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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 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 State Geotechnical Engineer.

Temporary retaining walls are defined as walls that are in service or have a design life of three years or less. Any retaining wall that is expected to be in service for more than three years shall be designed for seismic loading. Temporary retaining walls shall be designed in accordance with the requirements of the current editions of the LRFD-BDS and interims, WSDOT Bridge Design Manual including all design memorandums, and the WSDOT Geotechnical Design Manual Chapter 15.

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 State 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:

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

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

#### 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 feet. The Materials Laboratory Geotechnical Branch 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 Manual 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 State Geotechnical Engineer designs the soil nail system whereas the Bridge and Structures Office designs the wall fascia. Presently, the FHWA Publication FHWA-NHI-14-007 "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 LRFD-BDS 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, latest edition.

The State 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 LRFD-BDS 4th Edition 2007 and interims through 2008.

1. **Western Washington Walls (Types 1 through 4)**
   
a. The seismic design of Standard Plan D-10.10 and D-10.15 was completed using an effective Peak Ground Acceleration of 0.51g. The seismic design of Standard Plan D-10.20 and D-10.25 was completed using an effective Peak Ground Acceleration of 0.32g. 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 LRFD-BDS (Figure 11.10.7.1-1). The corresponding $K_{ae}$ 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 LRFD-BDS Sections 3.4.1-1 and 2. Stability analysis performed in accordance with LRFD-BDS Section 11.6.3 and C11.5.5-1 & 2.

g. Wall Types 1 and 2 have not been designed for 42 inch traffic barrier height collision forces. The Standard Plans D-15.10, D-15.20 and D-15.30 are no longer consistent with WSDOT Bridge and Structures Office traffic barrier height policy and shall not be used on any Standard Plan retaining wall.

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 LRFD-BDS Figure 11.10.7.1-1. The corresponding K_{ae} 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 LRFD-BDS 3.4.1-1& 2. Stability analysis performed in accordance with LRFD-BDS Section 11.6.3 and C11.5.5-1 & 2.

g. Wall Types 7 and 8 have not been designed for 42 inch traffic barrier height collision forces. The Standard Plans D-15.10, D-15.20 and D-15.30 are no longer consistent with WSDOT Bridge and Structures Office traffic barrier height policy and shall not be used on any Standard Plan retaining wall.
B. Non-Standard Reinforced Concrete Retaining Walls

1. Bearing 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 $\beta^*$ is based on the LRFD-BDS Figure C3.11.5.8.1-1.

c. The wall backfill interface friction angle is $\delta = \frac{2}{3} \phi_f$ but not greater than $\beta$ or $\beta^*$ 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 LRFD-BDS 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 LRFD-BDS 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, $\gamma_p$, 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 General Footing Criteria provided in [Section 7.7.1](#) shall be applicable to both retaining wall footings and leveling pads. For footings with steps, the bottom of the footing step is to be sloped no steeper than 1H:2V (See Std. Plan D-2.04 for an example). Footings with 90 degree steps at the bottom of the footing shall not be permitted.

The minimum reinforcement criteria for bottom reinforcement of #6 bars at 12” centers and top reinforcement of #5 bars at 12” centers required in [Section 7.7.4F](#) is not applicable to retaining wall footings.

The structural design of the footing shall assume a triangular or trapezoidal bearing pressure distribution in accordance with the [LRFD-BDS 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 [LRFD-BDS Article 5.6.7](#) shall be checked for the top and bottom face of the footing.

For retaining walls supported by deep foundations (shafts or piles), refer to Sections 7.7.5, 7.8, 7.9 and 7.10.

5. **Wall Stem Structural Design**

Refer to [Sections 7.5.4](#) and 7.5.10 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.
Figure 8.1.4-1  Application of Lateral Loads for walls with a horizontal backfill

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

Figure 8.1.4-3  Application and Distribution of Vehicular Collision Load occurring near the midsection
8.1.5 Design of Cantilever Soldier Pile and Soldier Pile Tieback Walls

A. Ground Anchors (Tiebacks)

See LRFD-BDS Section 11.9 “Anchored Walls”. The State 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 LRFD-BDS 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 LRFD-BDS. 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 LRFD-BDS 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 LRFD-BDS 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 factored horizontal force equilibrium and factored moment equilibrium about the bottom of the soldier pile for cantilever soldier piles without permanent ground anchors.

e. As required to satisfy factored moment equilibrium of factored 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 LRFD-BDS.

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 State 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(6B).

2. Permanent Lagging

Permanent lagging shall be designed for all lateral loads that could occur during the life of the wall in accordance with LRFD-BDS 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 State Geotechnical Engineer.

Timber lagging shall be designed in accordance with LRFD-BDS Section 8.6. The size effect factor (CFb) should be considered 1.0, unless a specific size is shown in the wall plans. The wet service factor (CMb) 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 LRFD-BDS Section 11.8.5.2. Lateral earth pressure loads shall not be reduced for soil arching. 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 shall extend below ground the maximum of the following:

a. 2 feet minimum below the finish ground line adjacent to the face of the wall.

b. 3 feet minimum below the lowermost PGA.

c. 2 feet minimum below the scour elevation, unless a greater depth is specified.

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 all loads that could occur during the life of the wall. Lateral earth pressure loads shall not be reduced for soil arching. 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.

Use of structural earth walls in marine environments or areas having a soil chemistry or water chemistry that is considered to be aggressive or corrosive shall not be permitted without the approval of the State Geotechnical Engineer and the State Bridge and Structures Engineer.

Use of structural earth walls in areas of full or periodic freshwater inundation shall meet the following criteria

1. Modular block wall facings shall not be used.
2. Welded wire faced structural earth walls shall not be located below the 100 year mean recurrence interval water surface.
3. The soil and water chemistry shall meet the nonaggressive criteria as described in LRFD-BDS Section 11.10.6.4.2.
4. Free draining backfill material shall be used below the expected high water elevation.
5. The number of weep holes shall be increased beyond that required for a standard design.
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-NHI-14-007 “Geotechnical Engineering Circular No. 7 Soil Nail Walls” February 2015.

The seismic design parameters shall be determined in accordance with the most current edition of the AASHTO Guide Specifications for LRFD Seismic Bridge Design (LRFD-SGS).

Typical soil nail wall details are provided in Appendix 8.1.

8.1.9 **Scour of Retaining Walls**

The foundation for all walls constructed along rivers and streams shall be evaluated during design by the State Hydraulics Engineer for scour in accordance with LRFD-BDS.

The bottom of the wall foundation and bottom of wall elements such as, the fascia panel, lagging, leveling pad, footing, pile cap or shaft cap shall be located a minimum of 2 feet below the scour elevation in accordance with the *Geotechnical Design Manual* Section 15.4.5 unless a greater depth is otherwise specified.

In situations where scour (e.g., due to wave or stream erosion) can occur in front of the wall, the bottom of the wall foundation (e.g., structural earth or Geosynthetic wall leveling pad, concrete wall spread footing, the cap for pile or shaft supported walls), and the bottom of fascia panel or lagging, shall meet the minimum embedment requirements relative to the scour elevation in front of the wall.

