## Chapter 8  Walls and Buried Structures

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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 Geotechnical Design Manual 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 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 Walls

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

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.

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.

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, 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 Plans 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 or precast concrete lagging 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.

F. **Noise Barrier Walls** – 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. Design criteria for noise barrier walls are based on AASHTO’s *Guide Specifications for Structural Design of Sound Barriers*. Details of these walls are available in the Standard Plans D-2.04 to D-2.68. The Noise Barriers chapter of the WSDOT Design Manual M 22-01 tabulates the design wind speeds and various exposure conditions used to determine the appropriate wall type.

### 8.1.3 Design

A. **General** – All designs shall follow procedures as outlined in AASHTO LRFD 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.

B. **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).

C. 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 Extreme Event loading for vehicular collision must be analyzed. These loads are tabulated in LRFD 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 LRFD 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 of this manual.

D. Soldier Pile and Soldier Pile Tieback Walls

1. Permanent Ground Anchors (Tiebacks) – See AASHTO LRFD 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 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 Specification Section 6-17.3(8) and WSDOT Geotechnical Design Manual M 46-03).

a. 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.

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

c. 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).

2. **Permanent Ground Anchor Corrosion Protection** – The Geotechnical Engineer will specify the appropriate protection system; the two primary types are:

a. Simple Protection: The use of simple protection relies on Portland cement grout to protect the tendon, bar, or strand in the bond zone. The unbonded lengths are sheaths filled with anti-corrosion grease, heat shrink sleeves, and secondary grouting after stressing. Except for secondary grouting, the protection is usually in place prior to insertion of the anchor in the hole.

b. Double Protection: a corrugated PVC, high-density polyethylene, or steel tube accomplishes complete encapsulation of the anchor tendon. The same provisions of protecting the unbonded length for simple protection are applied to those for double protection.

3. **Design of Soldier Pile** – The soldier piles shall be designed for shear, bending, and axial stresses according to the latest AASHTO LRFD 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

a. Lateral Loads

   1. 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.

   2. 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.

   3. Passive earth pressure usually acts over three times the shaft diameter or pile spacing, whichever is smaller.
b. Depth of Embedment

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

1. 10 feet
2. As recommended by the Geotechnical Engineer of Record
3. As required for skin friction resistance and end bearing resistance.
4. 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.
5. As required to satisfy moment equilibrium of lateral force about the bottom of the soldier pile for soldier piles with permanent ground anchors.

4. 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.

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.

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 Section 6-16.3(6) of the Standard Specifications. Temporary timber lagging shall be designed by the contractor in accordance with Section 6-16.3(6)B of the Standard Specifications.

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 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 Section 8.6. The size effect factor \( C_F \) should be considered 1.0, unless a specific size is shown in the wall plans. The wet service factor \( C_M \) 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:

1. 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,
2. The lagging is visible for inspections during this life cycle.
5. **Design of Fascia Panels** – Cast-in-place concrete fascia panels shall be designed as a permanent load carrying member in accordance with AASHTO LRFD 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.

### 8.1.4 Miscellaneous Items

**A. 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 thickness of gravel backfill shall be shown in the Plans behind the cantilever wingwalls. 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”.

**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 Sec. 2.6.4.4.2. The wall foundation shall be located at least 2 feet below the scour depth in accordance with the WSDOT Geotechnical Design Manual M 46-03 Section15.4.5.
C. **Joints** – For cantilevered and gravity walls constructed without a traffic barrier attached to the top, 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, joint spacing should be a maximum of 48 feet on centers or that determined for adequate distribution of the traffic collision loading. For counterfort walls, joint spacing should be a maximum of 32 feet on centers. For soldier pile and soldier pile tieback walls with concrete fascia panels, 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. 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.

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.

D. **Architectural Treatment** – The type of surface treatment for retaining walls is decided on a project specific basis. Consult the State Bridge and Structures Architect during preliminary plan preparation for approval of all retaining wall finishes, materials and configuration. The wall should blend in with its surroundings and complement other structures in the vicinity.

