Chapter 8 Walls and Buried Structures — Contents

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 the *Geotechnical Design Manual* Chapter 15.

Standard designs for 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 *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 "preapproved" by the Bridge and Structures Office and the Materials Laboratory Geotechnical Branch. The PS&E for "preapproved" 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 *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 *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 reinforced concrete cantilever walls with attached traffic barriers, 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 *Design Manual* M 22-01 and *Geotechnical Design Manual* Chapter 15, 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:

1. Proprietary Structural Earth (SE) Walls – *Standard Specifications* Section 6-13.
2. Geosynthetic Walls (Temporary and Permanent) – *Standard Plan* D-3 and *Standard Specifications* Section 6-14.
3. Standard Reinforced Concrete Cantilever Retaining Walls - *Standard Plans* D-10.10 through D-10.45 and *Standard Specifications* Section 6-11.
4. Soldier Pile Walls and Soldier Pile Tieback Walls – *Standard Specifications* Sections 6-16 and 6-17.
5. Soil Nail Walls–*Standard Specifications* Section 6-15.

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. Preapproved Proprietary Walls

A wall specified to be supplied from a single source (patented, trademark, or copyright) is a proprietary wall. Walls are generally preapproved for heights up to 33 ft. The Materials Laboratory Geotechnical Division will make the determination as to which preapproved 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 principle 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 Bridge Standard Drawing 8.1-A2 for details that need to be provided in the Plans for manufacturer designed walls.

A list of current preapproved proprietary wall systems is provided in the *Geotechnical Design Manua* Appendix 15-D. For additional information see the retaining walls chapter in the *Design Manual* M 22-01 and *Geotechnical Design Manual* Chapter 15. For the SEW shop drawing review procedure see *Geotechnical Design Manual* Chapter 15.

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 preapproved proprietary wall systems and their height limitations is provided in the *Geotechnical Design Manual* Appendix 15-D. The Region shall refer to the retaining walls chapter in the *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 Plans D-3.09, D-3.10 and D-3.11.

C. 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 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* Chapter 15. See Bridge Standard Drawing 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 Bridge Standard Drawing 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 *Geotechnical Design Manual* M 46-03. 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 preapproved 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 percent of the wall inertia force combined with 50 percent 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 in accordance with *Geotechnical Design Manual* M 46-03, Nov. 2008 Section 15.4.2.9, and was supplemented by AASHTO *LRFD Bridge Design Specifications* (Figure 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 in accordance with AASHTO *LRFD Bridge Design Specifications* 3.4.1-1 and 2. Stability analysis performed in accordance with 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 percent of the wall inertia force combined with 50 percent of the seismic earth pressure.

b. Active Earth pressure distribution was linearly distributed in accordance with Section 7.7.4. 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 in accordance with *Geotechnical Design Manual* Section 15.4.2.9, and was supplemented by AASHTO *LRFD Bridge Design Specifications* (Figure 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 in accordance with AASHTO *LRFD Bridge Design Specifications* 3.4.1-1& 2. Stability analysis performed in accordance with 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

1. Resistance, Eccentricity, and Sliding Stability

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. Otherwise, the passive resistance shall be ignored above the bottom of the footing for the Strength Limit States and ignored above the top of the footing for the Extreme Event Limit States

The design soil bearing pressure at the toe of the footing shall not exceed the factored soil bearing capacity supplied by the Geotechnical Engineer.

2. Application of Lateral Loads

The lateral loads for reinforced concrete retaining walls with a horizontal backfill shall be applied as shown in Figure 8.1.4-1.

The lateral loads for reinforced concrete retaining walls with a sloping backfill shall be applied as shown in Figure 8.1.4-2.

a. The sloped backfill can be a 2H:1V maximum slope with a limited surcharge height (broken back backfill) or a 3H:1V maximum slope with no surcharge height (infinite backfill).

b. For the broken back backfill condition, the slope angle β^* is based on the AASHTO LRFD Figure 3.11.5.8.1-3.

c. The wall backfill interface friction angle is $\delta = 2/3\ \phi_f$ but not greater than β or β^* which is consistent with the Coulomb wedge theory.

3. Application of Collision Loads

For walls with traffic barriers constructed integral with the wall stem, the vehicular collision load shall be included in the design. To ensure that any failure due to the collision remains in the barrier section, the top of the wall stem shall have sufficient resistance to force the yield line failure pattern to remain within the barrier. The top of the wall stem shall be designed in accordance with the requirement of the AASHTO LRFD article A13.4.

As shown in Figures 8.1.4-3 and 8.1.4-4, the collision force (CT, F_t) is assumed to be distributed over the longitudinal length (L_t) at the top of the traffic barrier and is assumed to distribute downward to the top of the footing at a 45 degree angle. See AASHTO LRFD Table A13.2-1 for L_t and F_t values. The distribution of the collision force in the footing shall be the distance between expansion joints.

For the Extreme Event II Limit State, the load factor, γ_p, for EH is 1.0 to account for the dynamic nature of the collision load.

4. Wall Footing Structural Design

Refer to Section 7.7 for additional footing structural design criteria. The plan detailing criteria specified in Section 7.7.3.A are not applicable to retaining wall plans.

For retaining walls supported by deep foundations (shafts or piles), refer to Sections 7.7.5, 7.8 and 7.9.