At any location where a retaining wall or reinforced slope can be in contact with water (such as a culvert outfall, ditch, wetland, lake, river, or floodplain), there is a risk of scour at the toe. The wall designers shall address this risk, based on the Hydraulics Engineer's assessment of the scour potential at the wall site.

It is important to differentiate between scour and stream migration. In this discussion, scour is the amount of streambed vertical elevation drop at a given location due to the removal of streambed material caused by flowing water. In accordance with LRFD-BDS Section 3.7.5 consequences of changes in foundation conditions (due to scour) shall be considered at Strength and Service Limit States.

Stream migration is a natural occurrence in some streams, and can occur slowly or rapidly, with or without accompanying scour. LRFD-BDS provides little guidance on the subject of stream migration. LRFD-BDS Section 2.6.4 states only that lateral movements of the stream shall be considered.

Information for stream migration risk shall be provided in the Preliminary Hydraulic Design Report. The Report should indicate the risk of stream migration (low or high) and
the Report may also predict how far a stream may migrate, and whether it may migrate incrementally or all at once.

Retaining walls or portions of retaining walls that are located within the scoured ground line of a stream shall be designed to resist scour as shown in Figure 8.1.9-1.

For retaining walls that are located outside of the scoured ground line, the following criteria shall be used to determine whether or not to design these adjacent retaining walls for the scour or stream migration condition:

1. If stream migration risk is classified as “Low” in the Preliminary Hydraulic Design Report, stream migration considerations for retaining walls outside of the scoured ground line may be ignored if the following two conditions are met:
   a. Scour and stream migration requirements may be ignored for retaining walls with a 10 feet or less differential between finish ground lines in front of and behind the wall.
   b. Scour and stream migration requirements may be ignored for fill-type retaining walls (SE, Geosynthetic, Gabion, Gravity Rock or Gravity Concrete Block).

2. For retaining walls that do not meet the above two conditions (1a and 1b), and for stream migration risk classified as anything other than “Low” in the Preliminary Hydraulic Design Report, the retaining walls shall be designed for scour and stream migration in the zones described in Figures 8.1.9-2 and 8.1.9-3. The Hydraulics Engineer may need to provide input on the likely limits of stream migration for these cases.

This situation will also require that abutment footings or shaft/pile caps be placed below the stream migration and scour lines, as shown in Figure 8.1.9-3, to protect the roadway fill contained behind the retaining walls.

Alternatives for designing retaining walls adjacent to bridge ends for scour and stream migration could include:

1. Increasing bridge span lengths or number of spans to move any associated retaining walls beyond the stream migration and scour lines.

2. Including revetment or scour countermeasure designs at the bridge ends, and obtaining the permits required for these features.

Both of the above mentioned approaches may allow the abutment shaft/pile caps to be placed higher, above the stream migration and scour lines.

Retaining walls built on or adjacent to WSDOT designated lifeline roadways require a reduced level of risk acceptance by WSDOT. Prevention measures for scour and channel migration such as lowering bridge abutments to protect the end fills from scour and stream migration shall be utilized. Retaining wall footings, bottom of aforementioned wall elements, and bottom of bridge footing/shaft cap elevations shall be located below the scour and stream migration lines.
Figure 8.1.9-1  Scour without Stream Migration

INTERSECTION OF FINISH GROUND LINE AND SCOUR LINE
FINAL GROUND LINE, OR TOP OF ROADWAY, AT BACK OF WALL

DESIGN THIS PORTION OF RETAINING WALL FOR SCOUR
INTERSECTION OF FINISH GROUND LINE AND RETAINING WALL

FINISH GROUND LINE
SCoured GROUND LINE

SCOUR DEPTH

BOTTOM OF RETAINING WALL ELEMENT SEE SECTION 8.1.9
Figure 8.1.9-2  Stream Migration without Scour

- **Final Ground Line, or Top of Roadway, at Back of Wall**
- **Intersection of Finish Ground Line and Migration Line**
- **Bottom of Retaining Wall Element See Section 8.1.9**
- **Design This Portion of Retaining Wall for Stream Migration**
- **Intersection of Finish Ground Line and Retaining Wall**
- **Finish Ground Line**
- **Stream Migration Line**
Figure 8.1.9-3 Stream migration WITH Scour

8.1.10 Miscellaneous Items

A. Architectural Finishes and Top of Wall Profile

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

B. 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.

For retaining walls with a fascia, the fall protection shall be located directly on top of the fascia or attached to the back face of the fascia. For retaining walls without a fascia, the fall protection shall be located within 6 inches of the face of the wall.

Fall protection shall be required regardless of the location of a traffic barrier placed behind the wall, unless the traffic barrier has a minimum height of 3'-6" and is either a moment slab traffic barrier located on top of the wall or a traffic barrier constructed integral with the top of the wall. The Standard Plan Chain Link Fence Types 3 and 4, and Glare Screen Types 1 and 2 are not considered acceptable fall protection systems.
C. **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”.

D. **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 in the wall stem shall 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 in the wall stem shall be consistent with the length determined to be adequate distribution of the traffic collision loading.

For counterfort walls, expansion joint spacing in the wall stem shall 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 in footings shall align with the expansion joints in the wall stem and shall be spaced at a maximum of 96 feet on centers. The expansion joint in the footing shall have either sleeved dowels across the joint or a shear key as described in Standard Specification Section 6-11.3(3).

Expansion joints in footings shall be provided at the interface between the retaining wall footing and the 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 preformed expansion joint through both the footing and the wall. Expansion joints at these locations do not require a shear key or sleeved dowels.

2. **Contraction Joints**

Contraction joints shall be spaced at a maximum of 30 feet for wall stems 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 96 feet. The footing construction joints should have a 6 inch minimum offset from the expansion or contraction joints in the wall stem and footing.
E. 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 8.1.10-1.

2. Follow the example format shown in Figure 8.1.10-1.


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.

F. Embankment Widening at End of Wall

The minimum clearances for the embankment at the ends of all wall types shall be as indicated on Standard Plans A-50.10 through A-50.40.
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 LRFD-BDS, LRFD-SGS, AASHTO LRFD Bridge Construction Specifications, WSDOT General & Bridge Special Provisions and the WSDOT Standard Specifications 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.

Approval by the State Bridge and Structures Architect is required on all noise barrier wall aesthetics, including finishes, materials, configuration, and top of wall profile.

8.2.2 Loads

Noise barrier walls and their components shall be designed for all applicable loads defined in the current LRFD-BDS Chapter 3.

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

Seismic load shall be as follows:

The effect of earthquake loading on noise barrier walls shall be investigated using the Extreme Event I limit states of LRFD-BDS Table 3.4.1-1 with the load factor $\gamma_p = 1.0$.

Seismic loads shall be taken to be horizontal design force effects determined in accordance with the LRFD-BDS provisions of Article 4.7.4.3.3 on the basis of the elastic response coefficient, $C_{sm}$, specified in Article 3.10.4 and BDM Section 4, and the dead load of sound barrier. The seismic design force effects for connections shall be determined by dividing the force effects resulting from elastic analysis by the response modification factor, R, specified in Table 8.2-1.