E. **Shaft Backfill for Soldier Pile Walls** – Specify controlled density fill (CDF, 145pcf) for soldier pile shafts (full height) when shafts are anticipated to be excavated in the dry. When under water concrete placement is anticipated for the soldier pile shafts, specify pumpable lean concrete.

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 Miscellaneous Underground Structures

8.2.1 General

Miscellaneous underground structures consist of box culverts, precast reinforced concrete three-sided structures, detention vaults, and metal pipe arches.

Where miscellaneous underground structures pass under or support roadways and other structures, they shall be designed for seismic effects as follows:

- Seismic effects need not be considered for structures with span lengths of 20 feet or less.
- Seismic effects shall be considered for structures with span lengths more than 20 feet. The potential effects of unstable ground conditions (e.g., liquefaction, liquefaction induced settlement, landslides, ground motion attenuation with depth, and fault displacements) on the function of the underground structures shall be considered. The *AASHTO LRFD Bridge Design Specifications* Section 12.6.1 exemption from seismic loading shall not apply.

As with any structure, a geotechnical soils report with loading or pressure diagrams, settlement criteria, and ground water levels will be needed from the Materials Laboratory Geotechnical Office in order to complete the design. The requirement of BDM *Section 3.5* for inclusion of live load in Extreme Event-I load combination is applicable.

In addition to the *AASHTO LRFD Bridge Design Specifications*, the FHWA Publication No. FHWA-NHI-09-010 dated November 2008, *Technical Manual for Design and Construction of Road Tunnels Civil Elements*, may also be used as a design specification reference for the seismic design requirement.

8.2.2 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. Now box culvert design is standardized under applicable AASHTO material specifications, and design plans are not required in the PS&E. Box culverts shall be in accordance with ASTM C1433.

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 fabricators of systems which have received WSDOT pre-approval are specified in the GSP’s. 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. Several manufacturers advertise spans over 40 feet.

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 PCA publications *Rectangular Concrete Tanks*, Revised 5th Edition (1998) and *Design of Liquid-Containing Structures for Earthquake Forces* (2002).

The geotechnical field investigations and recommendations shall comply with the requirements given in 8.16 of the WSDOT *Geotechnical Design Manual* 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 ($W_A$) load factor shall be taken as $\gamma_{W_A} = 1.25$ 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:

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

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

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.

**Jct 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.

### 8.2.3 References

<table>
<thead>
<tr>
<th>Wall Types</th>
<th>Design Specifications</th>
</tr>
</thead>
</table>
| Pre-Approved Proprietary Structural Earth Walls| **General**  
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).  
**Seismic**  
AASHTO LRFD Bridge Design Specifications 1000 year map design acceleration.  
**Traffic**  
Moment slab barrier shall be designed in accordance with the WSDOT BDM and the 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.                                                                                                                                                                                                 |
<table>
<thead>
<tr>
<th>Wall Types</th>
<th>General</th>
<th>Design Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soldier Pile Walls With &amp; Without Tie-Backs</td>
<td>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></td>
<td>Seismic</td>
<td>AASHTO LRFD Bridge Design Specifications 1000 year map design acceleration.</td>
</tr>
<tr>
<td></td>
<td>Traffic Barrier</td>
<td>AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load. $F_t$ is distributed over $L$, at the top of barrier. Load from top of barrier is distributed downward into the wall spreading at a 45 degree angle.</td>
</tr>
<tr>
<td>Non-Standard Noise Barrier Walls</td>
<td>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></td>
<td>Seismic</td>
<td>AASHTO LRFD Bridge Design specifications 1000 year map design acceleration.</td>
</tr>
<tr>
<td></td>
<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</td>
<td>General</td>
<td>All soil nail walls and their components shall be designed using the publication “Geotechnical Engineering Circular No. 7” 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></td>
<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</td>
<td>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></td>
<td>Seismic</td>
<td>AASHTO LRFD Bridge Design specifications 1000 year map design acceleration.</td>
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<tr>
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<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

