</tr>
<tr>
<td><strong>Seismic</strong></td>
<td>AASHTO LRFD Bridge Design Specifications 1000 year Seismic Acceleration map.</td>
</tr>
<tr>
<td><strong>Traffic Barrier</strong></td>
<td>WSDOT Bridge Design Manual and the AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load. Load from top of barrier is distributed at a 45 degree angle into the wall. Cross section of the wall shall be designed for TL-4 impact loading distributed over 48 ft at the base of the wall.</td>
</tr>
<tr>
<td><strong>Soldier Pile Walls</strong></td>
<td>Design shall be based on current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM.</td>
</tr>
<tr>
<td><strong>General</strong></td>
<td>AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load. Load from top of barrier is distributed downward into the wall spreading at a 45 degree angle.</td>
</tr>
<tr>
<td><strong>Seismic</strong></td>
<td>AASHTO LRFD Bridge Design Specifications 1000 year Seismic Acceleration map.</td>
</tr>
<tr>
<td><strong>Traffic Barrier</strong></td>
<td>AASHTO LRFD Bridge Design Specifications 1000 year Seismic Acceleration map.</td>
</tr>
<tr>
<td>Wall Types</td>
<td>Design Specifications</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Seismic</td>
<td>Current Standard Plans D-2.04 through D-2.34, D-2.42, D-2.44, and D-2.48 through D-2.68 are designed in accordance with AASHTO Guide Specifications for Structural Design of Sound Barriers – 1989 &amp; Interims. Standard Plans D-2.36 and D-2.46 are designed in accordance with AASHTO LRFD Bridge Design Specifications 1000 year map design acceleration.</td>
</tr>
<tr>
<td>Non-Standard Noise Barrier Walls General</td>
<td>Design shall be based on current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM.</td>
</tr>
<tr>
<td>Seismic</td>
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<tr>
<td>Traffic Barrier</td>
<td>WSDOT Bridge Design Manual and the AASHTO LRFD Bridge Design Specifications section a13.3 for Concrete Railings considering a minimum TL-4 impact load.</td>
</tr>
<tr>
<td>Soil Nail Walls General</td>
<td>All soil nail walls and their components shall be designed using the publication &quot;Geotechnical Engineering Circular No. 7&quot; FHWA-IF-03-017. The Geotechnical Engineer completes the internal design of the soil nail wall and provides recommendations for nail layout. The structural designer will layout the nail pattern. The geotechnical engineer will review the nail layout to insure compliance with the Geotechnical recommendations. The structural designer shall design the temporary shotcrete facing as well as the permanent structural facing, including the bearing plates, and shear studs. The upper cantilever of the facing that is located above the top row of nails shall be designed in accordance with current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM.</td>
</tr>
<tr>
<td>Seismic</td>
<td>AASHTO LRFD Bridge Design Specifications 1000 year map design acceleration.</td>
</tr>
<tr>
<td>Traffic Barrier</td>
<td>Moment slab barrier shall be designed in accordance with the WSDOT Bridge Design Manual and the AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load.</td>
</tr>
<tr>
<td>Non Standard Non Proprietary Walls Gravity Blocks, Gabion Walls General</td>
<td>Design shall be based on current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM.</td>
</tr>
<tr>
<td>Seismic</td>
<td>AASHTO LRFD Bridge Design specifications 1000 year map design acceleration.</td>
</tr>
<tr>
<td>Traffic Barrier</td>
<td>WSDOT Bridge Design Manual and the AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load.</td>
</tr>
</tbody>
</table>
TYPICAL CROSS SECTION

NOTES TO DESIGNER:
1. Specify type of architectural finish.
2. Specify type of wall facing. (Concrete panels or concrete blocks)
3. Minimum reinforcement length shall be as specified in the geotechnical recommendations and STD. SPEC. 6-13.3(2)A.
4. Minimum wall embedment as shown on this sheet is to be verified with the geotechnical recommendations.
### Soldier Pile Schedule

<table>
<thead>
<tr>
<th>FILE</th>
<th>SOLDIER PILE SIZE</th>
<th>DEPTH (Ft)</th>
<th>TOTAL LENGTH OF SOLDIER PILE</th>
<th>DIAMETER</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note to Designer:**

The schedule above represents the minimum amount of information to be shown on the plans. Additional information may be needed.