<table>
<thead>
<tr>
<th>Table 8.2-1</th>
<th>Response Modification Factors, R</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Connection</strong></td>
<td><strong>R</strong></td>
</tr>
<tr>
<td>Monolithic connection</td>
<td>1.0</td>
</tr>
<tr>
<td>Connection of precast wall to bridge barrier</td>
<td>0.3</td>
</tr>
<tr>
<td>Connection of precast wall to retaining wall or moment slab barrier</td>
<td>0.5</td>
</tr>
<tr>
<td>Connection of precast wall to shaft</td>
<td>0.8</td>
</tr>
</tbody>
</table>
8.2.3 Design

A. Standard Plan Noise Barrier Walls

1. Noise Barrier Walls detailed in Standard Plans D-2.04 through D-2.34, D-2.42 through D-2.44, D-2.48 through D-2.68 have been designed in accordance with the following criteria.


   b. The seismic design was based on a PGA of 0.35g which corresponds to a peak bedrock acceleration of 0.3g with an amplification factor of 1.18 for stiff soil.

   c. The 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 LRFD-BDS, 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 LRFD-BDS Tables 3.4.1-1 and 3.4.1-2.

   b. Seismic design is in accordance with LRFD-BDS 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 LRFD-BDS 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 LRFD-BDS Article 15.8.2 considering surface conditions characterized as “Sparse Suburban”. The 50 year return period maximum wind velocity, as determined from LRFD-BDS 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 LRFD-BDS 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 LRFD-BDS 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. The traffic barrier shown in Standard Plan D-2.46 is designed for minimum Test Level 4 (TL-4) vehicular collision loads in accordance to LRFD-BDS Article 13, and shafts are designed for an equivalent static load of 10 kips.

f. The traffic barrier shown in Standard Plan D-2.46 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 2 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 2 inches.
      - Both vertical and horizontal reinforcement shall be placed in two parallel layers.
      - Minimum clear distance between parallel layers of reinforcement shall be 2½ inches.
8.3 Buried Structures

Buried Structure is a generic term for a structure built or assembled inside an excavation employing embankment or trench methods, which works with granular backfill to derive its support from both the structure and the surrounding soil through soil-structure interaction. Buried Structure types considered herein consist of metal structural plate pipes, arches, and boxes, along with cast-in-place and precast reinforced concrete arch, box, split box, and three-sided structures.

8.3.1 General Policy

Cast-in-place or precast reinforced concrete, and metal structural plate are authorized materials for Buried Structures. Use of alternate materials, other than reinforced concrete and metal structural plate for Buried Structures having a Structural Clear Span of 20.0 feet and greater, requires approval by the WSDOT State Geotechnical Engineer and the WSDOT State Bridge Design Engineer. All Buried Structures shall be designed for a minimum service life of 75 years.

The Structural Clear Span of a Buried Structure shall be used to determine the Buried Structure Class. When supporting a Roadway, the Structural Clear Span shall be the widest horizontal opening from interior face to interior face of the end walls measured parallel to Roadway centerline. When not supporting a Roadway, the Structural Clear Span shall be the widest horizontal opening from interior face to interior face of the end walls measured perpendicular to the Buried Structure centerline.

<table>
<thead>
<tr>
<th>Structure Class</th>
<th>Structural Clear Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 1</td>
<td>Less than 20.0 feet</td>
</tr>
<tr>
<td>Class 2</td>
<td>20.0 feet and greater</td>
</tr>
</tbody>
</table>

When supporting a Roadway, the Fill Depth shall be defined as the total backfill and surfacing depth above the top of the Buried Structure. When not supporting a Roadway, the Fill Depth shall be defined as the total backfill above the top of the Buried Structure.

Consideration must be given to the degradation of Buried Structure materials resulting from corrosive and abrasive conditions. For hydraulic structures, the invert receives the largest impact due to corrosion and abrasion; however, the surrounding soil properties and groundwater may impact other portions of Buried Structures.

The use of different metals, protective linings, increased gauge thickness, or a combination of these methods are common approaches used for metal structures, and additional concrete cover or protective coatings over reinforcing steel are common approaches used for concrete structures to ensure the service life criteria is met.

Buried Structures conveying vehicles, or pedestrians shall consider the applicability of safety systems such as, but not limited to, fire life-safety elements, ventilation, lighting, emergency egress, traffic control, and communications in accordance with Section 8.3.8.

Additional provisions other than those cited herein may be specified, on a case-by-case basis, to achieve higher performance criteria for Buried Structures. Where such additional requirements are specified, they shall be site or project specific and shall be tailored to a particular structure type.
8.3.2 **WSDOT Buried Structure Templates**

The WSDOT Bridge and Structures Office (BSO) has developed standard design drawings for precast reinforced concrete Buried Structures for use as project templates. See Section 8.4 for the list of Bridge Standard Drawings for Buried Structures consisting of a geometrics table, typical sections and general details.

WSDOT’s current Buried Structure series is summarized on Bridge Standard Drawing 8.3.2-A2, comprising the following structure types:

- **Concrete Split Boxes**
  
  SB20 through SB25, see Bridge Standard Drawing 8.3.2-A3
  
  SBS20 through SBS25, see Bridge Standard Drawing 8.3.2-A13

- **Concrete Three-Sided Structures**
  
  FC20 through FC40, see Bridge Standard Drawing 8.3.2-A3
  
  VC45 through VC50, see Bridge Standard Drawing 8.3.2-A4
  
  VC55 through VC60, see Bridge Standard Drawing 8.3.2-A5

The BSO standard design drawings are templates only, and should be modified for each project per site specific conditions, design requirements, precaster capabilities, and jurisdiction.

8.3.3 **General Design Requirements**

The design of Buried Structures shall be in accordance with the requirements and guidance cited herein and in the current AASHTO LRFD Bridge Design Specifications (hereafter referred to as the LRFD-BDS), AASHTO Guide Specifications for LRFD Seismic Bridge Design (hereafter referred to as the LRFD-SGS), WSDOT Geotechnical Design Manual M 46-03, and Standard Specifications M 41-10, unless otherwise required in the project-specific criteria.

A. **Design Delivery Methods**

1. **Structural Clear Spans less than 30.0 feet**

   The Region Project Engineer Office may allow Contractor supplied designs of Buried Structure while under contract.

2. **Structural Clear Spans 30.0 feet and greater**

   The Region Project Engineer Office may utilize Contractor supplied designs of Buried Structure while under contract when approved by the WSDOT Geotechnical Office and the WSDOT Bridge and Structures Office.

   When a contractor supplied design is chosen, ensure the contract allows sufficient time from award to any construction windows to allow for design, review, and fabrication of the Buried Structure which is estimated to take 4 to 6 months.
When a contractor supplied design is not selected:

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
   • Consultant engineering staff
   • Proprietary Supplier identified as a sole source by WSDOT

B. Application of Loads

Buried Structures shall be designed for force effects in accordance with LRFD-BDS, Section 12.6.1 (as shown in Figure 8.3.3.B), except exemption from seismic loading shall not apply for Class 2 Buried Structures.

