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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 pertaining to retaining walls.

Standard designs for noise barrier walls (precast concrete), 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 State Geotechnical Office. 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 State Geotechnical Office, 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, or State Geotechnical Office.

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 pertaining to retaining walls.

All other retaining walls not covered by the Standard Plans such as reinforced concrete 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 Office.

The State Hydraulics 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 pertaining to retaining walls, which provide valuable information on the design of retaining walls.

8.1.2 Common Types of Retaining Walls

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

1. Proprietary Structural Earth (SE) Walls – *Standard Specifications* Section 6-13.
2. Geosynthetic Walls (Temporary and Permanent) – *Standard Plan* D-3 and *Standard Specifications* Section 6-14.
3. Standard Reinforced Concrete Retaining Walls - *Standard Plans* D-10.10 through D-10.45 and *Standard Specifications* Section 6-11.
4. Standard Precast Reinforced Concrete Retaining Wall – Standard Plan D-20.10 and General Special Provision 6-11.
5. Soldier Pile Walls and Soldier Pile Tieback Walls – *Standard Specifications* Sections 6-16 and 6-17.
6. Soil Nail Walls–*Standard Specifications* Section 6-15.

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

8.1.2.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 State Geotechnical Office 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:

8.1.2.A.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 State Geotechnical Office. 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*. For additional information see the retaining walls chapter in the *Design Manual* M 22-01 and *Geotechnical Design Manual* chapter pertaining to retaining walls. For the SEW shop drawing review procedure see *Geotechnical Design Manual* chapter pertaining to retaining walls.

8.1.2.A.2 *Other Proprietary Walls*

Other proprietary wall systems such as crib walls, bin walls, or precast 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*. 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 State Geotechnical Office and the Bridge and Structures Office Preliminary Plans Unit must approve the concept prior to development of the PS&E.

8.1.2.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.

8.1.2.C Reinforced Concrete Walls

Reinforced concrete walls are typically rigid gravity or semi-gravity walls as defined in LRFD-BDS Article 11.6. Semi-gravity walls are suitable for heights up to 35 feet. Standard Plans D-10.10 to D-10.45 provide details for construction and the maximum bearing pressure in soil of WSDOT designed cast-in-place semi-gravity walls. Standard Plan D-20.10 provides details for construction and the maximum bearing pressure in soil of WSDOT designed precast semi-gravity walls.

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

8.1.2.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 pertaining to retaining walls. See Bridge Standard Drawing 8.1-A3 for typical soldier pile wall details.

8.1.2.E Soil Nail Walls

The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing steel bars called “nails” into a slope or excavation as construction proceeds from the “top down”. Soil nailing is a technique used to stabilize moving earth, such as a landslide, or as temporary shoring. Soil anchors are used along with the strength of the soil to provide stability. The Geotechnical Engineer designs the soil nail system whereas the Bridge and Structures Office designs the wall fascia. See Bridge Standard Drawing 8.1-A4 for typical soil nail wall details.

8.1.3 General Design Considerations

Design of retaining walls shall be in accordance with the requirements and guidance cited herein and in the current LRFD-BDS, LRFD-SGS, WSDOT General and Bridge Special Provisions, the *Standard Specifications* M 41-10 and the *Geotechnical Design Manual* M 46-03 unless otherwise cited herein. See Appendix 8.1-A1 for a summary of design specification requirements for walls.

Seismic design of all retaining walls shall be in accordance with LRFD-BDS and Section 4.5.

All construction shall follow procedures as outlined in the WSDOT *Standard Specifications* latest edition.

The Geotechnical Engineer will provide the earth pressure diagrams and other geotechnical design requirements. Pertinent soil data will also be provided for preapproved proprietary structural earth walls (SEW), non-standard reinforced concrete retaining walls, and geosynthetic walls.

Retaining walls and their components that are in service for a maximum of 36 months are considered to be temporary. Temporary retaining walls need not be designed for the Extreme Event I Limit State.

The live load factor for Extreme Event-I Limit State load combination, γ_{EQ} as specified in the AASHTO LRFD Table 3.4.1-1 for all permanent retaining walls shall be taken equal to 0.50.

8.1.4 Design of Reinforced Concrete Retaining Walls

8.1.4.A Standard Reinforced Concrete Retaining Walls

Standard Walls are those walls for which standard designs are provided in the WSDOT *Standard Plans*.

The Standard Plan cast-in-place reinforced concrete retaining walls have been designed in accordance with the requirements of the LRFD-BDS 4th Edition 2007 and interims through 2008.

The Standard Plan precast reinforced concrete retaining walls have been designed in accordance with the requirements of the LRFD-BDS 9th Edition 2020.

The Standard Plan reinforced concrete retaining walls have not been designed for hydrostatic pressure due to water accumulating behind the wall.

Reinforced concrete retaining walls containing design parameters which exceed those used in the standard reinforced concrete retaining wall design are considered to be non-standard reinforced concrete retaining walls.

8.1.4.A.1 *Standard Cast-In-Place Reinforced Concrete Retaining Wall Types 1 through 8*

1. The design parameters used to complete the design are summarized below and in Table 8.1.4-1.
2. Extreme Event stability of the walls were based on 100 percent of the wall inertia force combined with 50 percent of the seismic earth pressure.
3. Active Earth pressure distribution was linearly distributed per Section 7.7.4.

4. 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).
5. Passive Earth pressure distribution was linearly distributed. For Types 1, 2, 7, and 8 the passive earth pressure was taken over the depth of the footing. For Types 3, 4, 5, and 6 the passive earth pressure was taken over the depth of the footing and the height of the shear key.
6. 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.
7. 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.
8. Wall Types 1, 2, 7, and 8 have not been designed for 42-inch traffic barrier height collision forces. The archived 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.

Table 8.1.4-1 Standard Cast-In-Place Reinforced Concrete Retaining Wall Design Parameters

Standard Plan #	Wall Type	Effective Peak Ground Acceleration	Ka	Kp	Kae
D-10.10	Type 1	0.51g	0.24	1.5	0.43
D-10.15	Type 2	0.51g	0.24	1.5	0.43
D-10.20	Type 3	0.32g	0.36	1.5	0.94
D-10.25	Type 4	0.32g	0.36	1.5	0.94
D-10.30	Type 5	0.20g	0.36	1.5	0.55
D-10.35	Type 6	0.20g	0.36	1.5	0.55
D-10.40	Type 7	0.20g	0.24	1.5	0.30
D-10.45	Type 8	0.20g	0.24	1.5	0.30

8.1.4.A.2 *Standard Precast Reinforced Concrete Retaining Wall*

1. The design parameters used for Standard Plan D-20.10 are summarized below and in the table and General Notes provided in the Standard Plan.
2. The retained fill was assumed to have an angle of internal friction of 34 degrees and a unit weight between the range of 125 to 145 pounds per cubic foot.
3. For the footing without a key, the coefficient of sliding was based on a friction angle of 36 degrees for sliding stability considering the six inches of bedding material required to be placed under the precast footing. For the footing with a key the coefficient of friction was based on the native subgrade angle of internal friction shown in Table 8.1.4-2b and in the table provided in the Standard Plan.
4. The design for walls having a flat backslope considered a 2 ft of equivalent height of soil for vehicular surcharge loading. The design for walls having a backslope steeper than 10H:1V considered a 1 ft equivalent height of soil for vehicular surcharge loading. The unit weight of the equivalent height of soil was 125pcf.

5. For the Service, Strength, and Extreme Event I limit states, the design considered a hydrostatic pressure due to water accumulating behind the wall up to the bottom of the weep hole. For the Extreme Event II limit state, the design considered a hydrostatic pressure due to water accumulating behind the wall up to one-half the wall height “H”.

6. The design considered a 200 lb lateral force applied at 42 inches above the top of the wall to account for the application of a worker fall protection system as described in Section 8.1.11.B.

7. The static and seismic active earth pressures were distributed as shown in Figure 8.1.4-1 and Figure 8.1.4-2 and applied at an angle of δ = 23 degrees. The active lateral earth pressure coefficients, Ka, are shown in Table 8.1.4-2a.

8. The seismic active earth pressure coefficient, K_{AE}, provided in the Standard Plan table considered the precast reinforced concrete wall is free to move a minimum of 1 to 2 inches laterally when under the influence of seismic loading. The Geotechnical Engineer shall determine the site-specific K_{AE} value considering the seismic conditions, slope configuration, and subsurface conditions to determine applicability of the Standard Plan.

9. The Passive earth pressure was distributed over the footing depth and where applicable, the height of the footing key. The passive lateral earth pressure coefficients, Kp, for the soil in front of the footing toe and the footing key are shown in Table 8.1.4-2b.

Table 8.1.4-2a Design Static Lateral Earth Pressure Coefficient

Standard Plan #	Wall Type	Ka
D-20.10	Type 1	0.25
	Type 2	0.25
	Type 3	0.41
	Type 4	0.41
	Type 5	0.41

Table 8.1.4-2b Design Passive Horizontal Earth Pressure Coefficient

Standard Plan #	Native Subgrade ϕ (DEG.)	Footing Toe Kp	Footing Key Kp
D-20.10	30	7.4	5.4
	34	7.4	7.4
	38	7.4	10.4

8.1.4.B Non-Standard Reinforced Concrete Retaining Walls

Differential grade concrete barriers are considered to be rigid gravity retaining walls and shall be designed in accordance with this BDM, See Section 8.1.3.

Wingwalls and headwalls are considered to be retaining walls and shall be designed in accordance with this BDM, See Section 8.1.3.

Precast reinforced concrete retaining walls shall have a 6 inch minimum thickness layer of bedding material as defined in *Standard Specification* Section 6-20.3(6)A.