The structural design of the footing shall assume a triangular or trapezoidal bearing pressure distribution in accordance with the AASHTO LRFD Article 10.6.5.

When designing the transverse reinforcement located in the bottom of the footing, the contribution of the soil located over the toe of the footing shall be ignored.

When designing the transverse reinforcement located in the top of the footing, the contribution of the bearing pressure under the footing shall be ignored.

Control of cracking by distribution of reinforcement as specified in AASHTO LRFD article 5.7.3.4 shall be checked for the top and bottom face of the footing.

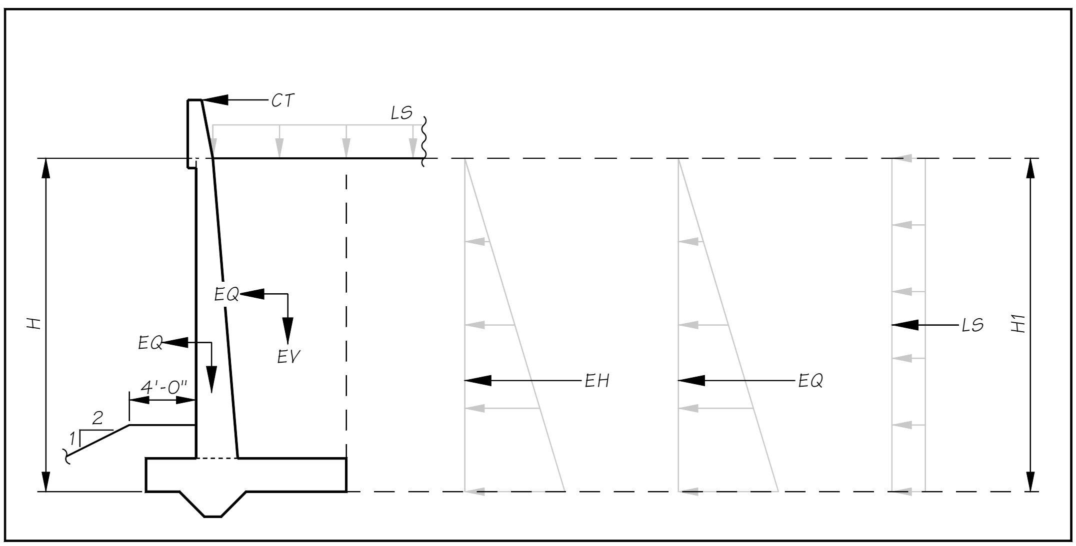
When designing the transverse reinforcement located in the top of the footing, the contribution of the bearing pressure under the footing shall be ignored.

Control of cracking by distribution of reinforcement as specified in AASHTO LRFD Article 5.7.3.4 shall be checked for the top and bottom face of the footing.

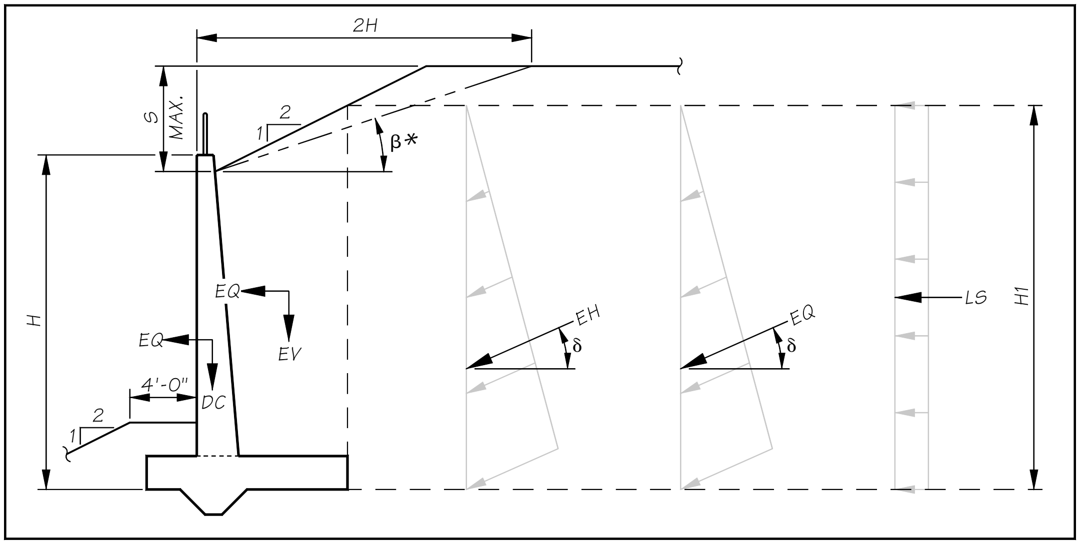
5. Wall Stem Structural Design

Refer to Sections 7.5.3 and 7.5.9 for additional wall stem structural design criteria.