### Soldier Pile/Tieback Wall

**Elevation**

- **FINISHED GROUND LINE**
  - In front of retaining wall: EL. 121.3
  - Behind retaining wall: EL. 133.1

- **EXPANSION JOINT (TYP.)**
  - SEE DETAIL BR. SHT.

- **WEEP HOLES**
  - @ 12'-0" MAX.

- **SOLDIER PILES**
  - With P.G.A.: [Details]
  - Without P.G.A.: [Details]

**Total Wall Length:** 562'-10"
2" MIN. BEARING LENGTH. SHIM AS NECESSARY FOR FULL BEARING.

BACKFILL Voids Behind Laging With A FREE DRAINING MATERIAL AS APPROVED BY THE ENGINEER.

CHIP OUR SHAFT BACKFILL TO PLACE LAGING.

4'-0" WIDE STRIP OF PREFABRICATED DRAINAGE MAT (TYP.) CENTERED BETWEEN SOLDIER PILE FLANGES.

FRACURED FINISH WITH PIGMENTED SEALER.

REMAINING PORTION OF SOLDIER PILE SHAFT.

PLAN

SOLDIER PILE WALL

WITHOUT P. G. A.

PLAN

SOLDIER PILE WALL

WITH P. G. A.

Notes to Designer:
1. Depths and sizes shown are for example only. Fill in the table according to the earth pressure diagram and recommendations from the Geotechnical Services Branch, based on CFS timber design for permanent lagging.
2. Determine, if possible, the length of time that the wall lagging will be used as the primary structural member in the transverse direction before a permanent wall fascia is applied.
3. For walls with P.G.A., use a section size with a flange width bigger than or equal to HP12x53 or W12x65.
4. For walls without concrete fascia panels:
   a. Hem-fir timber lagging shall not be used.
   b. Douglas fir-larch, grade no. 2 or better, treated in accordance with section 9-09.3(1), shall be used and shall be specified in the plan sheets and Special Provisions.

TYPICAL SECTION

SHOWN FOR SOLDIER PILE WITH P.G.A. SHEARLE - PERMANENT GROUND ANCHOR

TIMBER LAGGING SIZES

<table>
<thead>
<tr>
<th>DEPTH (FT)</th>
<th>0 - 9</th>
<th>9 - 18</th>
<th>18 - 30</th>
<th>30 - 48</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 x</td>
<td>4 x</td>
<td>4 x</td>
<td>4 x</td>
<td>4 x</td>
</tr>
<tr>
<td>6 x</td>
<td>6 x</td>
<td>6 x</td>
<td>6 x</td>
<td>6 x</td>
</tr>
<tr>
<td>8 x</td>
<td>8 x</td>
<td>8 x</td>
<td>8 x</td>
<td>8 x</td>
</tr>
</tbody>
</table>

TIMBER LAGGING SIZES

<table>
<thead>
<tr>
<th>SIZE</th>
<th>DEPTH (FT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 x</td>
<td>0 - 9</td>
</tr>
<tr>
<td>6 x</td>
<td>9 - 18</td>
</tr>
<tr>
<td>8 x</td>
<td>18 - 30</td>
</tr>
</tbody>
</table>

LAGING IN SERVICE

36 MONTHS OR LONGER

USE CONTROL DENSITY FILL WHEN PLACED IN THE DRY. USE PUMPABLE LEAN CONCRETE WHEN PLACED IN THE WET.

Revised on: 5/1/2013

Washington State Department of Transportation

Soldier Pile/Retaining Wall

Details 2 of 3
ELEVATION - SOLDIER PILE
WITH P.G.A. THRU WEB

BEARING PLATE
BEARING PLATE SHALL BE DESIGNED BY THE CONTRACTOR AND SUBMITTED TO THE ENGINEER FOR APPROVAL IN ACCORDANCE WITH THE STANDARD SPECIFICATION SECTION 6-17.3(5).

Notes to Designer:
1. Plates must be checked for size and welds. Plates are used to replace flange steel removed for pipe installation.
2. Weld must be checked along web to pipe and plate to flange. Welds must be capable of transferring PDA loads and flexural loads.
3. Flange width bigger than or equal to HP12x53 or W12x65.