8.1.4.B.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.

8.1.4.B.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.

1. 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).
2. For the broken back backfill condition, the slope angle β^* is based on the LRFD-BDS Figure C3.11.5.8.1-1.
3. The wall backfill interface friction angle is $\delta = 2/3\ \phi_f$ but not greater than β or β^* which is consistent with the Coulomb wedge theory.

8.1.4.B.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, γ_p, for EH is 1.0 to account for the dynamic nature of the collision load.

8.1.4.B.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. 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 7.7.4.F 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.

8.1.4.B.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

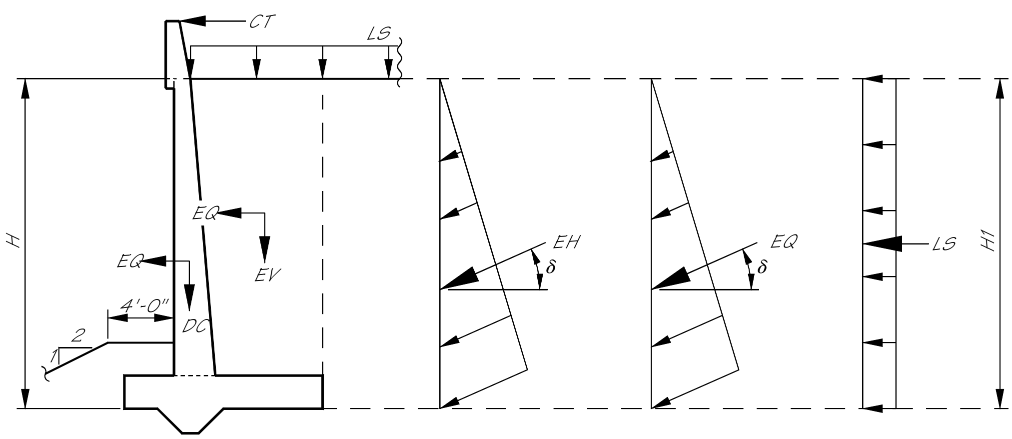


Figure 8.1.4-2 Application of Lateral Loads for walls with a sloping backfill

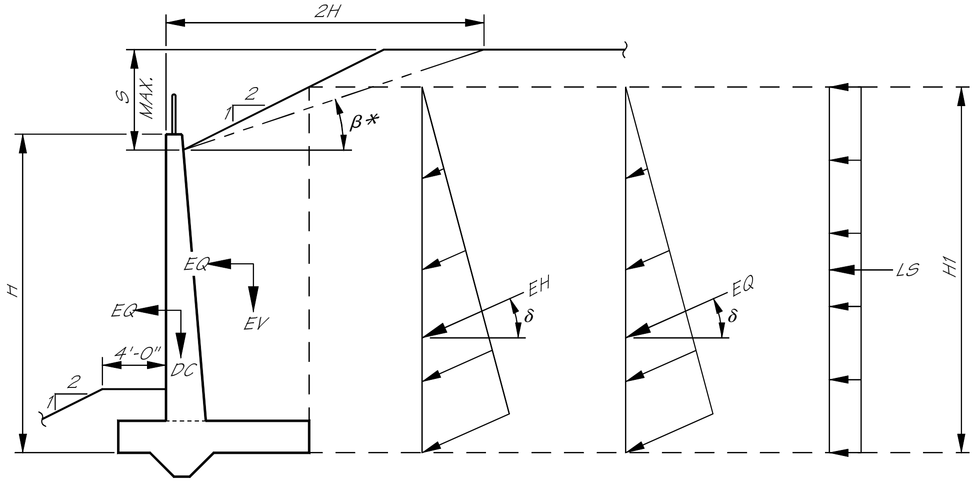


Figure 8.1.4-3 Application and Distribution of Vehicular Collision Load occurring near the midsection

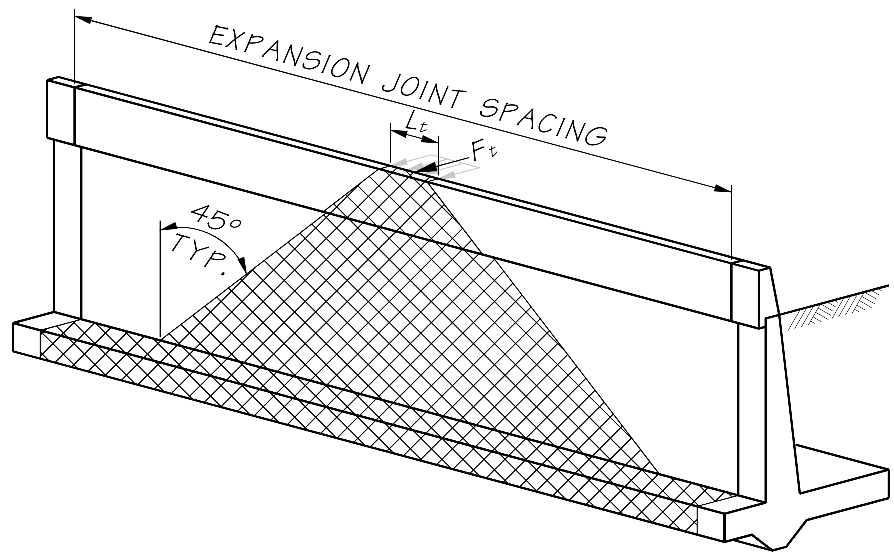
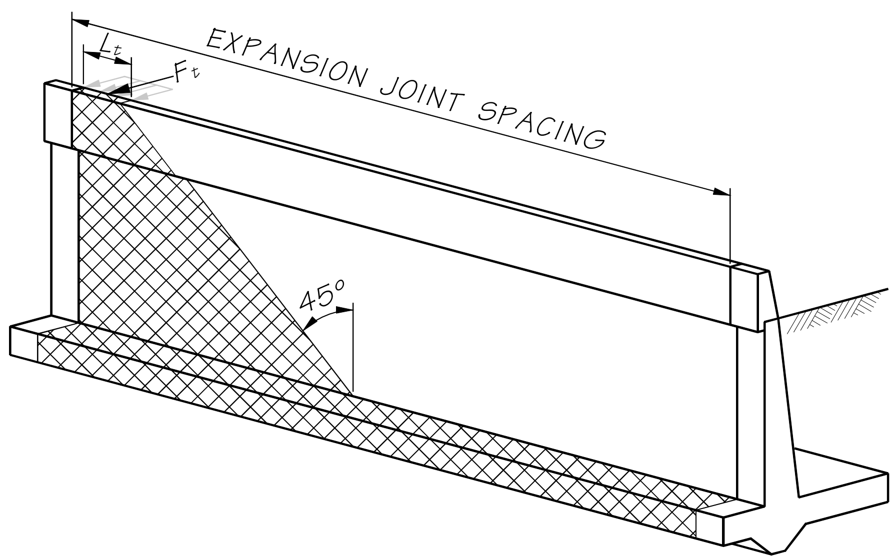


Figure 8.1.4-4 Application and Distribution of Vehicular Collision Load occurring near the end.



8.1.5 Design of Cantilever Soldier Pile and Soldier Pile Tieback Walls

Typical soldier pile wall details are provided in the Appendix 8.1-A3.

8.1.5.A Ground Anchors (Tiebacks)

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

The anchor may consist of bars 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 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 minimum vertical anchor spacing shall be the same as defined in LRFD-BDS Section 11.9.4.2 for the minimum horizontal anchor spacing. 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 anchor lock-off load is 60 percent of the controlling factored design load for temporary and permanent walls (see *Geotechnical Design Manual* chapter pertaining to retaining walls).

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-A3.

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.

8.1.5.B Design of Soldier Pile (discrete vertical elements)

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.

8.1.5.B.1 *Application of Lateral Loads*

1. Lateral loads acting on the back of the wall for the portion of the wall located above the base of excavation, are applied over an effective width equal to the center-to-center pile spacing. These lateral loads result from horizontal earth pressure, live load surcharge, seismic earth pressure, or any other applicable load.
2. Lateral loads acting on the back of the pile for the portion of the pile located below the base of excavation, are applied over an effective width equal to the shaft diameter. These lateral loads result from horizontal earth pressure, seismic earth pressure or any other applicable load.
3. Passive earth pressure is applied over an effective width equal to the smaller of three times the shaft diameter or the center-to-center pile spacing.

8.1.5.B.2 *Determining Depth of Pile Embedment*

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

1. 10 feet
2. As recommended by the Geotechnical Engineer of Record
3. As required for skin friction resistance and end bearing resistance.
4. As required to satisfy factored horizontal force equilibrium and factored moment equilibrium about the bottom of the soldier pile for cantilever soldier piles without permanent ground anchors.
5. 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.

8.1.5.B.3 *Soldier Pile Shaft Backfill*

Specify controlled density fill (CDF, 145 pcf) or pumpable lean concrete (lean concrete Type 1) 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 (lean concrete Type 1) for the full height of the soldier pile shaft when shafts are anticipated to be excavated and concrete placed in the wet.

Specify Class 4000P in the embedded portion of the shaft for soldier pile walls that support large vertical loads such as bridge foundations or high permanent ground anchor loads. Class 4000P shall not extend above the bottom of the lagging or permanent fascia.

8.1.5.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. If construction operations are likely to occur above and behind the soldier pile wall alignment, the lagging shall be designed for an additional 250 psf surcharge due to temporary construction load. The lateral pressure transferred from a moment slab shall be considered in the design of soldier pile walls and laggings.