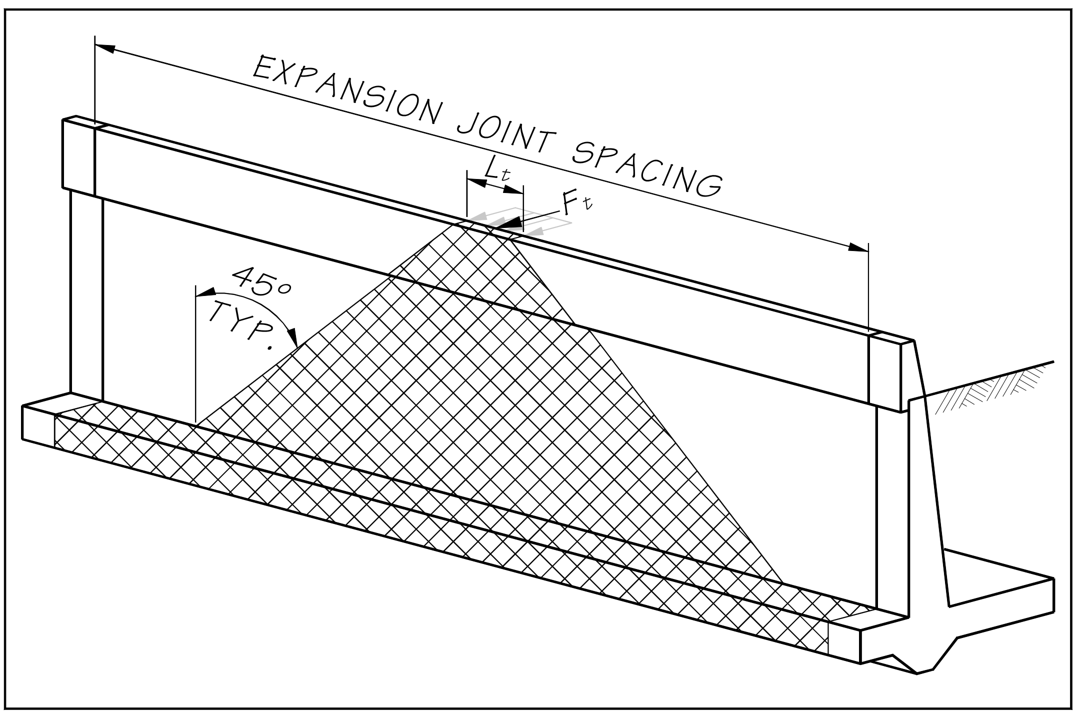
In accordance with *Standard Specifications* Section 6-11.3(3), the Contract Plans or Special Provisions are to state whether the cast-in-place semi-gravity concrete cantilever wall may be constructed with precast concrete wall stem panels. For cast-in-place semi-gravity concrete cantilever walls with traffic barriers cast integral with the wall stem, the Contract Plans or Special Provisions are to provide explicit direction regarding whether the traffic barrier is permitted to be precast with the precast wall stem or cast-in-place after the precast wall stems are installed. When permitting the traffic barrier to be precast integral with the wall stem, the wall stem design and detailing shall account for the collision load transfer path into the wall stem.



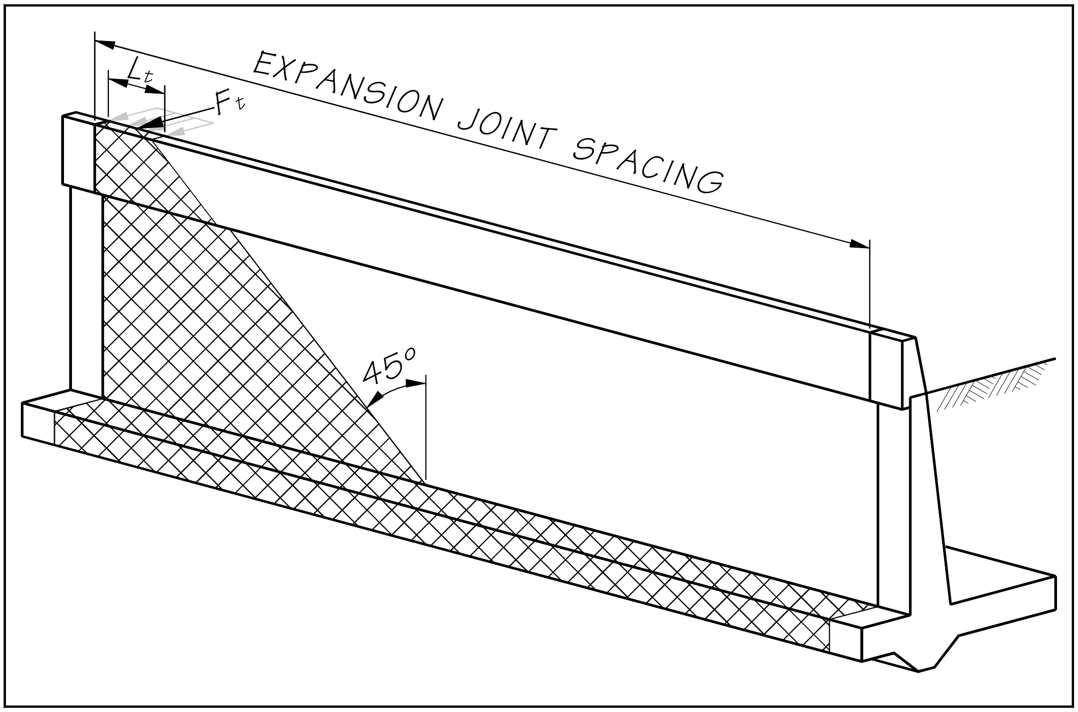
Application of Lateral Loads for walls with a horizontal backfill
Figure 8.1.4-1



Application of Lateral Loads for walls with a sloping backfill
Figure 8.1.4-2



Application and Distribution of Vehicular Collision Load occurring near the midsection.
Figure 8.1.4-3



Application and Distribution of Vehicular Collision Load occurring near the end.
Figure 8.1.4-4

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 degrees to 45 degrees), 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 *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 *Geotechnical Design Manual* 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-A1.