**Figure 8.3.3.B** Typical Split Box Loading Diagram

Buried Structures shall be investigated for Construction Loads, Service I Load Combination, Strength I Load Combination, and Extreme Event I and II Load Combinations, enveloping all controlling force effects resulting from, but not limited to, the following general load combinations:

- Maximum Vertical, Maximum Horizontal
- Maximum Vertical, Minimum Horizontal
- Minimum Vertical, Maximum Horizontal

The requirement of Section 3.5 for inclusion of live load in the Extreme Event I Load Combination is applicable. The load factor $\gamma_{EQ}$ as specified in LRFD-BDS, Table 3.4.1-1 shall be taken equal to 0.50, regardless of location or congestion.
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 effects of live load may be neglected for;

- A simple span (single barrel) Buried Structure, when the Structural Clear Span is less than or equal to 24.0 feet, and the minimum Fill Depth exceeds 13.0 feet.
- A simple span (single barrel) Buried Structure, when the Structural Clear Span exceeds 24.0 feet, and the minimum Fill Depth exceeds the Structural Clear Span.
- A multiple span (multiple barrel) Buried Structure, when the Fill Depth exceeds the Structural Clear Span.

C. Soil Cover

If soil cover is not provided, Buried Structures shall be designed for the direct application of vehicular loads. When the top of a concrete Buried Structure is directly exposed to vehicular traffic, bridge approach slabs shall be provided in accordance with Section 10.6, and a concrete or HMA overlay or reinforced concrete deck shall be provided.

D. Buried Structure Foundation Design

Foundations for Buried Structures shall be designed and detailed in accordance with this Bridge Design Manual M 23-50, and the *Geotechnical Design Manual*, and shall include the effects of potential scour as described in Section 8.1.9.

E. Buried Structure Wingwall, and Headwall Design

The term ‘wingwall’ as it relates to Buried Structures, is a retaining wall as defined in Section 8.1.1, used to retain the Roadway embankment adjacent to the Buried Structure, or to furnish protection against erosion.

The term ‘headwall’ is an integral structural element employed at the inlet and/or outlet of Buried Structures, as a means to retain the structural and/or Roadway fill adjacent to the structure.

Wingwalls, and headwalls for Buried Structures shall be designed in accordance with the current versions of this Bridge Design Manual, the *Geotechnical Design Manual*, and Chapter 11 of the LRFD-BDS. The bottom of wingwall foundations, and headwalls shall be located a minimum of 2.0 feet below the 500 year scour elevation in accordance with the *Geotechnical Design Manual*, Section 15-4.5, unless a greater depth is otherwise specified. The structure shall be designed for the effects of scour as described in Section 8.1.9.

Portions of wingwalls below the 100 year mean recurrence interval water surface shall be reinforced concrete or have a reinforced concrete fascia.

Headwalls shall be reinforced concrete or shall have a reinforced concrete fascia.

Headwalls shall be designed for any lateral load due to the overburden.
Headwalls, wingwalls, and railings shall be designed for vehicular collision and pedestrian or worker fall protection forces where applicable in accordance with Section 10.2 and Section 10.5.

F. Fall Protection

Fall protection shall be provided on headwalls and wingwalls in accordance with Section 8.1.10.B for exposed wall heights of 4.0 feet or more. For fall protection features that are exposed to the public, design of railings shall be in accordance with Chapter 13 of the LRFD-BDS.

G. W-Beam Guardrail on Low Fill Buried Structure (TL-3)

When Standard Plan C-20.41 guardrail is attached to a Buried Structure, the top slab and adjacent joints shall be designed for the following:

• A minimum equivalent static lateral force of 10.0 kips
• The force shall be distributed in accordance with LRFD-BDS, Figure A13.4.3.1-1.
• The center of the guardrail post shall be located a minimum of 18.0 inches away from any concrete edge, including but not limited to edges of block-outs, shear keys, and keyways.

For details see Standard Plan C-20.41 and the WSDOT Design Manual M 22-01.

The configuration shown in the Standard Plan was crash tested in 2011 by the Texas A&M Transportation Institute (TTI) following the MASH Test 3.11 specifications and reported under the Roadside Safety Research Program Pooled Fund Study No. TPF-5(114), Test Report No. 405160-23-2.

H. 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 Buried Structures. Where such additional requirements are specified, they shall be site or project specific and shall be tailored to a particular structure type.

Seismic design need not be considered for Class 1 Buried Structures.

All Class 2 Buried Structures shall be designed for seismic effects in accordance with Section 13, Seismic Considerations in the AASHTO Technical Manual for Design and Construction of Road Tunnels – Civil Elements (hereafter referred to as the Technical Manual), with current interims.
1. **Seismic Loading Effects**

Class 2 Buried Structures shall be designed in accordance with AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, 1st Edition, 2017, Sections 10.8.3 and 10.8.4 respectively to accommodate the effects resulting from two types of seismic loading:

- **Ground Shaking (i.e., transient ground displacement, TGD); and**
- **Ground Failure (i.e., permanent ground displacement, PGD)**

For TGD seismic loads, Buried Structures shall be evaluated using pseudo-static or dynamic soil-structure interaction analysis using a ground displacement approach.

The overall effects of seismically induced external earth loading on a Buried Structure causes the structure to deform with the surrounding soil or "rack". It is therefore, more reasonable to approach the problem by specifying the loading in terms of deformations. The design goal is to ensure the structure can adequately absorb the imposed racking deformation (i.e., the deformation method), rather than designing to resist a specified dynamic earth pressure (i.e., the force method). For this reasoning, the effects of transient racking or ovaling deformations on Buried Structures in soil or rock, due to the shear distortions of the ground, shall be used to determine the EQ force effects identified in Section 8.3.3.B, in accordance with the Technical Manual, Section 13.5, unless otherwise required in the project-specific criteria.

Alternately, a refined analysis utilizing a finite element approach, accounting for free-field displacement and soil-structure interaction may also be used.

For PGD seismic loads, the structural designer shall consider the potential for ground failure (e.g., liquefaction, liquefaction induced settlement, downdrag, landslides, and fault displacements) on the function of the Buried Structure.

The geotechnical designer shall evaluate the site and soil conditions to provide recommendations based on impacts of seismic geologic hazards including fault rupture, liquefaction, lateral spreading, flow failure, and slope instability, along with estimated loads and deformations acting on the structure, and options to mitigate seismic geological hazards in accordance with the *Geotechnical Design Manual*. The structural designer shall evaluate, design, and detail all elements for any geological hazards as warranted by the geotechnical engineer.
2. **Attenuation of Peak Ground Motion Parameters**

The ground motion parameters shall be derived at the elevation of the Buried Structure closest to the finished grade surface. The peak ground motion parameters shall be adjusted to reflect attenuation of ground motion with depth according to Table 8.3.3.H-2, unless detailed site-specific analysis is performed to evaluate attenuation with depth.

<table>
<thead>
<tr>
<th>Depth to Top of Buried Structure (feet)</th>
<th>Ratio of Ground Motion at Buried Structure Depth to Motion at Ground Surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 20</td>
<td>0.95 – 1.00*</td>
</tr>
<tr>
<td>20 to 50</td>
<td>0.75 – 0.95*</td>
</tr>
<tr>
<td>50 to 100</td>
<td>0.50 – 0.75*</td>
</tr>
<tr>
<td>≥ 100</td>
<td>0.50</td>
</tr>
</tbody>
</table>

*For depths between the limits of each range, corresponding ground motion attenuation ratio shall be estimated by linear interpolation – larger ratio corresponding to shallower depth.