All lagging support details shall be shown in the Contract Plans including unique supports details for wall angle points and lagging adjacent to other structures such as culverts, bridge abutments and curtain walls.

8.1.5.C.1 *Temporary Timber Lagging*

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

8.1.5.C.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 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:

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

   And,

2. The lagging is visible for inspections during this life cycle.

8.1.5.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;

1. a 2 feet minimum below the finish ground line adjacent to the face of the wall.
2. 3 feet minimum below the lowermost PGA.
3. 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. Shotcrete fascia panels may not be suitable in areas where the fascia will be exposed to chlorides. If the fascia may be exposed to chlorides, the reinforcement shall be corrosion resistant and the reinforcement clear cover shall be increased.

8.1.6 Design of Structural Earth Walls

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 unless the project specific assessment criteria and approval as specified in the *Geotechnical Design Manual* is met.
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.

8.1.6.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 State Geotechnical Office. A list of current pre-approved proprietary wall systems and their limitations is provided in the *Geotechnical Design Manual*. For the SE wall shop drawing review procedure, see the *Geotechnical Design Manual*.

8.1.6.B Non-Preapproved Proprietary Structural Earth Walls

Structural earth walls that exceed the limitations as provided in the *Geotechnical Design Manual* are considered to be non-preapproved. Use of non-preapproved structural earth walls shall require the approval of the State Geotechnical Engineer and the State Bridge and Structures Engineer.

8.1.7 Design of Geosynthetic Walls

8.1.7.A Standard Geosynthetic Wrapped Face Walls

Standard Walls are those walls for which standard designs are provided in the WSDOT *Standard Plans*.

The design criteria for Standard Plan D-3.09 is provided in the *Geotechnical Design Manual*.

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.

8.1.7.B Non-Standard Geosynthetic Walls

The fascia for non-standard geosynthetic wrapped face walls shall be designed for forces such as, inertia forces on the fascia due to a seismic event, external soil pressure not contained within the geosynthetic wrapped face wall, forces from elements attached to the fascia, and applicable wind loads. If a dowel connection similar to that shown in Standard Plan D-3.10 is utilized, the connection shall be designed considering the following.

1. The dowel connection shall be designed considering the shear friction pullout of the dowel from the soil in the reinforced soil zone. The contribution of passive soil bearing resistance on the hooked portion of the dowel shall be neglected.
   A. The friction coefficient for the soil against steel shall be 0.4. This value is based on the F* value provided in LRFD-BDS Figure 11.10.6.3.2-2 for smooth steel strips.
   B. The surface area of the dowel utilized for shear friction pullout shall be 2/3 of the 75 year design life corroded surface area.

2. The dowel connection design shall consider other failure mechanisms as well such as dowel tension and shear failure, pullout from the fascia, crushing of the concrete fascia, etc.

Fascia having a thickness greater than 6 inches shall have 2 mats of vertical and horizontal reinforcement placed in two parallel layers.

8.1.8 Design of Soil Nail Walls

Soil nail walls shall be designed in accordance with LRFD-BDS Section 11.12.

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-A4.

Design of the cantilevered portions of the soil nail wall facing which typically occurs at the top, bottom, or ends of the wall shall be in accordance with the requirements of LRFD-BDS Section 11.6 for conventional retaining walls.

8.1.9 Design of Shaft Walls

Shaft retaining walls are often referred to as cylinder pile walls, secant pile walls, or tangent pile walls. This wall type may be comprised of closely spaced structural concrete shafts or closely spaced structural concrete shafts with interlocking or tangent non-structural shafts.

8.1.10 Scour of Retaining Walls

The foundation for all walls constructed along rivers and streams shall be evaluated during design by the Hydraulics Engineer for total scour in accordance with LRFD-BDS and Hydraulic Engineering Circular No. 23 (HEC-23). 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 total scour at the check flood elevation in accordance with the *Geotechnical Design Manual* 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 total 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 total scour potential at the wall site.

Total Scour includes the amount of streambed vertical elevation drop at a given location due to the removal of streambed material caused by flowing water and the effects of lateral migration. In accordance with LRFD-BDS Section 3.7.5 and Section 2.6.4 consequences of changes in foundation conditions (due to total scour) shall be considered at Extreme Event, Strength, and Service Limit States.

Retaining walls or portions of retaining walls that are located within the total scour at scour check flood line of a stream shall be designed to resist scour as shown in Figures 8.1.10-1 and 8.1.10-2.

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

1. Increasing bridge span lengths or number of spans to move any associated retaining walls beyond the total scour at scour check flood line.
2. Including revetment or scour countermeasure designs at the bridge ends, and obtaining the permits required for these features. See Figure 8.1.10-3.
   A. All revetement or scour countermeasures shall be designed and constructed in accordance with the most current version of the HEC-23.
   B. The estimated scour groundline above the scour countermeasure shall extend from the top of the scour countermeasure at an angle equal to the angle of repose. The bottom of wall foundation or element shall be located a minimum of 2 feet below the estimated scour groundline as shown in Figure 8.1.10-3.

Retaining walls associated with Bent-Type Abutments as shown in Figure 7.5.1-4, Isolated Abutments as shown in Figure 7.5.1-5, and Abutments Supported by MSE walls as shown and described in Section 7.5.2, shall be designed for total scour at the design flood. For these situations, the bottom of the retaining wall element or foundation shall be located a minimum of 2 feet below the total scour at scour check flood line as shown in Figures 8.1.10-1 and 8.1.10-2. The permissible modifications to the location of the bottom of wall foundation or element due to the addition of revetments or scour countermeasures does not apply to these design scenarios.