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 *Geotechnical Design Manual* M 46-03 design criteria. The flexural design shall be based on the elastic section modulus "S" for the entire length of the pile for all Load combinations. The flexural design of soldier piles with tiebacks shall consider the requirements of AASHTO *LRFD Bridge Design Specifications* Article 6.10.8.2 and 6.10.3.2.

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 *Standard Specifications* Section 6-16.3(6)B.

2. Permanent Lagging

Permanent lagging shall be designed for 100 percent 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 fascia is to extend 2 feet minimum below the finish ground line adjacent to the face of the wall.

When concrete fascia panels are placed on soldier piles, a generalized detail of lagging with strongback (see Bridge Standard Drawing 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 percent 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 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. Preapproved 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 the *Geotechnical Design Manual* Appendix 15-D. For the SE wall shop drawing review procedure, see the *Geotechnical Design Manual* Chapter 15.

B. Non-Preapproved Proprietary Structural Earth Walls

Structural earth walls that exceed the limitations as provided in the *Geotechnical Design Manual* Appendix 15-D are considered to be non-preapproved. Use of non-preapproved 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* 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* Section 15.4.5.

C. Fall Protection

For retaining walls with exposed wall heights of 4 feet or more, fall protection shall be provided in accordance with WAC 296-155-24615(2) and WAC 296-155-24609 and as described in the *Design Manual* 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 *Geotechnical Design Manual* 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. See Figure 7.5.10-1.

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 for wall, underdrain pipe and gravel backfill for drain are not included in the bridge quantities. The size of the underdrain pipe should not be shown on the bridge plans as this is a Design PE Office item and is subject to change during the design phase. 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. Expansion, Contraction and Construction Joints

Odd panels for all types of walls shall normally be made up at the ends of the walls. All expansion, contraction and construction joints shall be shown in the plan sheets and are typically shown on the elevation.

1. Expansion Joints

For cast-in-place construction, a minimum of ½ inch premolded filler should be specified in the expansion joints.

Precast concrete cantilever wall expansion joints shall be in accordance with the *Standard Specifications* Section 6-11.3(3).

For cantilevered and gravity walls, expansion joint spacing should be a maximum of 60 feet on centers. For cantilevered and gravity walls constructed with a traffic barrier attached to the top, expansion joint spacing should be consistent with the length determined to be 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.

Expansion joints are not permitted in footings except at bridge abutments and where the substructure type changes such as locations where spread footing to pile footing occurs. In these cases, the footing shall be interrupted by a ½ inch premolded expansion joint through both the footing and the wall.

2. Contraction Joints

Contraction joints shall be spaced at a maximum of 30 feet for walls with expansion joints spaced at intervals exceeding 32 feet.

3. Construction Joints

Construction joints are only permitted in the footing. 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 or contraction joints in the wall stem.

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 8.1.9-1.

2. Follow the example format shown in Figure 8.1.9-1.
3. Calculate approximate quantities using the Standard Plans.
4. Wall dimensions shall be determined by the designer using the Standard Plans.
5. Do not show any details given in the Standard Plans.
6. Specify in the Plans all deviations from 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.