- Design Height, H
- Mesh Length = 70% H for single S.E. walls
- 6" x 1'-0" non-reinforced concrete leveling pad
- Precast concrete panels or precast concrete blocks
- Sew barrier to match barrier on bridge, reinforcement in sew barrier to be designed by manufacturer
- Precast barrier to match barrier on bridge.
Appendix A

Chapter 8

BRIDGE DESIGN MANUAL

AUGUST 2010

Soldier Pile/Tieback Wall

Elevation

GENERAL NOTES

1. ALL MATERIAL AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE WASHINGTON STATE DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS FOR ROAD, BRIDGE AND MUNICIPAL CONSTRUCTION - ENGLISH, DATED 2010, AND AMENDMENTS.


3. W SECTION STEEL SOLDIER PILES SHALL CONFORM TO ASTM A992. HP SECTION STEEL SOLDIER PILES SHALL CONFORM TO ASTM A572. SOLDIER PILES SHALL BE PAINTED TO THE LIMITS SHOWN IN THE PLANS IN ACCORDANCE WITH SECTION 6-16.3(4).

4. ALL DIMENSIONS ARE HORIZONTAL AND VERTICAL UNLESS OTHERWISE SHOWN.

5. PERMANENT GROUND ANCHOR LOCK OFF LOAD = 60 PERCENT OF FACTORED DESIGN LOAD.

6. EXISTING GROUND LINE IS APPROXIMATE AND SHALL BE VERIFIED BY THE CONTRACTOR IN THE FIELD.

7. ALL MATERIAL AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE WASHINGTON STATE DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS FOR ROAD, BRIDGE AND MUNICIPAL CONSTRUCTION - ENGLISH, DATED 2010, AND AMENDMENTS.


9. W SECTION STEEL SOLDIER PILES SHALL CONFORM TO ASTM A992. HP SECTION STEEL SOLDIER PILES SHALL CONFORM TO ASTM A572. SOLDIER PILES SHALL BE PAINTED TO THE LIMITS SHOWN IN THE PLANS IN ACCORDANCE WITH SECTION 6-16.3(4).

10. UNLESS OTHERWISE SHOWN IN THE PLANS, THE CONCRETE COVER MEASURED FROM THE FACE OF THE CONCRETE TO THE FACE OF ANY REINFORCING STEEL SHALL BE 1½".

11. ALL DIMENSIONS ARE HORIZONTAL AND VERTICAL UNLESS OTHERWISE SHOWN.

12. EXISTING GROUND LINE IS APPROXIMATE AND SHALL BE VERIFIED BY THE CONTRACTOR IN THE FIELD.

13. PERMANENT GROUND ANCHOR LOCK OFF LOAD = 60 PERCENT OF FACTORED DESIGN LOAD.
Appendix 8.1-A3-2 Soldier Pile/Tieback Wall Details 1 of 2

- **W SECTION OR HP SECTION (TYP.)**
  - 8"ø XS PIPE (TYP.)
  - W SECTION OR HP SECTION (TYP.)
  - 2" MIN. BEARING LENGTH
  - SHIM AS NECESSARY FOR FULL BEARING.
  - WHERE NECESSARY CHIP OUT SHAFT BACKFILL TO PLACE LAGGING.

- **2" MIN. BEARING LENGTH**
  - SHIM AS NECESSARY FOR FULL BEARING.
  - 4'-0" WIDE STRIP OF PREFABRICATED DRAINAGE MAT (TYP.) CENTERED BETWEEN SOLDIER PILE FLANGES.

- **FRACTURED FINISH WITH PIGMENTED SEALER**

- **¾"ø x 6" WELDED SHEAR STUDS**
  - AT 1'-0" (TYP.)