Figure 8.1.10-1 Scour without Lateral Migration

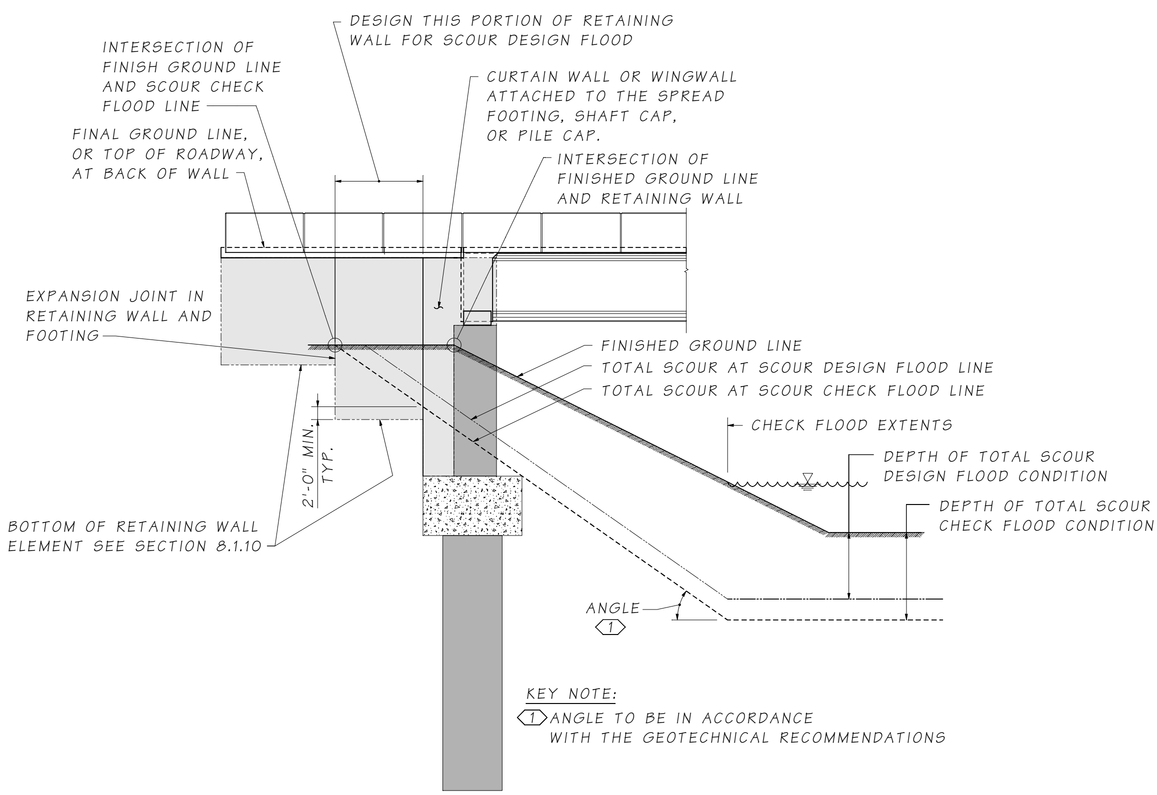


Figure 8.1.10-2 Scour with Lateral Migration

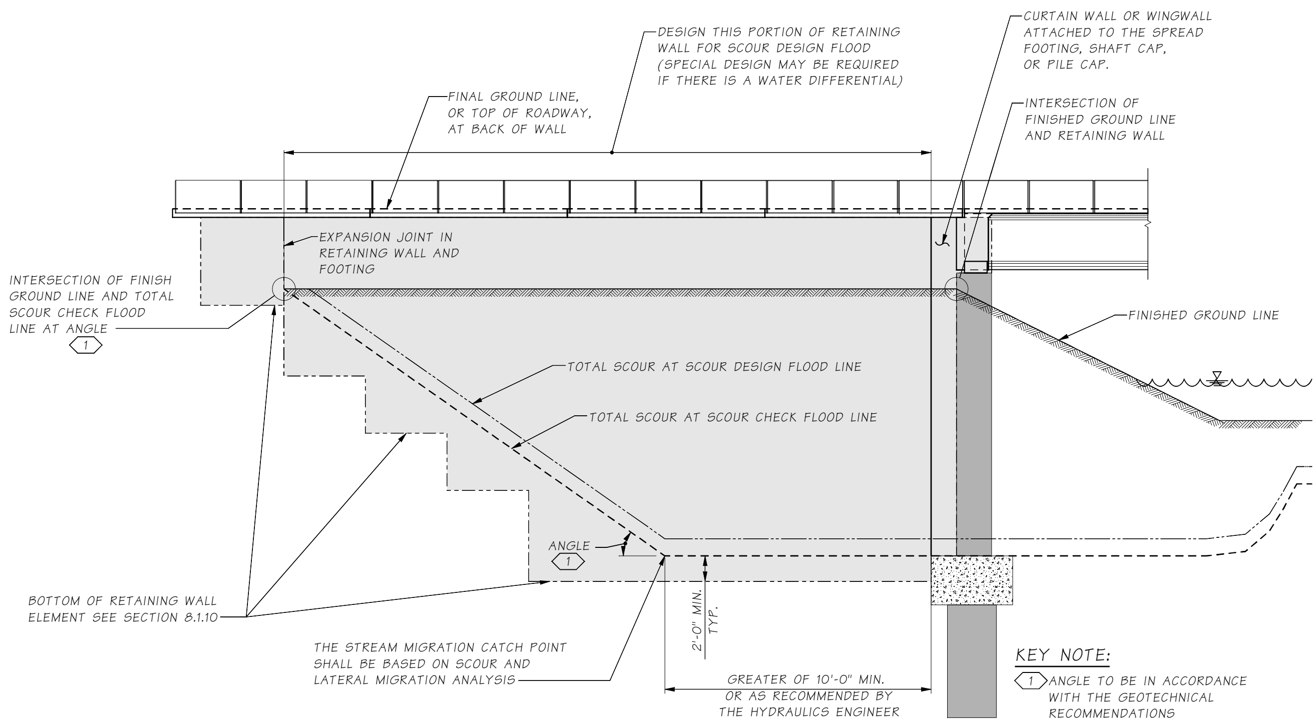
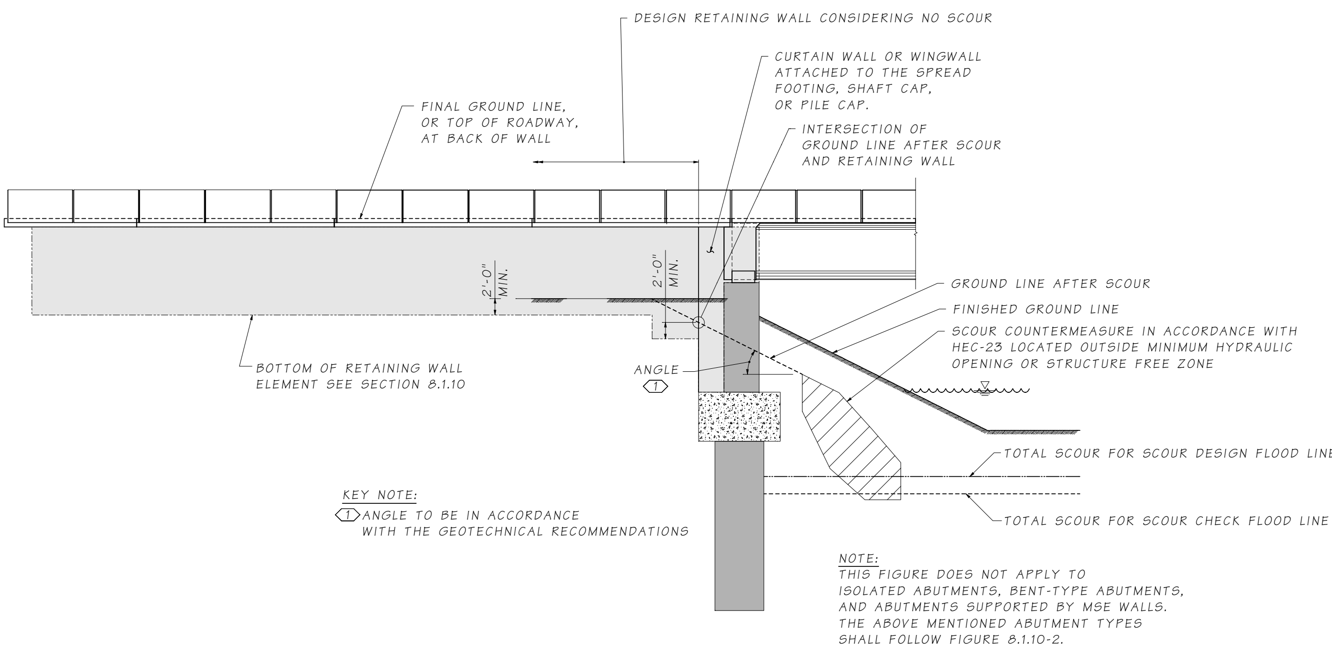


Figure 8.1.10-3 Scour with Lateral Migration and Scour Countermeasures



8.1.11 Miscellaneous Items

8.1.11.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.

8.1.11.B Worker Fall Protection

For retaining walls with exposed wall heights of 4 feet or more, worker fall protection shall be provided in accordance with WAC 296-880 and as described in the *Design Manual* Chapter 1060.

For retaining walls with a fascia, the worker 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 worker fall protection shall be located within 6 inches of the face of the wall.

Worker 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 following *Standard Plans* are designed and detailed to meet worker fall protection requirements:

- L-5.10 Bridge Railing Type Chain Link Pile Rail
- L-5.15 Cable Rail

Other Standard Plan fence types are not considered acceptable worker fall protection systems.

8.1.11.C Pedestrian and Bicycle Railings for Fall Protection

Fall protection systems that are intended to be used by the public or intended to provide a physical guide for pedestrians and bicyclists to minimize the likelihood of a pedestrian or bicyclist falling over the system shall be designed in accordance with Chapter 13 of the LRFD-BDS.

Pedestrian and bicycle railings for fall protection shall be located as described above for worker fall protection.

8.1.11.D Drainage

Drainage features shall be detailed in the Plans.

All retaining walls shall have concrete gutters constructed behind the back of the wall unless a moment slab traffic barrier is located on top of the wall, a traffic barrier is attached to the top of the wall, or a sidewalk is located immediately behind the wall.

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*).

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. 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.

All reinforced concrete retaining walls, wingwalls, headwalls, semi-gravity, and gravity walls shall have a vertical layer of gravel backfill for walls placed behind the wall for a distance equal to the width of the footing heel or 3 feet minimum. See Standard Plan D-4.

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”.

8.1.11.E Expansion, Contraction and Construction Joints

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

Precast concrete retaining wall units (stem and footing) shall be connected with shear keys in accordance with *Standard Specifications* Section 6-11.3(3), grouted shear keys, ship lap joints, or weld tie joints. All joints shall be covered with a 1 foot wide sealing band on the retained soil side of the wall. The sealing band shall extend from the bottom of the footing to the finished ground line behind the wall.

8.1.11.E.1 *Expansion Joints*

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

For semi-gravity and rigid gravity walls, expansion joint spacing in the wall stem shall be a maximum of 60 feet on centers. For semi-gravity and rigid 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 semi-gravity 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 cast-in-place footings shall align with the expansion joints in the wall stem and shall be spaced at a distance determined by the design engineer.

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 premolded expansion joint through both the footing and the wall stem.

8.1.11.E.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.

8.1.11.E.3 *Construction Joints*

Construction joints are only permitted in the footing. The maximum spacing of construction joints in the footing shall be 120 feet. The footing construction joints shall have a 6 inch minimum offset from the expansion or contraction joints in the wall stem.

8.1.11.F Detailing of Standard Reinforced Concrete Retaining Walls

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

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

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

   The value for “H” shall be shown in a block in the center of the panel or segment. See Example, Figure 8.1.10-1.

2. Follow the example format shown in Figure 8.1.11-1.
3. Calculate approximate quantities using the Standard Plans.
4. Wall dimensions shall be determined by the designer using the Standard Plans.
5. Do not show any details given in the Standard Plans.
6. Specify in the Plans all deviations from the Standard Plans.
7. Do not detail reinforcing steel, unless it deviates from the Standard Plans.
8. For pile footings, use the example format with revised footing sizes, detail any additional steel, and show pile locations. Similar plan details are required for footings supported by shafts.

8.1.11.G 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.

8.1.11.H Wall Face Embedment

At the face of the wall, all retaining walls shall have a 10H:1V or flatter horizontal bench at least 4 feet wide.

The face of all retaining walls shall have a 2 feet minimum embedment depth or as required for scour, or as required by the *Geotechnical Design Manual* or as required by the Geotechnical Engineer. The embedment depth shall also meet the requirements of LRFD-BDS 11.10.2.2 and Table C11.10.2.2-1.

8.2 Noise Barrier Walls

8.2.1 General

Design of noise barrier walls shall be based on the requirements and guidance cited herein and in the current LRFD-BDS, LRFD-SGS, 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.

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

Noise Barrier Walls either exposed to traffic, attached to a barrier, or within the clear zone require roadside safety hardware considerations and should be designed and detailed accordingly.

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 γ_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 8.2-1 Response Modification Factors, R

Connection	R
Monolithic connection	1.0
Connection of precast wall to bridge barrier	0.3
Connection of precast wall to retaining wall or moment slab barrier	0.5
Connection of precast wall to shaft	0.8

8.2.3 Design

8.2.3.A Standard Plan Noise Barrier Walls

1. 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 are 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. Drilled shaft foundations are designed for an equivalent static traffic impact load of 10 kips.