I. **Load Rating**

All Class 2 Buried Structures shall be load rated in accordance with Section 13.

J. **Usage of Buried Structure Design Software and/or Spreadsheets**

The use of structural analysis software and/or spreadsheets shall be completely transparent and include all relevant information necessary to verify compliance with applicable design criteria. Accompanying supporting calculations may be necessary for verification. Any hidden code, function, design input, or result shall be explained thoroughly and be easily verifiable. Spreadsheets shall be unlocked. All calculations shall be logical to follow. All calculations shall include references to all applicable requirements in the design standards.

8.3.4 **Materials**

A. **Concrete**

All cast-in-place concrete shall be in accordance with Section 5.1.1.

Precast concrete shall be in accordance with Section 5.1.1. Class 5000 through 7000 are commonly used. Self-Consolidating Concrete (SCC) may be used.

Concrete cover measured from the face of concrete to the face of any reinforcing steel shall be 2.0 inch minimum at all faces.

For an HMA overlay, the minimum concrete cover from the top surface of the Buried Structure to the top mat of reinforcement shall be 2½ inches. For a concrete overlay or reinforced concrete deck, the minimum concrete cover from the top surface of the Buried Structure to the top mat of reinforcement shall be 2.0 inches.
B. Reinforcing Steel

Reinforcing steel shall be in accordance with Section 5.1.2.

The nominal yield strength for reinforcement bar shall be limited to a maximum of 80 ksi.

Welded wire reinforcement may be used to replace steel reinforcing bars in Buried Structures. Welded wire reinforcement shall be deformed and shall conform to the requirements of AASHTO M 336 (ASTM A1064). The specified minimum yield strength of welded wire reinforcement shall be limited to a maximum of 75 ksi per Section 5.1.2.I.

Prestressing steel shall be in accordance with Section 5.1.3.

When the Fill Depth of the Buried Structure is less than 2.0 feet at any point, all reinforcement in the top slab shall be corrosion resistant as defined in Section 5.1.2. Reinforcement in the top slab need not be corrosion resistant, when a 5.0 inch minimum composite, cast-in-place concrete topping, meeting the requirements for a Type 4 Bridge Protection System in accordance with Section 5.7.4 is provided.

C. Bedding Material

Foundation subgrade and Buried Structure bedding material shall be prepared in accordance with Standard Specifications, Section 7-02.3(6)A4.

The upper layer of bedding course shall be a 6.0 inch minimum thickness layer of bedding material, defined as:

1. Precast Reinforced Concrete Three-Sided Structures (PRCTSS)
   Crushed Surfacing Base Course.

2. Precast Reinforced Concrete Split Box Culverts (PRCSBC)
   Standard Specifications, Section 9-03.12(3), or to AASHTO Grading No. 57 as specified in Standard Specifications, Section 9-03.1(4)C.

3. Precast Wingwalls
   Crushed Surfacing Base Course.

D. Joint Sealant and External Sealing Bands

All flexible joints between concrete segments shall be sealed by joint sealant in accordance with Standard Specifications, Section 9-04.11.

All joints between concrete segments shall be wrapped with an external sealing band in accordance with Standard Specifications, Section 9-04.12, except that bottom slab joints are not required to be wrapped. The external sealing bands shall be installed before any tie plates.

See Section 8.4 Bridge Standard Drawings for joint sealing details.
E. Corrosion

Consideration shall be given to the degradation of Buried Structure materials resulting from corrosive conditions as defined in Section 6.7. The following corrosion mitigation efforts are commonly used to ensure service life criteria is met;

1. Metal Structural Plate Structures

Minimum corrosion rates and design service life analysis shall be in accordance with Section 6.7.2.

2. Concrete Structures

Corrosion resistant reinforcement as defined in Section 5.1.2 shall be used in Marine or Non-Marine: Corrosive environments, and additional concrete cover may also be provided. The minimum cover requirements for direct exposure to salt water and coastal situations of the LRFD-BDS shall apply.

Other corrosion protection measures to achieve a minimum service life of 75 years shall be approved by the WSDOT Bridge Design Engineer.

8.3.5 Limit States and Design Methodologies

A. Service Limit State

Service Limit State is used to satisfy stress limits, deflection, and control of cracking requirements as applicable.

1. Total and Differential Settlement

The geotechnical designer shall provide an estimated total settlement, and evaluate the potential for differential settlement between Buried Structure units, including wingwalls. The designer shall evaluate, design, and detail all elements for any settlement(s) warranted by the geotechnical engineer.

2. Deflection

Concrete structures with less than 2.0 feet of Fill Depth and top slabs that are thinner than specified in LRFD-BDS, Table 2.5.2.6.3-1 may experience excessive differential deflection of adjacent units imposed by vehicular live loads. Excessive differential deflection of the top slab can cause premature deterioration of the wearing surface such as debonding, fracturing or pavement cracking.

To mitigate differential deflection between adjacent units the minimum top slab depths stipulated in LRFD-BDS, Table 2.5.2.6.3-1 are required unless computation of deflection indicates that lesser depths may be used without adversely affecting the strength or serviceability of the structure. The vehicular deflection limits for concrete structures in accordance with LRFD-BDS, Section 2.5.2.6.2 may be used to meet these requirements.
For concrete structures where the top slab is less than two feet from the Roadway surface, the design shall provide a method of shear transfer between the top slabs of adjacent units to equalize deflections by incorporating at least one of the following:

- Provide a structural connection between adjacent units capable of transferring the imposed shear and equalizing deflections. The structural connection shall include cast-in-place reinforced concrete closures or grouted shear keys.
- Provide a 5.0 inch minimum composite cast-in-place concrete topping, meeting the requirements of a Type 4 Protection System in accordance with Section 5.7.4.

Arch-top structures, because of their geometry and interaction with the surrounding soil, do not exhibit significant differential deflections that could cause pavement cracking for structures with less than 2.0 feet of fill. Thus, the requirements for thickness, differential deflection, and shear transfer between adjacent units does not apply.

For top slabs thinner than 1/20 of the Structural Clear Span, consideration should be given to prestressing in the direction of that Structural Clear Span in order to control cracking.

3. **Control of Cracking**

Reinforcement shall be provided and spaced to meet the requirements of LRFD-BDS, Section 5.6.7. The exposure factor shall be based upon a Class 2 exposure condition.

Temperature and Shrinkage reinforcement shall be provided and spaced to meet the requirements of LRFD-BDS, Section 5.10.6. Temperature and Shrinkage reinforcement shall be provided in fillets and/or haunches.

B. **Strength Limit State**

Strength Limit State is used to satisfy flexural, shear, thrust, and radial tension requirements as applicable.

C. **Extreme Limit State**

Extreme Limit State is used to satisfy seismic, check flood, and scour requirements as applicable.

D. **Boundary Conditions**

Rigid frames are statically indeterminate structures, and as such, require more rigorous analysis than statically determinate structures. This has led to some simplifying assumptions to facilitate rapid computations, which do not capture the inherent advantages of rigid frames, such as continuity, stiffness, and economy, creating inefficiencies in the design.
Rigid frame members designed assuming simply supported boundary conditions, produce conservative reinforcement requirements in positive moment regions, and leave negative moment regions with deficiencies in reinforcement requirements. This is not an efficient design approach, because the assumed boundary conditions do not capture the essence of the structure's behavior. Therefore, to accurately capture a rigid frame's behavior and eliminate possible reinforcement deficiencies, concrete three-sided structures and box structures shall be analyzed as a rigid frame, applying appropriate boundary conditions.