- **BACKFILL VOIDS BEHIND LAGGING WITH A FREE DRAINING MATERIAL AS APPROVED BY THE ENGINEER.**

- **CHIP OUT SHAFT BACKFILL TO PLACE LAGGING.**

- **3" MIN. CLR. COVER TO SOLDIER PILE (TYP.) ANGLE OF NO LOAD ZONE**

- **SOLDIER PILE WALL WITH P.G.A.**
  - 1'-2" MIN. FOR WALLS WITH P.G.A.
  - 9" MIN FOR WALLS WITHOUT P.G.A.

- **LIFTING HOLES**
  - TO BE DRILLED IN THE SHOP PRIOR TO PAINTING THE PILE.

- **REMAINING PORTION OF SOLDIER PILE SHAFT**

- **TYPICAL SECTION**
  - SHOWN FOR SOLDIER PILE WITH P.G.A.
  - SIMILAR FOR SOLDIER PILE WITHOUT P.G.A.
  - P.G.A. PERMANENT GROUND ANCHOR

- **LAGGING SYSTEM SHALL BE DESIGNED BY THE CONTRACTOR AND SUBMITTED TO THE ENGINEER FOR APPROVAL IN ACCORDANCE WITH THE STANDARD SPECIFICATION SECTION 6-16.3(6).**

**Note to Designer:**
- For walls with P.G.A., use a section size with a flange width of at least equal to HP12x53 or W12x65.

**LAGGING IN SERVICE LESS THAN 36 MONTHS**

- * USE CONTROLLED DENSITY FILL WHEN PLACED IN THE DRY. USE PUMPABLE LEAN CONCRETE WHEN PLACED IN THE WET.

**PLAN**

- **SOLDIER PILE WALL WITH P.G.A.**
  - 3" MIN. CLR. COVER TO P.G.A. ASSEMBLY (TYP.)
  - BACKFILL VOIDS BEHIND LAGGING WITH A FREE DRAINING MATERIAL AS APPROVED BY THE ENGINEER.

- **SOLDIER PILE WALL WITHOUT P.G.A.**
  - 1½" MIN. CLR. COVER TO P.G.A. ASSEMBLY (TYP.)

- **W.P.**
  - 15'-0" MIN. (TYP.)
  - 15° (TYP.)
  - 3'-0" VARIES

- **P.G.A. = PERMANENT GROUND ANCHOR SHOWN FOR SOLDIER PILE WITH P.G.A. SIMILAR FOR SOLDIER PILE WITHOUT P.G.A.**

- **LAGGING MATERIAL**
  - USE CONTROLLED DENSITY FILL WHEN PLACED IN THE DRY. USE PUMPABLE LEAN CONCRETE WHEN PLACED IN THE WET.

- **ELECTRICAL CONDUIT**
  - 2" ø 4D PIPE (TYP.)

- **CONCRETE FASCIA PANELS**
  - 2'-0" BEHIND FINAL GROUND LINE

- **LIMITS OF PAINT ON SOLDIER PILE**
  - 3:1 SLOPE

- **HP**
  - 2'-0" BELOW FASCIA

- **EMBEDMENT "D" VARIES**

- **CONTROLLED DENSITY FILL**
  - * USE CONTROLLED DENSITY FILL WHEN PLACED IN THE DRY. USE PUMPABLE LEAN CONCRETE WHEN PLACED IN THE WET.

- **PLAN**
  - **SOLDIER PILE WALL WITH P.G.A.**
    - 2'-0" BELOW FINAL GROUND LINE
    - 1'-0" EMBEDMENT "D" VARIES

- **LIMITS OF PAINT ON SOLDIER PILE**
  - 3:1 SLOPE

- **EMBEDMENT "D" VARIES**

- **CONTROLLED DENSITY FILL**
  - * USE CONTROLLED DENSITY FILL WHEN PLACED IN THE DRY. USE PUMPABLE LEAN CONCRETE WHEN PLACED IN THE WET.