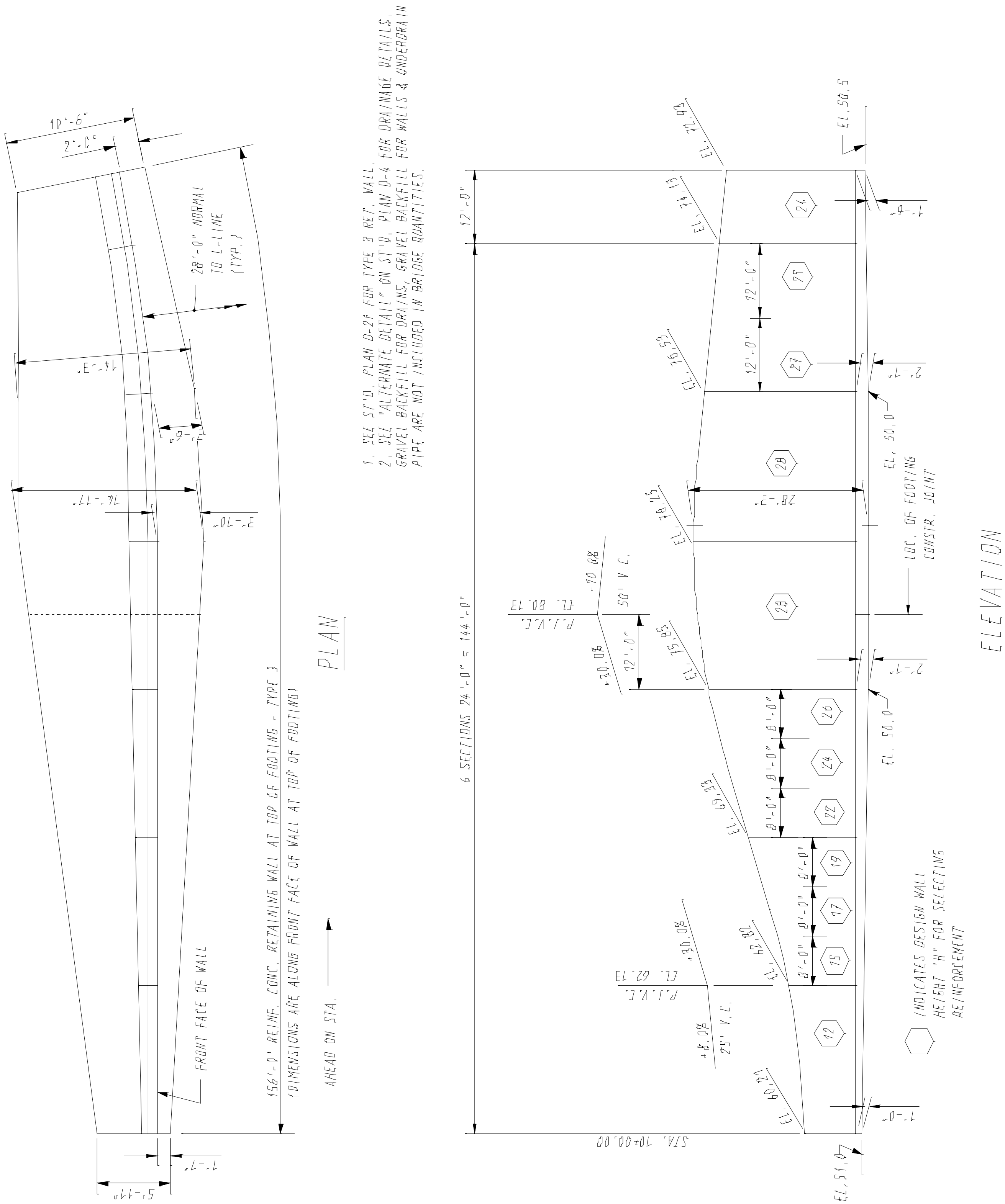

PLAN
ELEVATION
156'-0" REINF. CONC. RETAINING WALL AT TOP OF FOOTING - TYPE 3
(DIMENSIONS ARE ALONG FRONT FACE OF WALL AT TOP OF FOOTING)
AHEAD ON STA.
FRONT FACE OF WALL
6 SECTIONS 24'-0" = 144'-0"
LOC. OF FOOTING CONSTR. JOINT
INDICATES DESIGN WALL HEIGHT "H" FOR SELECTING REINFORCEMENT
STA. 10+00.00

Figure 8.1.9-1

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:

$$\text{Seismic Dead Load} = A_s \times f \times D \tag{8.2-1}$$

In which:

$$A \times f \geq 0.10 \tag{8.2-2}$$

Where:

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

Location	f
Monolithic connection	1.0
Monolithic connection on bridges	2.5
Connection of precast wall to bridge barrier	8.0
Connection of precast wall to retaining wall or moment slab barrier	5.0

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.
   a. AASHTO *Guide Specifications for Structural Design of Sound Barriers,* 1989 and interims through 2002.
   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 *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 *Geotechnical Design Manual* 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, 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 Buried Structures

8.3.1 General

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 structure types listed above are for hydraulic uses. In accordance with current WSDOT policy, only the cast-in-place reinforced concrete and precast concrete arch, box, and elliptical structures shall be used for buried highway and hydraulic structures.

The term culvert used in this chapter and in the *Standard Specifications* applies to all buried hydraulic structures only. The term tunnel applies to all buried highway structures conveying vehicles or pedestrians.

8.3.2 WSDOT Designed Standard Culverts

WSDOT Bridge and Structures Office has developed culvert standards for the Precast Reinforced Concrete Split Box Culvert (PRCSBC) and Precast Reinforced Concrete Three-Sided Structures (PRCTSS) with span lengths from 20' to 60'. See Section 8.4 for the list of Bridge Standard Drawings for Buried Structures containing the geometry table, typical sections and general details. See Appendix 8.3-A1 to 8.3-A3 for the Design Criteria used.

8.3.3 General Design Requirements

Design of buried structures shall be in accordance with the requirements and guidance cited herein and in the current AASHTO *LRFD Bridge Design Specifications*, AASHTO Guide Specifications for LRFD Seismic Bridge Design, Special Provisions and the *Standard Specifications for Road, Bridge, and Municipal Construction* M 41-10.

All buried structures shall be designed for a minimum service life of 75 years.

The span length shall be the widest opening from interior face to interior face as measured along the centerline of the roadway.

A. Span Length Limitations

1. **Span lengths less than 20 feet**

   Region Project Engineer Office may allow Contractor supplied designs of the buried hydraulic structure while under contract.