Additionally, the bottom slabs of box structures are entirely supported by the underlying bedding material. Often times for ease of computations, the bottom slab of these structures are analyzed as a simply supported beam. Again, this does not capture the structure's behavior efficiently, and has a tendency to leave reinforcement deficiencies in negative moment regions. Analyzing the slab supported on an elastic foundation is a more appropriate approach. This type of analysis can be achieved using the modulus of subgrade reaction, as determined by the geotechnical engineer, to determine a spring constant, and applying a series of compression springs along the bottom slab of the structure. To effectively capture this soil-structure interaction behavior, the bottom slab of box structures shall be analyzed on an elastic foundation.

Alternately, assuming a uniform or trapezoidal support reaction, can sufficiently capture the soil-structure interaction behavior for the design of the bottom slab of box structures.

E. Structural Modeling

1. Three-Sided Structures

   Should be modeled as a rigid frame, chorded arch, or arch with pin or fixed support reactions as applicable.

2. Split Box Structures

   Should be modeled as shown in Figure 8.3.5.E-2.

   There are two primary boundary condition functions that need to be addressed within the model. The first function should be to maintain global stability, therefore the bottom left node is restrained in the global x direction. The second function should be to provide displacement-dependent resistance to vertical loads by supporting the structure with compression springs.

   The hinges shown in the model corresponds to the joints between the upper and lower segments. Due to the behavior of a shiplap joint, capable of transferring shear in only one direction but not moment, the shear output from both joints should be added together and applied to a single joint for design.
Figure 8.3.5.E-2 Split Box on an Elastic Foundation Model

Nodes should be placed at points of interest such as corners, fillet/uniform cross-section interface locations, and midspan of members. Additional nodes should be placed along the bottom element of the model, and restrained using compression springs in the global y direction.

The spring constant (stiffness of ground spring) in the beam-spring model is used in the development of the compression springs and shall be computed by multiplying the moduli of subgrade reaction with the tributary area at the corresponding node as follows:

\[ K = k_s b \]

Where:
- \( K \) = Compression Spring Constant
- \( k \) = Modulus of Subgrade Reaction
- \( s \) = Tributary Length Associated with a Node
- \( b \) = Unit Slab Width

3. **Split Box Structures with Top Slab**

Should be modeled as shown in Figure 8.3.5.E-2, except that the hinges with moment release shown should be replaced with transverse springs to represent elastomeric bearing pads, or pins as applicable, and the upper unit should be replaced with a flat slab.

Alternatively, the upper flat slab unit (superstructure) may be analyzed using PGSuper with the bearing reactions applied to the lower unit as an external axial load on a per foot basis. Additionally, in lieu of a racking analysis, earthquake loads, and bearing shear forces could be applied to the lower unit as described in Section 7.5.4.D, and 7.5.4.E respectively.
8.3.6 **Provisions for Structure Type**

**A. Concrete Box and Split Box Structures**

Concrete Boxes are four-sided rigid frame structures. Split Boxes consist of either a rigid three-sided frame lid, or flat top slab called the upper unit, and a rigid three-sided frame base or lower unit.

Concrete box structures shall be designed and constructed in accordance with *Standard Specifications, Section 7-02.3(6).*

1. **Precast Geometric Limitations**

Formwork used in the precast industry for rigid three-sided frame structures with span lengths ranging from 8.0 feet up to 35.0 feet, contain geometric limitations that should be considered when establishing a structure’s geometry. The following are not proven design ranges, they are recommendations from the precast industry based on their form capabilities and reference Figure 8.3.6.A-1:

- \( S = \) Span Length, shall be varied in 1'-0" increments from a minimum of 8'-0" to a maximum of 35'-0".
- \( W_1 = \) Wall Thickness at the Fillet, is typically either 10.0" or 12.0", and tapers to \( W_2 = \) Wall Thickness at the Joint of upper unit, or \( W_3 = \) Wall Thickness at the Joint of lower unit respectively, typical taper is 2H:103V.
- \( H_1 = \) Height of Tapered Wall of upper unit, and \( H_2 = \) Height of Tapered Wall of lower unit, shall be varied in 1'-0" increments from a minimum of 0'-0" to a maximum of 10'-0". If project limitations require a dimension between 1'-0" increments, the designer is encouraged to contact precast manufacturers for available options.
- \( T_T = \) Thickness of Top Slab, and \( T_B = \) Thickness of Bottom Slab, shall be varied in 2.0" increments from a minimum of 10.0" to a maximum of 24.0".
- \( F = \) Fillet, shall be 17.0" x 17.0".

Skewed units shall be limited to skew angles of 45 degrees or less, and be varied in 1.0 degree increments.

- \( L_W = \) Lay Width, typically varies in 1'-0" increments from a minimum of 4'-0" to a maximum of 10'-0".

If the lay width of the individual precast segments is not critical on a project, the designer is encouraged to provide a minimum lay width of the segments, along with an overall lay width of structure to allow fabricators to choose lay widths of individual segments based on their form capabilities and shipping requirements (typical shipping weight limits are 60-65 kips).
Figure 8.3.6.A-1  Typical Split Box, Comprising Two Precast Three-Sided Frames

L = Length (Out To Out)
L_W = Lay Width Or 'Lay Length'
S = Span Length
H_U = Height Of Upper Unit
H_L = Height Of Lower Unit
H_1 = Height Of Tapered Wall
H_2 = Height Of Tapered Wall
W_1 = Wall Thickness At Fillet
W_2 = Wall Thickness At Joint
W_3 = Wall Thickness At Joint
T_T = Thickness Of Top Slab
T_B = Thickness Of Bot. Slab
F = Fillet Height & Width

LONGITUDINAL JOINT
VERTICAL JOINT
TRANSVERSE JOINT
2. Distribution of Live Load through Earth Fill

The distribution of wheel loads through earth fill shall be in accordance with LRFD-BDS, Section 3.6.1.2.6 as follows;

Where the depth of fill is:

- Less than 2.0 feet, live load shall be distributed to the top slab in accordance with LRFD-BDS, Section 4.6.2.10 as axle loads.
- 2.0 feet or greater, live load shall be distributed to the top slab in accordance with LRFD-BDS, Section 3.6.1.2.6 as wheel loads and shall consider interaction effects.

3. Joint Design and Details

Joints shall be designed to carry the applied horizontal and vertical forces, and so formed that they can be assembled to transmit those forces, and provide joint tightness consistent with tolerances outlined in the Contract Documents.

- **Vertical Joints** (See Figure 8.3.6.A-1):
  Joints between adjacent units shall be designed for the applied lateral forces identified in Section 8.3.3.B.