- **PLAN**
  - **SOLDIER PILE WALL WITHOUT P.G.A.**
    - 2'-0" BELOW FINAL GROUND LINE
    - 1'-0" EMBEDMENT "D" VARIES

- **LIMITS OF PAINT ON SOLDIER PILE**
  - 3:1 SLOPE

- **EMBEDMENT "D" VARIES**

- **CONTROLLED DENSITY FILL**
  - * USE CONTROLLED DENSITY FILL WHEN PLACED IN THE DRY. USE PUMPABLE LEAN CONCRETE WHEN PLACED IN THE WET.
2" MIN. BEARING LENGTH, SHIM AS NECESSARY FOR FULL BEARING.

CHIP OUT SHAFT BACKFILL TO PLACE LAGGING.

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

FRACTURED FINISH WITH PIGMENTED SEALER

REMAINING PORTION OF SOLDIER PILE SHAFT

W PILE

8" ø X 8" WELDED SHEAR STUDS

1½" MIN. CLR. COVER TO P.G.A. ASSEMBLY (TYP.)

1'-2" MIN. FOR WALLS WITH P.G.A.
9" MIN FOR WALLS WITHOUT P.G.A.

1'-0" (TYP.)
2"ø HOLE TOP OF W SECTION

LIFTING HOLE TO BE DRILLED IN THE SHOP PRIOR TO PAINTING THE PILE.

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 BES 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:
   1. Hem-fir timber lagging shall not be used.
   2. 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.
SIMILAR FOR SOLDIER PILE WITHOUT P.G.A.

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

**LAGGING IN SERVICE**

36 MONTHS OR LONGER
Appendix 8.1-A3-4 Soldier Pile/Tieback Wall Details 2 of 2

**SOLDIER PILE/TIEBACK WALL DETAIL**

**ELEVATION - SOLDIER PILE WITH P.G.A. THRU WEB**

**SECTION B**

**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. Welds must be checked along web to pipe and plate to flange. Welds must be capable of transferring PGA loads and flexural loads.

3. For walls with P.G.A. use a section size with a flange width larger than or equal to HP12x53 or W12x65.

---

**Bearings and Structures Office**

Washington State Department of Transportation

SOLDIER PILE/TIEBACK WALL DETAILS 2 OF 2
Appendix 8.1-A3-5 Soldier Pile/Tieback Wall Fascia Panel Details

SOLDIER PILE/TIEBACK WALL
FASCIA PANEL DETAILS

For information not shown or noted, see Bridge sheets &

2" 2"
2 #4 & 3 #4 SPA. @ 1'-6" max.
with min. splice of 2'-0"

 expansion joint detail (typ.)

PARTIAL WALL ELEVATION

For information not shown or noted, see bridge sheet.

- See typical section on bridge sheet.
- Strongback(s) and ties spaced as required for forming.
- Face of wall 3'-0" min.
- Gravel backfill for drain.
- Construction debris for underground drain.
- Concrete fascia
- Gutter
- 3"ø PVC drain pipe
- 1'-8"
- Gravel backfill for drain
- Concrete fascia

TYPICAL FASCIA PANEL FORMWORK

- See section 6-16.3(2) for fascia panel forming requirements.
- Strongback(s) and ties spaced as required for forming.

WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION

SOLDIER PILE/TIEBACK WALL
FASCIA PANEL DETAILS
Appendix A

1. Anchorage Cover
2. Nut
3. Anti-Corrosion Grease
4. Bearing Plate
5. Trumpet
6. Anti-Corrosion Grease
7. Seal
8. Smooth PVC Bond Breaker
9. Protected Bar Coupler
10. Bar Tensioner
11. Encapsulation Grout
12. Centralizers
13. Corrugated PVC
14. Anchor Grout
15. End Cap
16. Non-structural Filler

NOTE:
The double corrosion protection system at the anchor head shall be detailed to allow a minimum of 0.2% variation in the slope of the soil anchor for placement tolerance.