2. **Span lengths equal to or greater than 20 feet and less than 26 feet.**

   Region Project Engineer Office may utilize Contractor supplied designs of the buried hydraulic structure while under contract if the structure meets all of the following criteria:

   a. Geotechnical Report foundation recommendation of spread footing support based on confirmed presence of competent soils at the site. No soft soil support embankment requiring lightweight fills or ground improvement, as confirmed by the Geotechnical Report.

b. Peak Seismic Ground Accelerations at the project site of 0.3g or less, as shown in the *Geotechnical Design Manual* Figure 6-8 "Determination of Seismic Hazard Level, Peak Horizontal Acceleration (%G) for 7% Probability of Exceedance in 75 Years for Site Class B (Adapted From AASHTO 2012).

c. No liquefaction, lateral spread risks, or within the earthquake fault line as confirmed by the Geotechnical Report.

d. Skew angle of waterway alignment limited to within 25 degrees of a normal 90-degree crossing of the roadway alignment if the soil fill is retained by headwalls.

e. Not scour critical, as confirmed by the HQ Hydraulics Office.

3. Span lengths equal to or greater than 20 feet and less than 26 feet and with geometric and site restrictions.

Buried hydraulic structures that do not meet the criteria listed in Section 8.3.3.A.2 above shall utilize the following procedure.

a. A preliminary plan shall be completed in accordance with the criteria listed in Chapter 2.

b. The design of the structure shall be completed prior to contract and the plans shall be included as a part of the Ad copy PS&E.

c. The design may be completed by one of the following;
- WSDOT engineering staff,
- Proprietary supplier identified as a sole source by WSDOT,
- Three proprietary suppliers with all three plan sets included as options in the Ad copy PS&E.

4. Span lengths greater than 26 feet.

a. A preliminary plan shall be completed in accordance with the criteria listed in Chapter 2.

b. The design of the structure shall be completed prior to contract and the plans shall be included as a part of the Ad copy PS&E.

c. The design may be completed by one of the following;
- WSDOT engineering staff,
- Proprietary supplier identified as a sole source by WSDOT,
- Three proprietary suppliers with all three plan sets included as options in the Ad copy PS&E.

B. Application of Loads

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 Section 3.5 for inclusion of live load in the Extreme Event-I load combination is applicable.

C. Buried Structure Foundation Design

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.

D. Buried Structure Wingwall and Headwall Design

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

E. Buried Structure Seismic Design

The provisions below 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 requirements are specified, they shall be site or project specific and are tailored to a particular structure type.

The seismic design need not be considered for buried structures with span lengths of less than 20 feet.

Buried structures greater than or equal to 20 feet shall be designed for seismic effects. Seismic design of buried structures shall be in accordance with 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 shall 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.

Depth to Top of Buried Structure, feet	Ratio of Ground Motion at Buried Structure Depth to Motion at Ground Surface
< 20	1.0
20 to 50	0.9
50 to 100	0.8
>100	0.7

Ground Motion Attenuation with Depth
Table 1

For buried structures, with span lengths equal to or greater 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.

F. **Buried Structure 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 the following;

1. Load rating for all buried structures with span lengths beyond 20 feet. The load rating shall be in accordance with Chapter 13.
2. Geotechnical design parameters, hydraulic analysis, including scour depth, installation procedures, backfill materials, and compacting sequences.
3. The structural adequacy of the buried structure for the required depth of fill shall be provided in the submittal.
4. Final as-built plans shall be submitted to the Bridge and Structures Office for records.

8.3.4 Design of Box Culverts

Box culverts are four-sided rigid frame structures and are either made from cast-in-place (CIP) reinforced concrete or precast concrete. See Appendix 8.3-A1 for design criteria specific to concrete four sided split box culverts.

Under the current *Standard Specifications*, 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.

Precast concrete fabricators are responsible for the structural design and the preparation of shop plans for the precast reinforced concrete box and split box culverts designed by the prefabricators.

A. **Materials**

1. **Concrete**

   Precast concrete shall be class 5000, 6000, or 7000 SCC. All cast-in-place concrete shall be class 4000.

2. **Steel**

   Nominal yield strength for reinforcement bar shall be 60 ksi. Wire fabric of yield strength of 65 ksi may be used.

3. **Cover**

   2” minimum cover for reinforcement at all faces.

B. **Joint Design and Details**

1. The joints shall be fabricated in accordance to ASTM C 1786 with tongue and groove connection. See Section 8.4 Bridge Standard Drawings for details.
2. The top slab joint shall designed as an edge beam in accordance with AASHTO Section 4.6.3.10.4, or capable of transferring a minimum of 3000 lbs per linear foot of top slab joint.
3. The grouted joint can be used for the cast-in-place concrete box culvert.

C. Connections

1. The joints between the upper and lower sections shall be designed for the lateral forces due to the seismic and soil pressures per requirements above. See *Standard Specifications* Section 7.02.3(6)C.
2. The segments at portals shall be designed for any lateral load due to the overburden.

D. Joint Filler and Cover

All joints between segments shall be sealed by joint sealant in accordance with ASTM C 990. All joints shall be wrapped with external sealing band in accordance with ASTM C 877, except the bottom slab. See Section 8.4 Bridge Standard Drawings for details.

8.3.5 Design of Precast Reinforced Concrete Three-Sided Structures

Precast reinforced concrete three-sided frame structures are chorded arch, arch, or elliptical structures. These systems require a CIP concrete or precast footing and walls. See Appendix 8.3-A4 for design criteria specific to three-sided precast concrete culverts.

A. Materials

1. Concrete

Precast concrete shall be class 5000, 6000, or 7000 SCC. All cast in place concrete shall be class 4000.

2. Steel

Nominal yield strength for reinforcement bar shall be 60 ksi. Wire fabric of yield strength of 65 ksi may be used.