F. The traffic barrier shown in Standard Plan D-2.46 could be either precast or cast-in-place The concrete barrier shall be MASH compliant with a railing test level of TL-4 or less.

G. These Standard Plan noise barrier walls have not been designed to accommodate a soil differential between the front and back of the wall panel.

8.2.3.B Non-Standard Noise Barrier Walls

Noise barrier walls containing design parameters which exceed those used in the standard noise barrier wall design shall be 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 State 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 or grade at the top of the retaining wall 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 specific 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. As such, this type of Structure necessitates a differentiation from the term 'culvert' as defined in *Standard Specifications*, Section 7-02, even when the Structure is conveying hydraulics.

Buried Structure types considered herein consist of metal structural plate pipes, arches, and boxes, along with composite concrete filled arches, cast-in-place and precast reinforced concrete arches, boxes, split boxes, and three-sided Structures.

8.3.1 General Policy

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

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.

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

Structure Class	Structural Clear Span
Class 1	Less than 20.0 feet
Class 2	20.0 feet and greater

Buried Structures can be even further classified using the Structure's expected dynamic response, categorized as either a Flexible or Rigid Structure type. Flexible Buried Structures include metal structural plate pipes, arches, and boxes as defined in Section 8.3.6.C. Rigid Buried Structures include composite concrete filled arches, cast-in-place and precast reinforced concrete arch, box, split box, and three-sided Structures. Precast, prestressed concrete elements would also be considered Rigid with respect to Buried Structures.

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.

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.

Architectural finishes shall be coordinated with the State Bridge and Structures Architect in accordance with the WSDOT *Design Manual*, and as otherwise shown in the Plans.

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 Templates and Standard Plans

8.3.2.A Buried Structure Templates

The WSDOT Bridge and Structures Office (BSO) has developed precast reinforced concrete standard design drawings for Buried Structures to use as project templates. See Section 8.4 for the complete list of Bridge Standard Drawings available for Buried Structures.

WSDOT's current precast reinforced concrete Buried Structure series is summarized on Bridge Standard Drawing 8.3-A1-1, comprising the following Structure types:

- **Split Box Structures**

  SB20 through SB25, see Bridge Standard Drawing 8.3-A2-1

- **Three-Sided Structures**

  FC20 through FC40, see Bridge Standard Drawing 8.3-A3-1

  VC45 through VC50, see Bridge Standard Drawing 8.3-A3-2

  VC55 through VC60, see Bridge Standard Drawing 8.3-A3-3

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

8.3.2.B Buried Structure Standard Plans

The WSDOT BSO has developed Buried Structure *Standard Plans* for the following:

- **E-20.10** Buried Structure – Split Box
- **E-20.20** Buried Structure – Three-Sided
- **D-20.10** Precast Reinforced Concrete Retaining Walls (for wingwalls)

Additionally, the following *Standard Plans* may be used in conjunction with the aforementioned Standards as applicable:

- **A-40.50** Bridge Approach Slab
- **C-20.40** Beam Guardrail Type 31 Placement 12'-6", 18'-9", or 25'-0" Span
- **C-20.41** Box Culvert Embedded Anchor Guardrail Steel Post - Type 31
- **C-20.43** Box Culvert Bolt-Thru Anchor Guardrail Steel Post - Type 31
- **C-81.10** 42" Single Slope Barrier on Structure (TL-4).
- **D-3.09** Permanent Geosynthetic Wall (for wingwalls and headwalls)
- **L-5.10** Bridge Railing Type Chain Link Pipe Rail (for worker fall protection)
- **L-5.15** Cable Fence (for worker fall protection)

Refer to Section 8.3.3.K for the basis of design utilized in developing the Buried Structure Standard Plans. Refer to Section 8.1.4.A.2 for the basis of design utilized in developing the Precast Reinforced Concrete Retaining Wall Standard Plan. The user shall verify the applicability of employing these *Standard Plans* for the given project's site conditions and design requirements.

The Buried Structure Standard Plans include options for various components, which should be considered for the given project. The designer may need to specify specific options within the Standard Plan that are needed or required for the project.

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 LRFD-BDS, LRFD-SGS, WSDOT *Geotechnical Design Manual*, and *Standard Specifications*, unless otherwise required in the project-specific criteria.

8.3.3.A Design Delivery Methods

8.3.3.A.1 *Contractor Supplied Design*

The Region Project Engineer may allow Contractor Supplied Designs for Buried Structures with a Structural Clear Span less than or equal to 30 feet in accordance with these design requirements, including a quality check in accordance with Section 1.4.6. For Structural Clear Spans greater than 30 feet, Contractor Supplied Designs may be used when approved by the State Geotechnical Office and the State Bridge Design Office.

Contractor Supplied Design of Buried Structures shall meet all project specific design requirements and site constraints.

The Contractor may reference the Buried Structure Standard Plans for Structural Clear Spans up to and including 30 feet, as applicable, given the project's site conditions and design requirements are met.

When a Contractor Supplied Design is selected, the Region Project Engineer shall ensure the contract allows sufficient time, from Award to any construction windows, accommodating design, review, and fabrication of the Buried Structure. Typical procurement times are estimated to take 4 to 6 months.

8.3.3.A.2 *Agency Supplied Design*

For Buried Structures designed by the Agency, a Preliminary Plan shall be completed in accordance with the criteria listed in Chapter 2.

Additionally, the structural design shall be completed prior to Advertisement and the Plans shall be included as part of the Ad Ready PS&E.

Agency Supplied Designs shall be completed by one of the following:

- WSDOT Engineering Staff
- Consultant Engineering Staff
- Proprietary Supplier identified as a Sole Source by WSDOT
- Buried Structure Standard Plans, as applicable, given the project's site conditions and design requirements are met.

8.3.3.A.3 *Design Build*

When Design Build is selected, the Contractor shall meet all design requirements and site constraints in accordance with the Project Documents.

The Contractor may reference the Buried Structure Standard Plans for Structural Clear Spans up to and including 30 feet, as applicable, given the project's site conditions and design requirements are met.

See Section 15.8.3 for specific design provisions pertaining to a Design Build delivery method.

8.3.3.B Application of Loads

Buried Structures shall be designed for force effects in accordance with LRFD-BDS, Article 12.6.1, except exemption from seismic loading shall not apply, but rather, shall be applicable in accordance with Section 8.3.3.H.

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

- Max. vertical load on top, Max. lateral inward load on walls.
- Max. vertical load on top, Min. lateral inward, Max. outward, load on walls.
- Min. vertical load on top, Max. lateral inward load on walls.
- Min. vertical load on top, Min. lateral inward, Max. outward, load on walls.

This is summarized on Bridge Standard Drawings 8.3-A1-3, *Split Box General Load Combinations* and 8.3-A1-4, *Three-Sided Structure General Load Combinations*.

8.3.3.B.1 *Live Load, LL*

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

Where the Fill Depth is less than 2.0 feet at any point above the Buried Structure, and the Structure is supporting a Roadway, it shall be considered directly exposed to vehicular traffic.

When directly exposed to vehicular traffic, the following applies:

- Live Load shall be distributed directly to the top surface of the Structure, ignoring the effects of Live Load distribution through the Fill Depth.
- A method of shear transfer, capable of transferring the imposed shear and deflection between adjacent units, shall be provided (see Section 8.3.5.A.2).
- All reinforcement in the top unit shall be corrosion resistant as defined in Section 5.1.2, except when a 5.0-inch minimum composite, Cast-In-Place (CIP) concrete topping, meeting the requirements for a Type 4 Bridge Protection System in accordance with Section 5.7.4 is provided.
- A concrete or HMA overlay, or a 5.0-inch minimum reinforced concrete topping slab shall be provided.

The application of the Design Lane Load as defined in LRFD-BDS, Article 3.6.1.2.4 may be excluded, providing a Multiple Presence Factor of 1.2 is applied on the Live Load.

The decrease in Live Load effect due to increase in Fill Depth (distribution of wheel load through earth fill), shall be considered in both design and load rating of Buried Structures with Fill Depths 2.0 feet and greater.

The effects of live load may be neglected in accordance with LRFD-BDS, Article 3.6.1.2.6a.

8.3.3.C Deck Protection and Approach Slabs

When an HMA overlay is provided, a waterproofing membrane in accordance with *Standard Specifications,* Section 6-08 shall be installed. If a 6.0-inch minimum Base or Top Course is placed between the Structure's top unit and HMA, the waterproofing membrane may be omitted.

For an HMA overlay, the minimum concrete cover from the top surface of the Structure to the top mat of reinforcement shall be 2½ inches.

For a concrete topping, the minimum concrete cover from the top surface of the concrete topping to the topping's top mat shall be 2½ inches.

Bridge approach slabs shall be provided in accordance with Section 10.6.

8.3.3.D Buried Structure Foundation Design

Foundations for Buried Structures shall be designed and detailed in accordance with this *Bridge Design Manual*, and the *Geotechnical Design Manual*, and shall include the effects of potential scour as described in Section 7.1.7 and Section 8.1.10.

8.3.3.E Buried Structure Wingwall and Headwall Design

The term 'wingwall' as it relates to Buried Structures, is a retaining wall as defined in Section 8.1.4.B, used to retain the Roadway embankment adjacent to the Buried Structure, or to furnish protection against erosion.

The term 'headwall' is a structural element employed as an end treatment at the inlet and/or outlet of Buried Structures, as a means to retain the structural and/or Roadway fill adjacent to the Structure comprising, at a minimum, parapets, slope collars, and cut-off walls.

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. Alternatively, Standard Plan D-3.09 or Standard Plan D-20.10 may be employed, provided the user verifies the applicability of the Standard Plan(s) for the given project's site conditions and design requirements.