All anchorage covers shall be bolted to the bearing plates.
Appendix A

Bridge Design Manual

Chapter 8

AUGUST 2000

Soil Nail Layout

[Diagram showing soil nail layout with elevations and dimensions marked.]
Appendix A

Bridge Design Manual

Chapter 8

August 2010

Soil Nail Wall Section

Appendix 8.1-A4-2 Soil Nail Wall Section
Appendix A

Bridge Design Manual

August 2010

Soil Nail Wall

Fascia Panel Details

Typical Section

Fascia Wall Reinforcement

Note:
Expansion joints to be located at a maximum spacing of 24'-0" C. to C., centered between nails except if the joint is within 1'-6" of a step at the top of wall, the joint is to be located at that step.

Typical Between Anchor Joint w/ Roughened Surface (Typ.)

#4 (Typ.) 6" #6 - 4 SPA. @ 1'-0" = 4'-0"

Detail A

Detail B

Detail C

Anchor Plate Details

Note:
Expansion joints to be located at a maximum spacing of 24'-0" C. to C., centered between nails except if the joint is within 1'-6" of a step at the top of wall, the joint is to be located at that step.

Note:
½" premolded joint filler

Vert. Surface

WWW 4 x 4 W 4.0 x W 4.0 1'-3" Min. Splice

Vert. Surface

WWW 4 x 4 W 4.0 x W 4.0 1'-3" Min. Splice

WWW 4 x 4 W 4.0 x W 4.0 1'-3" Min. Splice
Appendix A

Chapter 8

NOISE BARRIER ON BRIDGE

NOISE BARRIER WALL
ON BRIDGE

CURB LINE AT TOP OF ROAD

FRACURED PIN FINISH

FRACURED PIN FINISH

CONSTR. JOINT WITH ROUGHED SURFACE

TOP OF TRAFFIC BARRIER

TOP OF ROADWAY

2"Ø CONDUITS

BARRIER REINFORCEMENT, SEE TRAFFIC BARRIER SHEETS FOR DETAILS.
Appendix A

BRIDGE DESIGN MANUAL

Chapter 8

Cable Fence

AUGUST 2012

NOTES:

1. ALL PIPE SHALL BE STEEL PIPE ASMI ABS GRADE B.
2. ALL STEEL PLATE SHALL BE ASMI A 36.
3. ALL PARTS EXCEPT WIRE ROPE SHALL BE HOT DIP GALVANIZED IN ACCORDANCE WITH AASHTO M111 OR M232 AFTER FABRICATION.
4. SPELTER SOCKETS AND SOCKETING PROCEDURE SHALL BE AS PER ROPE MANUFACTURER.
5. WIRE ROPE SHALL BE INSTALLED TO 0.4 KIP TENSION LEAVING 6" OF TAKE UP AVAILABLE IN THE TURNBUCKLE.
6. EACH CONTINUOUS LENGTH OF CABLE SHALL HAVE A TURNBUCKLE AT ONE END ONLY AND BE ANCHORED TO END POST WITH BRACE AT BOTH ENDS.
7. CENTER SUPPORT NOT TO BE INSTALLED ACROSS EXPANSION JOINT.
8. ALL POSTS TO BE INSTALLED VERTICAL.

WASHINGTON STATE
Department of Transportation

BRIDGE AND STRUCTURES OFFICE
Appendix A

Bridge Design Manual

August 2010

Chapter 8

Cable Fence - Top Mount

**Elevation - Cable Fence**

- Angles vary (45° appear) with the slope of the top of wall.

**Notes:**

1. All pipe shall be steel pipe asew ABS grade B.
2. All steel plate shall be ASTM A 36.
3. Wire rope shall conform to ASTM A 604 with G, a weight zinc-coated wire.
4. All parts except wire rope shall be hot dip galvanized in accordance with added M10 or M12 after fabrication, unless noted otherwise.
5. Splicer sockets and socketing procedure shall be as per rope manufacturer.
6. Wire rope shall be installed to 400 lbs tension leaving a take up of 8' 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 braced shall not to be installed across expansion joint.
9. Cable fence was designed for a 200 lb. load in the top rail applied in any direction as required by Washington Administrative Code 230-15B-803.