3. Cover

2” minimum cover for reinforcement at all faces.

B. Joint Design and Details

1. Tongue and groove, shear key, and other types of connection can be used to control the differential settlements between segments or live load deflection. Tongue and groove connection shall be fabricated per ASTM C 1786.
2. For structures with 2’ or less fill cover on top, the top slab joint of the precast box shall be designed as an edge beam in accordance with AASHTO Section 4.6.3.10.4.
3. Cast-in-place joints can be used for culverts with highway inside the structure.

C. Connections

1. For the precast three-sided culvert, the joints between the precast and wall section shall be designed for the lateral forces due to the seismic and soil pressures per requirements above with shear key, block restrainer, or dowel bars. See Section 8.4 Bridge Standard Drawings for details.
2. The segments at portals shall be designed for any lateral load due to the overburden.

D. Joint Filler and Cover

All joints between segments shall be sealed by joint sealant in accordance with ASTM C 990. All joints shall be wrapped with external sealing band in accordance with ASTM C 877. See Section 8.4 Bridge Standard Drawings for details.

8.3.6 Design of 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 Section 8.16 of the *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 (WA) load factor shall be taken as γ_{WA} = 1.25 in AASHTO LRFD Table 3.4.1-1. In the Strength Limit State, the load factors that resist buoyancy (γ_{DC}, γ_{DW}, γ_{ES}, Etc.) shall be their minimum values, in accordance with 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, in accordance with 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 (ø) for tie-downs shall be provided by the Geotechnical Engineers. The buoyancy check shall be as follows:

For Buoyancy without tie-downs:

$$(R_{RES} / R_{UPLIFT}) \geq 1.0$$

For Buoyancy with tie-downs:

$$(R_{RES} / [R_{UPLIFT} + øR_n]) \geq 1.0$$

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 in accordance with AASHTO LRFD Section 5.7.3.4 with $\gamma_e = 0.5$ (in order to maintain a crack width of 0.0085 inches, in accordance with 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 Asset Management Unit (see Section 12.4.10.B). 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.

8.3.7 Design of 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.

8.3.8 Design of 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*, and FHWA-NHI-10-034 *Technical Manual for Design and Construction of Roadway Tunnels - Civil Elements*.

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.

8.4 Bridge Standard Drawings

Cable Fence

8.1-A6-1	Cable Fencing for Wall
8.1-A6-2	Cable Fencing for Wall w/Top Mounted Base

Soil Nail Wall

8.1-A4-1	Soil Nail Wall, 1 of 3
8.1-A4-2	Soil Nail Wall, 2 of 3
8.1-A4-3	Soil Nail Wall, 3 of 3

TieBack Walls

8.1-A2-1	SEW Wall Elevation
8.1-A2-2	SEW Wall Section
8.1-A3-1	Soldier Pile/Tieback Wall Elevation
8.1-A3-2	Soldier Pile/Tieback Walls Details A
8.1-A3-3	Soldier Pile/Tieback Walls Details B
8.1-A3-4	Soldier Pile/Tieback Walls Details
8.1-A3-5	Soldier Pile/Tieback Walls Fascia Panel Details
8.1-A3-6	Soldier Pile/Tieback Wall Perm Ground Anchor Details

Noise Barrier

8.1-A5-1	Noise Barrier on Bridge

Buried Structures

8.3.2-A1	Precast Split Box Typical Section
8.3.2-A2	Typical 3-Sided Precast Culvert Section and Table
8.3.2-A3	3-Sided Precast Culvert Series FC20 to FC40, and SB20 and SB25
8.3.2-A4	3-Sided Precast Culvert Series VC45 to VC50
8.3.2-A5	3-Sided Precast Culvert Series VC55 to VC60
8.3.2-A6	3-Sided Precast Culvert Footing Joint Connection Details
8.3.2-A7	3-Sided Precast Culvert Panel Joint Connection Details
8.3.2-A8	Precast Split Box Culvert Joint Seal Details
8.3.2-A9	Example of Precast Split Box Culvert Layout
8.3.2-A10	Example of Precast Split Box Culvert Typical Section
8.3.2-A11	Example of Precast Split Box Culvert Reinforcement Details
8.3.2-A12	Example of Precast Split Box Culvert Connection Details

8.5 Appendices

Appendix 8.1-A1	Summary of Design Specification Requirements for Walls
Appendix 8.3-A1	Precast Split Box Culvert Design Criteria
Appendix 8.3-A2	3-Sided Precast Culvert Design Criteria
Appendix 8.3-A3	Design Example of Racking Analysis of Precast Split Box Culvert