The bottom of wingwall foundations shall be located, a minimum of 2.0 feet below the total scour at the scour check flood elevation, in accordance with the *Geotechnical Design Manual* unless a greater depth is otherwise specified. The Structure shall be designed for the effects of scour as described in Section 8.1.10.

Portions of wingwalls below the water surface of the Hydraulic Design Flood Elevation shall be reinforced concrete or have a reinforced concrete fascia. The Hydraulic Design Flood Elevation will be established in the Hydraulics Report and is based on either the 100-year or the 2080 projected 100-year flood elevation.

Headwalls shall be reinforced concrete or shall have a reinforced concrete fascia, and shall be designed for any lateral load due to the overburden.

Headwalls, wingwalls, barriers, 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.

8.3.3.F Worker, Pedestrian and Bicycle Fall Protection

For Buried Structures and associated headwalls and wingwalls, worker, pedestrian and bicycle fall protection shall be provided in accordance with the WSDOT *Design Manual*, and shall be designed in accordance with Section 8.1.11.

Timber shall not be used as permanent fall protection material. Rigid fall protection systems shall not be used within the 'roadway clear zone' as described in the WSDOT *Design Manual*.

8.3.3.G W-Beam Guardrail on Buried Structures (TL-3)

When Standard Plan C-20.41 or Standard Plan C-20.43 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, or Standard Plan C-20.43, and the WSDOT *Design Manual*.

The configuration shown in Standard Plan C-20.41 was crash tested in 2011 by the Texas A&M Transportation Institute (TTI) following the MASH 2009 Test Designation No. 3-11 and reported under the Roadside Safety Research Program Pooled Fund Study No. TPF-5(114), Test Report No. 405160-23-2.

The configuration shown in Standard Plan C-20.43 was crash tested in 2017 & 2018 by the Midwest Roadside Safety Facility (MwRSF) following the MASH 2016 Test Designation No. 3-10 and 3-11 and reported under MwRSF Test Report No. TRP-03-383-20-R1.

8.3.3.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. Where such additional requirements are specified, they shall be site or project specific, and shall be tailored to a particular Structure type.

All Buried Structures shall be evaluated and designed for the Extreme Event I Limit State including the effects of seismic loading and all geotechnical seismic hazards, with the following exceptions:

i. Class 1 – Rigid and Flexible Buried Structures need not consider internal structural design (racking) of the Buried Structure elements.

ii. Class 2 – Flexible Buried Structures less than or equal to 30 feet, with shapes as defined in Section 8.3.6.C, need not consider internal structural design (racking) of the Buried Structure elements.

All Class 2 – Flexible Buried Structures with custom or project specific shapes outside of those as defined in Section 8.3.6.C shall consider internal structural design (racking) of the Buried Structure elements.

Where the Structure crosses an active fault, these exceptions shall not apply.

Seismic design and analysis for Buried Structures shall be performed in accordance with the:

- BDM
- LRFD-BDS
- *Geotechnical Design Manual*
- Section 10.8 of the AASHTO LRFD *Road Tunnel Design and Construction Guide Specifications*, 1st Edition, 2017
- Section 13 - *Seismic Considerations* of the AASTHO *Technical Manual for Design and Construction of Road Tunnels – Civil Elements*, June 2010, hereinafter referred to as the *'Technical Manual'*

Seismic design and analysis for wingwalls and headwalls shall be required and performed in accordance with the:

- *Geotechnical Design Manual*
- BDM
- LRFD-BDS

8.3.3.H.1 *Seismic Loading Effects*

Buried Structures shall be designed to accommodate the effects resulting from two types of seismic loading, except as excluded above:

Ground Shaking (i.e., transient ground displacement, TGD); and

Ground Failure (i.e., permanent ground displacement, PGD)

For TGD seismic loads, Buried Structures, except split-box ~ slabs considered directly exposed to vehicular traffic, 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 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 TGD seismic loads, split-box ~ slab Buried Structures considered directly exposed to vehicular traffic shall be designed using conventional force-based methods. The bottom unit ('U' shaped frame element) shall be designed for Active, or At-Rest Seismic Earth

Pressures as applicable, and the top unit shall be designed as a simple span superstructure in accordance with the LRFD-BDS. Longitudinal and transverse restraint of the top unit shall be provided as applicable.

For PGD seismic loads the geotechnical designer shall evaluate the site and soil conditions to provide recommendations based on impacts of seismic geologic hazards including fault rupture, liquefaction, liquefaction induced settlement, downdrag, 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.

8.3.3.H.2 *Load Combinations for Transient Seismic Motion*

The effects of vertically propagating shear waves perpendicular to a Buried Structure's longitudinal axis produces two independent deformations in the plane of the Structure's cross-section that shall be considered: (1) Horizontal racking or ovaling deformations, and (2) Inertia forces due to vertical seismic motions.

Therefore, these multi-directional effects of the seismic ground motion shall be applied using a percentage combination method. This method accounts for the simultaneous occurrence of earthquake forces in two perpendicular (horizontal and vertical) directions. The percentage combination is accomplished by considering two separate load cases as follows:

Load Case 1: $EQ = \pm EQ_{\text{Horiz.}} \pm \gamma(EQ_{\text{Vert.}})$

Load Case 2: $EQ = \pm \gamma(EQ_{\text{Horiz.}}) \pm EQ_{\text{Vert.}}$

Where, γ is assumed to be 0.30 for rectangular Structures, and 0.40 for circular Structures.

The seismic loads due to racking deformations and vertical seismic motions shall then be combined with non-seismic loads using the load combination and load factors for the Extreme Event I Limit State.

8.3.3.H.3 *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 may be adjusted to reflect attenuation of ground motion with depth according to Table 8.3.3.H-3, unless detailed site-specific analysis is performed to evaluate attenuation with depth.

Table 8.3.3.H-3 Ground Motion Attenuation with Depth (Modified after AASHTO, 2010)

Depth to Top of Buried Structure (feet)	Ratio of Ground Motion at Buried Structure Depth to Motion at Ground Surface (Attenuation Factor, R_{GMA})
≤ 20	0.95 – 1.00*
20 to 50	0.75 – 0.95*
50 to 100	0.50 – 0.75*
≥ 100	0.50

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

The vertical seismic inertia force, $EQ_{Vert.}$, shall be taken as ⅔ of the Effective Peak Horizontal Ground Acceleration, A_s, times the appropriate Attenuation Factor, R_{GMA}, times the sum of the dead load of the top unit and the vertical earth load on top of the top unit.

8.3.3.I Load Rating

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

8.3.3.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, and shall include references to all applicable requirements in the design standards.

The *Eriksson Culvert* software developed by Eriksson Software, Inc. may be used for the design and load rating of box, split box, and split box ~ slab Structures, along with the top unit (inverted 'U' shaped frame element) of three-sided Structures, provided '*WSDOT*' is selected from the dropdown menu as the *Agency Recommendations* in the *Project Settings*' window. The Engineer shall verify all the *Agency Recommendations*, listed in the dialog box for '*WSDOT*', are appropriate for the specific project and all design and load rating requirements of this manual are satisfied.

WSDOT has internally reviewed the software and found it to be in compliance with the requirements of this manual, with the following exceptions that should be considered and addressed by the designer:

1. The software does not analyze transverse Differential Settlement (SE), which will need to be analyzed independently and accounted for in the final design. Settlement need only be considered for the Service Limit State.

2. The software, when considering external hydrostatic pressure, doesn't reduce the Horizontal Soil Pressure, *EH*, to a buoyant weight. The engineer can conservatively include both the hydrostatic and dry soil pressures together or use alternative methods to account for buoyant soil weight.

   The software also provides users with input options to envelope the Horizontal Earth Load using a minimum and a maximum Equivalent Fluid Pressure. The designer shall not use Equivalent Fluid Pressure to envelope the Horizontal Earth Load, but rather shall use a site-specific Earth Pressure Coefficient, K_a or K_o as applicable, provided by the geotechnical engineer as the Horizontal Earth Load input. The minimum and maximum loading is addressed by assigning the appropriate load factors for the various Limit States.

3. WSDOT Buried Structure Standard Plans are detailed to combine vertical exterior face wall reinforcement around the corners and terminate in the top or bottom slab. The *Eriksson Culvert* software details slab and wall bars independently and adds a separate corner bar. The amount of reinforcement and termination of corner bars shall be verified to meet the design.

4. The software is not capable of designing three-sided Structures on a foundation unit (comprising a footing, or a footing with an integral stem wall). The designer may uncouple the analysis by using the software to design the top unit (inverted 'U' shaped frame element), and then transferring the support reaction outputs onto the top of the foundation unit in a separate analysis.

8.3.3.K Buried Structure Standard Plans

The WSDOT Buried Structure Standard Plans identified in Section 8.3.2.B were developed from the *WSDOT – Standard Plans for Split Box/Three-Sided Buried Structures and Wingwalls - Basis of Analysis*, dated September 2023.

Design criteria (parameters, properties, and assumptions), specific to Structure type, are summarized in Section 8.3.3.K.1 and Section 8.3.3.K.2 below.

Common design criteria, regardless of Structure type, are defined as follows:

HYDRAULIC:

a. No part of the Structure (foundations, headwalls, wingwalls, fillets, chamfers, etc.) are placed within the Minimum Hydraulic Opening.