- **Horizontal Joints** (See Figure 8.3.6.A-1):
  Transverse joints between the upper units and lower units shall be designed for the applied lateral forces identified in Section 8.3.3.B.
  Longitudinal joints in the bottom slab shall be designed for the applied vertical forces identified in Section 8.3.3.B, in addition to any differential settlement as warranted by the geotechnical engineer.
  Longitudinal joints in the top slab shall be designed for the applied vertical forces identified in Section 8.3.3.B, in addition to any differential settlement as warranted by the geotechnical engineer, and shall incorporate a method of shear transfer between adjacent units in accordance with Section 8.3.5.A-2 as applicable.

All joints shall be fabricated in accordance with *Standard Specifications*, Section 7.02.3(6)C, employing a bell & spigot (a.k.a. a tongue & groove, or shiplap) connection. See Bridge Standard Drawing 8.3.2-A10 for joint details.

Alternate joint types may be proposed addressing applied forces, differential settlement between segments, live load deflection, shear transfer, and prevention of water and soil migration through the joint. If an alternate joint design is accepted it shall have equal or greater capacity as the joint shown on the Contract Plans. The designer shall provide a note on the Plans stating the capacity of the joint detailed.

Example Note:

“This joint has been detailed and designed to resist ___ kips of shear per linear foot of joint”.

"This joint has been detailed and designed to resist ___ kips of shear per linear foot of joint".
Each joint shall be sealed to prevent exfiltration or infiltration of soil fines or water. Field tests may be required by the Engineer whenever there is a question regarding compliance. See Bridge Standard Drawing 8.3.2-A8 for joint sealing details.

As shown in Figure 8.3.6.A-3, joints shall be detailed such that the laying of sections on the prepared bedding material should start at the outlet, and with the bell (or groove) end pointing upstream or ahead on stationing, and the spigot (or tongue) end pointing downstream or back on stationing, with installation proceeding toward the inlet end to facilitate placement during construction and prevent undermining of downstream units at the joints for hydraulic structures in service.

Figure 8.3.6.A-3 Precast Box Joint Details to Facilitate Installation

The orientation of the bell and spigot should alternate between the bottom units and the top units. This alternating orientation is preferred, as it provides the most flexibility to facilitate staged construction requirements, and has an inherent placement advantage during construction.

If staging is not necessary, projects may incorporate an optional alternate detail to allow alignment of the vertical bell and spigot joint between the top and bottom units. The installation sequence is this aligned configuration should be such that the bottom segments are installed in the direction of stationing, whereas the top units should be installed in the reverse direction to alleviate constructability issues from trying to slide a bell end under a spigot.
B. Concrete Three-Sided Structures

Three-Sided Structures are rigid frame, chorded arch, and arch structures which may have open inverts, and are supported by concrete foundation units consisting of a footing and possibly a stem wall.

Three-Sided Structures shall be designed and constructed in accordance with Standard Specifications, Section 7-02.3(6).

1. Precast Geometric Limitations

Rigid Three-Sided Structures with span lengths ranging from 8.0 feet up to 35.0 feet, contain the same geometric limitations that should be considered when establishing a structure’s geometry as outlined in Section 8.3.6.A-1.

Three-Sided Structures with geometries matching WSDOT’s current Buried Structure series FC30 – FC40, and VC45 – VC60 should maintain the geometrics identified on Bridge Standard Drawing 8.3.2-A2. If project limitations require alternate dimensions, the designer is encouraged to contact precast manufacturers for available options.

2. Distribution of Live Load through Earth Fill

The distribution of wheel loads through earth fill shall be in accordance with LRFD-BDS, Section 3.6.1.2.6 as follows;

Where the depth of fill is:

• Less than 2.0 feet, live load shall be distributed to the top slab in accordance with LRFD-BDS, Equation 4.6.2.10 as axle loads.
• 2.0 feet or greater, live load shall be distributed to the top slab in accordance with LRFD-BDS, Section 3.6.1.2.6 as wheel loads and shall consider interaction effects.

3. Joint Design and Details

Joints shall be designed to carry the applied horizontal and vertical forces, and so formed that they can be assembled to transmit those forces, and provide joint tightness consistent with tolerances outlined in the Contract Documents.

• Vertical Joints (See Figure 8.3.6.A-1):
  Joints between adjacent units shall be designed for the applied lateral forces identified in Section 8.3.3.B.

• Horizontal Joints (See Figure 8.3.6.A-1):
  Transverse joints between the upper units and foundation units shall be designed for the applied lateral forces identified in Section 8.3.3.B, employing a shear key, block restrainer, or dowel bars. See Bridge Standard Drawing 8.3.2-A6 for connection details.
  Longitudinal joints in the top slab shall be designed for the applied vertical forces identified in Section 8.3.3.B, in addition to any
differential settlement as warranted by the geotechnical engineer, and shall incorporate a method of shear transfer between adjacent units accordance with Section 8.3.5.A-2 as applicable.

Portal units shall be designed for any lateral load due to the overburden.

Alternate joint types may be proposed addressing applied forces, differential settlement between segments, live load deflection, shear transfer, and prevention of water and soil migration through the joint. If an alternate joint design is accepted it shall have equal or greater capacity as the joint shown on the Contract Plans. The designer shall provide a note on the Plans stating the capacity of the joint detailed.

Example Note:

“This joint has been detailed and designed to resist ___ kips of shear per linear foot of joint”.

Each joint shall be sealed to prevent exfiltration or infiltration of soil fines or water. Field tests may be required by the Engineer whenever there is a question regarding compliance.

C. Design of Metal Structural Plate Structures

Metal structural plate structures considered herein comprise pipe, arch and box structures defined as:

- **Structural Plate Pipe**: A steel or aluminum structural plate around the entire circumference of a pipe shape. Structural plate pipes may contain multiple radiiuses and plate thicknesses. Structural plate pipe shapes include but are not limited to round, ellipse, underpass, pipe-arch and pear.

- **Structural Plate Arch**: A steel or aluminum structural plate arch shape placed on reinforced concrete foundations. Structural plate arches may contain multiple radiiuses and plate thicknesses.

- **Structural Plate Box**: A steel or aluminum structural plate box shape that meets the requirements or the LRFD-BDS, Section 12.9 placed on reinforced concrete foundations.

Design and construction of metal structural plate structures shall conform to the LRFD-BDS, Section 12, and the AASHTO LRFD Bridge Construction Specifications, Section 26.

Steel structural plate shall not be used in locations conforming to Marine or Non-Marine: Corrosive environments as defined in Section 6.7.1.

Minimum backfill cover over the top of the Buried Structure shall be in accordance with the LRFD-BDS.

Where aluminum will contact concrete or grout, two coats of paint shall be applied to the aluminum at the contact surface in accordance with Standard Specifications, Section 7-08.3(2)D.
8.3.7 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 in accordance with the AASHTO LRFD Bridge Design Specifications (LRFD-BDS) 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 $\gamma_{WA} = 1.25$ in LRFD-BDS, 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, in accordance with LRFD-BDS, 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 LRFD-BDS, Section 3.4.2.

In certain situations tie-downs may be required to resist buoyancy forces. The resisting force ($R_n$) and resistance factors ($\varphi$) 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:

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

Where:

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

$R_{UPLIFT} = | \gamma_{WA} WA |$
ACI 350-06 has stricter criteria for cover and spacing of joints than the LRFD-BDS. 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 LRFD-BDS, Section 5.6.7 with $\gamma_e = 0.5$ (in order to maintain a crack width of 0.0085 inches, in accordance with the commentary of 5.6.7).