8"Ø XS PIPE
¾"Ø x 6" WELDED SHEAR STUDS @ 1'-0" ON CTR.
ANCHOR HEAD ASSEMBLY
1½ x 12 x 1'-6" BEARING PLATE
P.G.A.
TYP. BOTH ENDS OF 8" Ø PIPE
NOTE:
THE DOUBLE CORROSION PROTECTION SYSTEM AT THE ANCHOR HEAD SHALL BE DETAILED TO ALLOW A MINIMUM OF ± 2° VARIATION IN THE SLOPE OF THE SOIL ANCHOR FOR PLACEMENT TOLERANCE.

ALL ANCHORAGE COVERS SHALL BE BOLTED TO THE BEARING PLATES.
NOISE BARRIER ON BRIDGE

NOTES TO DESIGNER:
1. The details shown on this sheet have not been evaluated for AASHTO LRFD Bridge DesignSPEC, article 15.8.4, case 1.
NOTES:

1. ALL PIPE SHALL BE STEEL PIPE ASTM A53 GRADE B.
2. ALL STEEL PLATE SHALL BE ASTM A36.
3. WIRE ROPE SHALL CONFORM TO ASTM A603 WITH CL. A WEIGHT ZINC-COATED WIRES.
4. ALL PARTS EXCEPT WIRE ROPE SHALL BE HOT DIP GALVANIZED IN ACCORDANCE WITH AASHTO M111 OR M232 AFTER FABRICATION, UNLESS NOTED OTHERWISE.
5. SPELTER SOCKETS AND SOCKETING PROCEDURE SHALL BE AS PER SPELTER SOCKET MANUFACTURER.
6. WIRE ROPE SHALL BE INSTALLED TO 400 LBS TENSION LEAVING A TAKE UP OF 6" STILL AVAILABLE IN THE TURNBUCKLE.
7. EACH CONTINUOUS LENGTH OF CABLE SHALL HAVE A TURNBUCKLE AT ONE END ONLY AND BE ANCHORED TO END POST WITH BRACE AT BOTH ENDS.
8. INTERMEDIATE POSTS AND BRACES SHALL NOT TO BE INSTALLED ACROSS EXPANSION JOINT.
10. CABLE FENCE SHALL BE DARK BROWN FEDERAL COLOR 595 20045.
11. WIRE ROPE, SPELTER SOCKETS, TURNBUCKLES AND THEIR CONNECTIONS SHALL HAVE A MINIMUM BREAKING STRENGTH OF 26 KIPS.
12. ALL POSTS TO BE INSTALLED VERTICAL AND WIRE ROPE TO BE INSTALLED PARALLEL TO TOP OF WALL.
NOTES:

- ALL PIPE SHALL BE STEEL PIPE ASTM A53 GRADE B.
- ALL STEEL PLATE SHALL BE ASTM A 36.
- WIRE ROPE SHALL CONFORM TO ASTM A 603 WITH CL. A WEIGHT ZINC-COATED WIRES.
- ALL PARTS EXCEPT WIRE ROPE SHALL BE HOT DIP GALVANIZED IN ACCORDANCE WITH AASHTO M111 OR M232 AFTER FABRICATION, UNLESS NOTED OTHERWISE.
- SPELTER SOCKETS AND SOCKETING PROCEDURE SHALL BE AS PER SPELTER SOCKET MANUFACTURER.
- ALL POSTS TO BE INSTALLED VERTICAL ... PARALLEL TO TOP OF WALL.
- WIRE ROPE SHALL BE INSTALLED TO 400 LBS TENSION LEAVING A TAKE UP OF 6" STILL AVAILABLE IN THE TURNBUCKLE.
- EACH CONTINUOUS LENGTH OF CABLE SHALL HAVE A TURNBUCKLE AT ONE END ONLY AND BE ANCHORED TO END POST WITH BRACE AT BOTH ENDS.
- INTERMEDIATE POSTS AND BRACES SHALL NOT TO BE INSTALLED ACROSS EXPANSION JOINT.
- CABLE FENCE WAS DESIGNED FOR A 200 LB. LOAD ON THE TOP RAIL APPLIED IN ANY DIRECTION, AS REQUIRED BY WASHINGTON ADMINISTRATIVE CODE 296-155-24615.
- CABLE FENCE SHALL BE DARK BROWN FEDERAL COLOR 595 20045.
- WIRE ROPE, SPELTER SOCKETS, TURNBUCKLES AND THEIR CONNECTIONS SHALL HAVE A MINIMUM BREAKING STRENGTH OF 26 KIPS.