GEOTECHNICAL:

b. Geotechnical design criteria are based on the recommendations given in the *WSDOT Geotechnical Office Memorandum for Buried Structure Standard Plan Development*, dated July 2023.

c. Foundation (or native subgrade) soil, embankment soil, and soil above the foundation level, adjacent to the Structure, are non-liquefiable.

d. Foundation (or native subgrade) soil comprises a Total Unit Weight, γ_{tot}, ranging from 120 to 135 pcf.

e. Bearing Resistance Factors, φ_b, are 1.0 for Service, 0.45 for Strength, and 0.90 for Extreme Limit States.

f. Structural Backfill and/or any existing soil within the Zone of Influence comprises granular soil having a minimum Drained Internal Angle of Friction, ϕ, of 34°, and a Total Unit Weight, γ_{tot}, ranging from 125 to 145 pcf.

g. The Structure was considered 'stiff' (displacing no more than 0.001 times the Structure height), therefore an At-Rest, lateral earth pressure condition was assumed. At-Rest Lateral Earth Pressure Coefficient, K_o, was established as 0.44.

h. Slopes behind headwalls are level or inclined to a maximum 2-Horizontal to 1-Vertical slope, having At-Rest Lateral Earth Pressure Coefficients, K_o, established as 0.44 and 0.64 respectively.

i. Seismic design is based on two Peak Seismic Ground Acceleration Coefficients (modified by short-period site factor), A_s, values of 0.32g and 0.64g.

j. Headwalls are structurally restrained against movement, therefore a Seismic Horizontal Acceleration Coefficient, K_{h0}, equal to the full design A_s is warranted.

k. Seismic At-Rest Lateral Earth Pressure Coefficients, K_{oe}, for headwalls with level backslope are established as 0.68 and 1.05 for A_s values of 0.32g and 064g respectively.

l. Racking analyses were based on an Effective Shear Modulus, G_m, of 2,400 ksf and 1,500 ksf for A_s values of 0.32g and 0.64g respectively, with Poisson's Ratio, ν, established as 0.35. The effective stiffness of structural elements was established as half of the gross values. The controlling Structure-Slip Interface between Full Slip and No Slip was used in the analyses.

m. The Geotechnical Engineer is required to evaluate global slope stability, settlement, and whether the Standard Plan(s) are geotechnically applicable to the site-specific conditions and constraints.

MATERIAL:

n. Concrete Compressive Strength (initial loading for handling), f'_{ci}, was established as 3,500 psi.

o. Concrete Compressive Strength, f'_c, was established as 7,000 psi for precast Structures, and 4,000 psi for headwalls and C.I.P. elements, unless otherwise shown.

p. Reinforcing steel is assumed to have a minimum yield strength, f_y, of 60 ksi.

LOADING:

q. **Live Load and Impact, *LL + IM***, analyses were performed assuming traffic travels parallel to the span, under an HL-93 loading (excluding a Design Lane Load), using a single-loaded lane, a single-lane Multiple Presence Factor of 1.2, and a Vehicular Dynamic Load Allowance, *IM*, in accordance with LRFD-BDS, Article 3.6.2.2.

For Structures less than 2.0 feet from the Roadway surface, Live Load, *LL*, was applied directly to the top surface of the Structure, ignoring the effects of Live Load distribution through the Fill Depth.

The Structures were load rated in accordance with Chapter 13.

r. **Vertical Soil Pressure, *EV***, loading was modified by a soil-structure interaction factor for embankment installation, F_e, of 1.15, assuming compacted fill.

s. **Dead Load, *DL***, computations assume a Unit Weight for Reinforced Concrete, w_{rc} of 0.155 kcf, which includes a 5.0 lbs./ft.^{3} allowance for reinforcement.

t. **Differential Settlement, *SE***, was only considered along the Structure's transverse axis (parallel to the span). This was applied as a pure shear load, to accommodate a structural differential displacement of 1.0-inch per 100-foot of span, reduced from 2.0-inch per 100-foot of span to account for the rotational effects not captured by a pure shear loading condition. The effective moment of inertia, I_{eff}, used in the settlement analyses was established as half of the gross moment of inertia, I_{gross}, to account for a superimposed deformation type of loading and long-term creep effects within the concrete. Differential Settlement along the Structure's longitudinal axis was not specifically analyzed as, perpendicular to the span, the Structure is segmental and therefore considered flexible.

u. **Water Load, *WA***, from hydrostatic pressures was resolved into two components. An internal (stream) pressure loading, $WA_{int.}$, and an external (groundwater) pressure loading, $WA_{ext.}$. The head pressure design height was established as 3'-0" + $H/3$ for Service and Extreme Limit States, and H for Strength Limit State, where 'H' is the Structure's 'Design Height'.

v. **Horizontal Soil Pressure, *EH***, using *Eriksson Culvert*, does not account for a buoyant soil weight when considering external hydrostatic pressure, and this was reconciled externally.

w. **Live Load Surcharge, *LS***, was applied in accordance with LRFD-BDS, Article 3.11.6.4 for Vehicular Loading Perpendicular to Traffic. The equivalent height of soil for vehicular load, h_{eq}, used for variable Fill Depths and Structure heights is as follows:

Table 8.3.3.K-1 Equivalent Height of Soil, h_{eq} for LS (ft.)

Fill Depth (ft.)	Height of Structure (ft.)		
	10	**15**	**20**
0	3.0	2.5	2.0
5	2.5	2.0	
≥ 10	2.0		

For load rating analyses, the equivalent height of soil for vehicular load, h_{eq}, was adjusted proportionally using the ratio of specific vehicle weight (varies) and HL-93 truck weight (72 kips).

x. **Earth Surcharge, *ES***, loading was not considered.

y. **Earthquake, *EQ***, loading was only considered along the Structure's transverse axis (parallel to the span) in accordance with Section 8.3.3.H. Expected material strengths were not used in capacity computations for the Extreme Event loading.

A longitudinal seismic analysis (perpendicular to the span) was not specifically performed as the Structure is segmental and therefore considered flexible.

STRUCTURAL:

z. No part of the Structure (foundations, headwalls, wingwalls, fillets, chamfers, etc.) are placed within the Structure Free Zone.

aa. The critical shear section was taken at a distance, d_v, past the end of the fillet or haunch.

bb. Flexural reinforcement in walls and slabs are based on the moments taken at the intersection of the fillet (or haunch) and uniform depth member, and at midspan.

cc. The Reinforcement Ratio, ρ, was limited to 2.0%, bar sizes to a range between No. 4's and No. 10's, and the spacing from a 4.0-in. min. to 6.0-in. max.

dd. Unit Weight for Concrete, w_c, used in stiffness computations was established as 0.150 kcf.

ee. Resistance Factors were established in accordance with LRFD-BDS. Except for combined flexure and axial effects used for the design of the Structure's walls. See the specific Structure type below for respective Resistance Factors for tension-controlled members. Resistance Factors for compression-controlled members was established as 0.75.

ff. An Effective Length Factor, K, of 1.0 was used for the lateral stability (slenderness) check of the Structure's walls.

gg. Tie plates are required to anchor the last three segments together, or for a minimum of 12.0 feet inboard from each end of the Structure, whichever is greater. Tie plates were designed to resist lateral forces and seismic earth pressure loading on the

Structure's headwall and frictional forces along the Structure's sidewall, which impose an overturning moment on the segment. Each tie plate has a 20-kip capacity for an Extreme Event loading, applying a Resistance Factor of 1.0. Nominal tie plates are required as a constructability aid, facilitating the erection of remaining segments, and to counter any nominal lateral forces.

Additional tie plates are required for Structures located within seismic zones with Peak Seismic Ground Acceleration Coefficients (modified by short-period site factor), A_s, greater than 0.55g, and having headwalls retaining back slopes steeper than 2.5-Horizontal to 1-Vertical.

hh. Approach slab seats were designed to accommodate a uniformly distributed load from an approach slab, designed in accordance with Section 10.6.2, assumed to have an Effective Span Length, S_{eff}, established as 17.0 feet. The uniform loading was established as 8.36 klf, which includes Dead Loads of 1.56 klf for DC, and 0.35 klf for DW, with a Live Load and Impact load established as 6.45 klf. The controlling HL-93 loading was determined as the Tandem (excluding a Design Lane Load) using a single-loaded lane, a single-lane Multiple Presence Factor of 1.2, and a Vehicular Dynamic Load Allowance in accordance with LRFD-BDS, Article 3.6.2.2.

ii. Headwalls are designed to accommodate a fall protection system. Fall protection loading was not applied to an Extreme Event Limit State.

jj. Structural analyses were performed using '*Eriksson Culvert*' engineering software, supplemented with CSiBridge, Mathcad, and spreadsheets, as applicable.

8.3.3.K.1 *Split Boxes Structures*

The general design criteria used for Standard Plan E-20.10 are summarized in Section 8.3.3.K above, specific design criteria for split box Structures are summarized below and, within the Tables and General Notes provided in the Standard Plan.

GEOTECHNICAL:

a. Foundation (or native subgrade) soil shall have a minimum Factored Bearing Resistance as specified in the Standard Plan.

b. Foundations are placed having the top of the bottom slab located a minimum of 2.0 feet below the total scour at the scour design flood.

STRUCTURAL:

c. The following configurations were selected to envelope the established design requirements considering reasonableness in number of iterations, feasibility of the designs, and economics:

- Split Box ~ Slabs with a 10'-0" Design Height
- Split Boxes with 15'-0" and 20'-0" Design Heights

Producing a set of tables from which users select an applicable Structure given the project's site conditions and design requirements.