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 1200 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 36 inch in diameter, have ladders that extend to the vault floor, and shall be designed to resist HL-93 live 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 LRFD-BDS, 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 Projects Unit. 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.

8.3.8 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, AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, 1st Edition, 2017 with current interims and AASHTO 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. NFPA 502 – Standard for Road Tunnels, Bridges, and Other Limited Access Highways (NFPA 502). This document shall be used for all WSDOT tunnels. NFPA 502, 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

### 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

### Soil Nail Wall
- **8.1-A4-1** Soil Nail Wall, 1 of 4
- **8.1-A4-2** Soil Nail Wall, 2 of 4
- **8.1-A4-3** Soil Nail Wall, 3 of 4
- **8.1-A4-4** Soil Nail Wall, 4 of 4

### Noise Barrier
- **8.1-A5-1** Noise Barrier on Bridge

### Cable Fence
- **8.1-A6-1** Cable Fencing for Wall
- **8.1-A6-2** Cable Fencing for Wall w/Top Mounted Base
- **8.1-A6-3** Cable Fence Details 1 of 3
- **8.1-A6-4** Cable Fence Details 2 of 3
- **8.1-A6-5** Cable Fence Details 3 of 3

### 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-B1** Precast Split Box Buried Structure Design Criteria

**Appendix 8.3-B2** 3-Sided Precast Buried Structure Design Criteria

**Appendix 8.3-B3** Soil Interaction Analysis for Culvert Structures Precast Split Box Buried Structure
## Appendix 8.1-A1 Summary of Design Specification Requirements for Walls

<table>
<thead>
<tr>
<th>Wall Types</th>
<th>Design Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pre-Approved Proprietary Structural Earth Walls</strong></td>
<td></td>
</tr>
<tr>
<td><strong>General</strong></td>
<td>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, WSDOT Geotechnical Design Manual (GDM) and WSDOT Bridge Design Manual (BDM).</td>
</tr>
<tr>
<td><strong>Seismic</strong></td>
<td>AASHTO Guide Specifications for LRFD Seismic Bridge Design using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 yrs (1000 yr Return Period) and the site coefficients or site-specific procedure provided in the WSDOT Design Memorandum dated January 8, 2017 and WSDOT BDM Chapter 4.</td>
</tr>
<tr>
<td><strong>Traffic Barrier</strong></td>
<td>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.</td>
</tr>
</tbody>
</table>

| **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 Guide Specifications for LRFD Seismic Bridge Design using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 yrs (1000 yr Return Period) and the site coefficients or site-specific procedure provided in the WSDOT Design Memorandum dated January 8, 2017 and WSDOT BDM Chapter 4. |
| **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 Geosynthetic Walls** |
| **General** | Current Standard Plan walls are designed in accordance with AASHTO LRFD Bridge Design Specifications 5th Edition 2010 and interims through 2011 and the WSDOT GDM through 2011. |
| **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 BDM and the AASHTO LRFD Bridge Design Specifications Section A13.3 for Concrete Railings considering a minimum TL-4 impact load. |

<p>| <strong>Non-Standard Geosynthetic Walls</strong> |
| <strong>General</strong> | Design shall be based on current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM. |
| <strong>Seismic</strong> | AASHTO Guide Specifications for LRFD Seismic Bridge Design using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 yrs (1000 yr Return Period) and the site coefficients or site-specific procedure provided in the WSDOT Design Memorandum dated January 8, 2017 and WSDOT BDM Chapter 4. |
| <strong>Traffic Barrier</strong> | Special design barriers to be constructed on Standard Plan or Non-Standard Geosynthetic Walls 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. |</p>
<table>
<thead>
<tr>
<th>Wall Types</th>
<th>Design Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Concrete Cantilever Walls</td>
<td><strong>General</strong> 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, WSDOT GDM and WSDOT BDM.&lt;br&gt;&lt;br&gt;<strong>Seismic</strong> AASHTO Guide Specifications for LRFD Seismic Bridge Design using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 yrs (1000 yr Return Period) and the site coefficients or site-specific procedure provided in the WSDOT Design Memorandum dated January 8, 2017 and WSDOT BDM Chapter 4.&lt;br&gt;&lt;br&gt;<strong>Traffic Barrier</strong> WSDOT BDM 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.</td>
</tr>
<tr>
<td>Soldier Pile Walls With and Without Tie-Backs</td>
<td><strong>General</strong> Design shall be based on current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM.&lt;br&gt;&lt;br&gt;<strong>Seismic</strong> AASHTO Guide Specifications for LRFD Seismic Bridge Design using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 yrs (1000 yr Return Period) and the site coefficients or site-specific procedure provided in the WSDOT Design Memorandum dated January 8, 2017 and WSDOT BDM Chapter 4.&lt;br&gt;&lt;br&gt;<strong>Traffic Barrier</strong> 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.</td>
</tr>
<tr>
<td>Non-Standard Noise Barrier Walls</td>
<td><strong>General</strong> Design shall be based on current editions, including current interims, of the following documents; AASHTO LRFD Bridge Design Specifications, WSDOT GDM and WSDOT BDM.&lt;br&gt;&lt;br&gt;<strong>Seismic</strong> AASHTO Guide Specifications for LRFD Seismic Bridge Design using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 yrs (1000 yr Return Period) and the site coefficients or site-specific procedure provided in the WSDOT Design Memorandum dated January 8, 2017 and WSDOT BDM Chapter 4.&lt;br&gt;&lt;br&gt;<strong>Traffic Barrier</strong> WSDOT BDM and the AASHTO LRFD Bridge Design Specifications Section A13.3 for Concrete Railings considering a minimum TL-4 impact load.</td>
</tr>
<tr>
<td>Wall Types</td>
<td>Design Specifications</td>
</tr>
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</tbody>
</table>
| **Soil Nail Walls**  | **General**  
All soil nail walls and their components shall be designed using the publication “Geotechnical Engineering Circular No. 7” FHWA-NHI-14-007. 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 Guide Specifications for LRFD Seismic Bridge Design using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 yrs (1000 yr Return Period) and the site coefficients or site-specific procedure provided in the WSDOT Design Memorandum dated January 8, 2017 and WSDOT BDM Chapter 4.  
**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.                                                                                                                                                                                                                                                                                                                                 |
| **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 Guide Specifications for LRFD Seismic Bridge Design using the USGS 2014 Seismic Hazard Maps with Seven Percent Probability of Exceedance in 75 yrs (1000 yr Return Period) and the site coefficients or site-specific procedure provided in the WSDOT Design Memorandum dated January 8, 2017 and WSDOT BDM Chapter 4.  
**Traffic Barrier**  
WSDOT BDM and the AASHTO LRFD Bridge Design Specifications Section A13.3 for Concrete Railings considering a minimum TL-4 impact load.                                                                                                                                                                                                                                                                                                                                 |
Chapter 8 Walls and Buried Structures

8.99 References


5. *Design Manual* M 22-01

6. Geotechnical Design Manual M 46-03

7. Standard Plans M 21-01


18. NFPA 502, Standard for Road Tunnels, Bridges, and Other Limited Access Highways.