Appendix 8.1-A1 Summary of Design Specification Requirements for Walls

Wall Types		Design Specifications
Preapproved Proprietary Structural Earth Walls	General	Design shall be based on current editions, including current interims, of the following documents; AASHTO *Standard Specifications for Highway Bridges*–17th Edition for projects initiated prior to October 1, 2010. AASHTO *LRFD Bridge Design Specifications* for projects initiated after October 1, 2010, *Geotechnical Design Manual* (GDM) and *Bridge Design Manual* (BDM).
	Seismic	AASHTO *LRFD Bridge Design Specifications* 1000 year map design acceleration.
	Traffic Barrier	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.
Non-Preapproved Proprietary Structural Earth Walls	General	Design shall be based on current editions, including current interims, of the following documents; AASHTO *LRFD Bridge Design Specifications*, WSDOT GDM and WSDOT BDM.
	Seismic	AASHTO *LRFD Bridge Design Specifications* 1000 year map design acceleration.
	Traffic Barrier	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.
Standard Plan and Non-Standard Geosynthetic Walls	General	Design shall be based on current editions, including current interims, of the following documents; AASHTO *LRFD Bridge Design Specifications*, *Geotechnical Design Manual* M 46-03 and *Bridge Design Manual* M 23-50.
	Seismic	AASHTO *LRFD Bridge Design Specifications* 1000 year Seismic Acceleration map.
	Traffic Barrier	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.
Standard Plan and Non-Standard Reinforced Concrete Cantilever Walls	General	Current Standard Plan walls are designed in accordance with AASHTO *LRFD Bridge Design Specifications* 4th Edition 2007 and interims through 2008 and the *Geotechnical Design Manual* M 46-03 Nov. 2008. Non-standard reinforced concrete cantilever walls shall be designed in accordance with the current editions, including current interims, of the following documents; AASHTO *LRFD Bridge Design Specifications*, *Geotechnical Design Manual* M 46-03 and *Bridge Design Manual* M 23-50.
	Seismic	AASHTO *LRFD Bridge Design Specifications* 1000 year map design acceleration.
	Traffic Barrier	WSDOT *Bridge Design Manual* and the AASHTO *LRFD Bridge Design Specifications* Section A13.3 for Concrete Railings considering a minimum TL-4 impact load. Ft is distributed over Lt at the top of barrier. Load from top of barrier is distributed at a 45 degree angle into the wall. Current Standard Plan walls are designed for TL-4 impact loading distributed over 48 feet at the base of wall
Soldier Pile Walls With & Without Tie-Backs	General	Design shall be based on current editions, including current interims, of the following documents; AASHTO *LRFD Bridge Design Specifications*, *Geotechnical Design Manual* M 46-03 and *Bridge Design Manual* M 23-50.
	Seismic	AASHTO *LRFD Bridge Design Specifications* 1000 year map design acceleration.
	Traffic Barrier	AASHTO *LRFD Bridge Design Specifications* Section A13.3 for Concrete Railings considering a minimum TL-4 impact load. Ft is distributed over Lt at the top of barrier Load from top of barrier is distributed downward into the wall spreading at a 45 degree angle.

Wall Types		Design Specifications
Standard Plan Noise Barrier Walls	**General**	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 & Interims. Standard Plans D-2.36 and D-2.46 are designed in accordance with AASHTO *LRFD Bridge Design Specifications* 6th Edition 2012 and interims through 2013.
	Seismic	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 & 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.
	Traffic Barrier	AASHTO *Guide Specifications for Structural Design of Sound Barriers* – 1989 & Interims. Standard Plan D-2.46 traffic barrier is designed as stated in Chapter 8.
Non-Standard Noise Barrier Walls	**General**	Design shall be based on current editions, including current interims, of the following documents; AASHTO *LRFD Bridge Design Specifications*, WSDOT GDM and WSDOT BDM.
	Seismic	AASHTO *LRFD Bridge Design Specifications* 1000 year map design acceleration.
	Traffic Barrier	WSDOT *Bridge Design Manual* and the AASHTO *LRFD Bridge Design Specifications* Section A13.3 for Concrete Railings considering a minimum TL-4 impact load.
Soil Nail Walls	**General**	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.
	Seismic	AASHTO *LRFD Bridge Design Specifications* 1000 year map design acceleration.
	Traffic Barrier	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
Non Standard Non Proprietary Walls Gravity Blocks, Gabion Walls	**General**	Design shall be based on current editions, including current interims, of the following documents; AASHTO *LRFD Bridge Design Specifications*, WSDOT GDM and WSDOT BDM.
	Seismic	AASHTO *LRFD Bridge Design Specifications* 1000 year map design acceleration.
	Traffic Barrier	WSDOT *Bridge Design Manual* and the AASHTO *LRFD Bridge Design Specifications* Section A13.3 for Concrete Railings considering a minimum TL-4 impact load.

Appendix 8.3-A1 Precast Split Box Culvert Design Criteria

Appendix 8.3-A2 3-Sided Precast Culvert Design Criteria

Appendix 8.3-A3 Design Example of Racking Analysis of Precast Split Box Culvert

8.99 References

1. AASHTO *LRFD Bridge Design Specifications*, 5th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.
2. AASHTO *Standard Specifications for Highway Bridges*, 17th Ed., 2002
3. WSDOT *Standard Specifications for Highway Bridges and Municipal Construction*, Olympia, Washington 98501.
4. ACI 350/350R-06 Code Requirements for Environmental Engineering Concrete Structures, ACI, 2006.
5. Munshi, Javeed A. *Rectangular Concrete Tanks*, Rev. 5th Ed., PCA, 1998.
6. Miller, C. A. and Constantino, C. J. "Seismic Induced Earth Pressure in Buried Vaults", PVP-Vol.271, *Natural Hazard Phenomena and Mitigation*, ASME, 1994, pp. 3-11.
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8. NFPA 502, *Standard for Road Tunnels, Bridges, and Other Limited Access Highways*.