To accommodate intermittent Design Heights, a 10'-0" unit may be reduced by a maximum of 3'-0", producing a completed unit with a 7'-0" height comprising reinforcement for the 10'-0" unit. This enables the designer to specify a structure with a Design Height ranging from 20' to 17' or with a Design Height ranging from 15' to 12'.

d. Loads, load factors, load combinations, and limit states enveloping all controlling force effects analyzed are summarized on Bridge Standard Drawings 8.3-A1-3 *Split Box General Load Combinations*.

e. Resistance Factors for tension-controlled members used for the design of the Structure's walls was established as 1.0.

f. Shear at the horizontal joint between the top and bottom units was resisted using a combination of the shear capacity of the 'shiplap' geometry, the friction between units, and supplemented using steel brackets as necessary. A factored coefficient of friction for concrete on concrete was assumed to be 0.40.

g. For a Structural model description see Section 8.3.5.E. The boundary condition was established as a uniformly distributed loading along the bottom unit.

8.3.3.K.2 *Three-Sided Structures*

The general design criteria used for Standard Plan E-20.20 are summarized in Section 8.3.3.K above, specific design criteria for three-sided Structures are summarized below and, within the Tables and General Notes provided in the Standard Plan.

GEOTECHNICAL:

a. Foundation (or native subgrade) soil comprises a minimum Drained Internal Angle of Friction, φ, of 30°, 34° and 38°, and a Total Unit Weight of 120 pcf, 125 pcf, and 135 pcf respectively.

b. Foundations are placed with the bottom of the footing 2.0 feet below the total scour at the scour check flood and rely on a minimum of 2.0 feet of passive resistance.

c. Passive Lateral Earth Pressure Resistance Factors, φ_p, are 1.0 for Service, 0.5 for Strength, and 1.0 for Extreme Limit States.

d. Coefficient of Sliding, μ_s, is based on Crushed Surfacing Base Course (CSBC) and assumed to have a Drained Internal Angle of Friction, φ, of 36°, and is established as 0.58 for precast, and 0.73 for C.I.P. footings respectively.

e. Sliding Resistance Factors, φ_s, are 1.0 for Service, 0.8 and 0.9 for C.I.P. and precast foundations respectively for Strength, and 1.0 for Extreme Limit States.

STRUCTURAL:

f. The following configurations were selected to envelope the established design requirements considering reasonableness in number of iterations, feasibility of the designs, and economics:
   - Three-sided Structures with a 10'-0" Design Height (no stem wall)
   - Three-sided Structures with a 12'-0", 14'-0", and 16'-0" Design Height, having stem walls with heights of 2'-0", 4'-0", and 6'-0" respectively.

   Producing a set of tables from which users select an applicable Structure given the project's site conditions and design requirements.

g. Loads, load factors, load combinations, and limit states enveloping all controlling force effects analyzed are summarized on Bridge Standard Drawings 8.3-A1-4, *Three-Sided Structure General Load Combinations*.

h. Resistance Factors for tension-controlled members used for the design of the Structure's walls was established as 0.95.

i. The Structural model can be described as follows:

The top unit was designed using 'Eriksson Culvert' engineering software, supplemented externally for settlement. Live Load reactions from Eriksson outputs were then applied to the foundation unit and subsequently design externally. The boundary condition was established as a trapezoidal distributed loading along the foundation units for stability and fixed for structural design.

8.3.4 Materials

8.3.4.A Concrete

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

Class 5000 through 7000 are commonly used for precast Structures. Self-Consolidating Concrete (SCC) may also be used.

Concrete cover measured from the face of concrete to the face of any reinforcing steel shall be a minimum of 2.0 inch at all faces, except as required in Section 8.3.3.C.

8.3.4.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.

Corrosion resistant reinforcement shall be provided in the top unit in accordance with Section 8.3.3.B.1.

Welded Wire Reinforcement (WWR) may be used to replace plain deformed steel reinforcing bars in Buried Structures provided the WWR has an equivalent bar area, equal or reduced bar spacing, and satisfies crack control and minimum reinforcement requirements of the LRFD-BDS. Welded Wire Reinforcement shall be deformed and shall conform to the requirements of ASTM A1064. The specified minimum yield strength of WWR 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.

8.3.4.C Bedding and Leveling Material

Buried Structure foundation subgrade, bedding and leveling material shall be prepared in accordance with the *Standard Specifications*.

The leveling course shall be a maximum of 2.0 inches and the bedding course shall be a minimum of 6.0 inches. The material requirements for bedding and leveling materials are defined as follows:

8.3.4.C.1 *Precast Reinforced Concrete Buried Structure – Three-Sided*

Bedding course only, consisting of Crushed Surfacing Base Course.

8.3.4.C.2 *Precast Reinforced Concrete Buried Structure – Split Box*

Bedding course, comprising Crushed Surfacing Base Course, or to AASHTO Grading No. 57 as specified in *Standard Specifications*, Section 9-03.1(4)C, followed by a leveling course in accordance with *Standard Specifications*, Section 9-03.1(2)B Grading Class 1 or Class 2 Sand.

8.3.4.C.3 *Precast Reinforced Concrete Retaining Walls (Wingwalls)*

Bedding course only, consisting of Crushed Surfacing Base Course.

8.3.4.D Joint Sealant and External Sealing Bands

All non-grouted joints between concrete units shall be sealed by Butyl Rubber Sealant in accordance with *Standard Specifications*, Section 9-04.11.

All joints between concrete units shall be wrapped with an External Sealing Band in accordance with *Standard Specifications*, Section 9-04.12, except that bottom unit longitudinal joints are not required to be wrapped. Additionally, the External Sealing Band may be omitted for top unit longitudinal joints when a waterproofing membrane, a concrete overlay, or 5-inch minimum concrete topping slab is installed. External Sealing Bands shall be installed before any tie plates or external shear restraining system(s).

See Bridge Standard Drawing 8.3-A2-7 and 8.3-A3-5 for typical joint sealing details for split box and three-sided Structures respectively.

8.3.4.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:

8.3.4.E.1 *Metal Structural Plate Structures*

Minimum corrosion rates and design Service Life analysis shall be in accordance with Section 6.7.2.

When using galvanized or zinc coated metal structural plate Structures below the Hydraulic Design Flood Elevation, a reinforced concrete splash wall is required. See Section 8.3.6.C for additional details and requirements. The Hydraulic Design Flood Elevation will be established in the Hydraulics Report and is based on either the 100-year or 2080 projected 100-year flood elevation.

8.3.4.E.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.

Alternative corrosion protection measures to achieve a minimum Service Life of 75-years shall be approved by the State Bridge Design Engineer.

8.3.5 Limit States and Design Methodologies

8.3.5.A Service Limit State

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

8.3.5.A.1 *Total and Differential Settlement*

The geotechnical engineer shall perform settlement calculations, evaluating total expected settlement, along with the potential for transverse and longitudinal differential settlement between Buried Structure units, including wingwalls, and provide settlement data and recommendations to the structural designer. The structural engineer shall evaluate, design, and detail all elements for any settlement(s) warranted by the geotechnical engineer.

8.3.5.A.2 *Deflection and Shear Transfer*

Concrete Structures directly exposed to vehicular traffic (Fill Depth < 2.0 feet), having top slabs 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 for all concrete Buried Structures, regardless of Fill Depth.

Where the top surface of the Structure is less than 2.0 feet from the Roadway surface, a method of shear transfer shall be provided between the top slabs of adjacent precast units to equalize deflections by incorporating one of the following:

1. A structural connection between adjacent precast units capable of transferring the imposed shear and equalizing deflections. The structural connections shall include reinforced cast-in-place concrete, or Ultra High-Performance Concrete (UHPC) closures, or grouted shear keys.
2. A 5-inch minimum thickness composite cast-in-place reinforced concrete topping slab, meeting the requirements of a Type 4 Protection System.
3. A full Roadway section (12.0-inch minimum depth), comprising a combination of CSBC and/or CSTC, and HMA.

In addition to meeting the requirements of LRFD-BDS, Table 2.5.2.6.3-1, the top slab thickness for boxes, split boxes and three-sided Structures shall meet a span-to-thickness ratio of 18 or less.

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 Depth. Thus, the requirements for thickness, differential deflection, and shear transfer between adjacent units does not apply.

8.3.5.A.3 *Control of Cracking*

Reinforcement shall be provided and spaced to meet the requirements of LRFD-BDS, Article 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, Article 5.10.6.

8.3.5.B Strength Limit State

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

8.3.5.C Extreme Limit State

Extreme Limit State is used to satisfy seismic, check flood, and scour requirements as applicable. Expected material strengths shall not be used for computing capacities for Extreme Event loading.

8.3.5.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 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 for ease of computations, the bottom slab of these Structures is analyzed as a simply supported beam. Again, this does not capture the Structure's behavior efficiently and tends to leave reinforcement deficiencies in negative moment regions. Analyzing the slab supported on a uniform or trapezoidal support reaction or on an elastic foundation are more appropriate approaches. An elastic foundation 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.

Through the development of the Buried Structure Split Box Standard Plan, applying a uniform or trapezoidal support reaction, when compared with an elastic foundation analysis, was found to sufficiently capture the soil-structure interaction behavior for the design of the bottom slab of box structures.

8.3.5.E Structural Modeling

8.3.5.E.1 *Three-Sided Structures*

Should be modeled as a rigid frame, chorded arch, or arch with pin or fixed support reactions as applicable. The top unit (inverted 'U' or chorded 'U' shaped frame element) of a three-sided Structure may be modeled in a software such as *Eriksson Culvert* or *CSiBridge* and then the support reaction outputs at the bottom of the wall(s) can be applied to the top of foundation units (comprising a footing, or a footing with an integral stem wall) when utilized. This essentially uncouples the analysis so the two elements can be analyzed and designed separately. In addition to the reactions from the top unit, foundation units should be designed for external stability and the appropriate earth and hydraulic lateral forces.

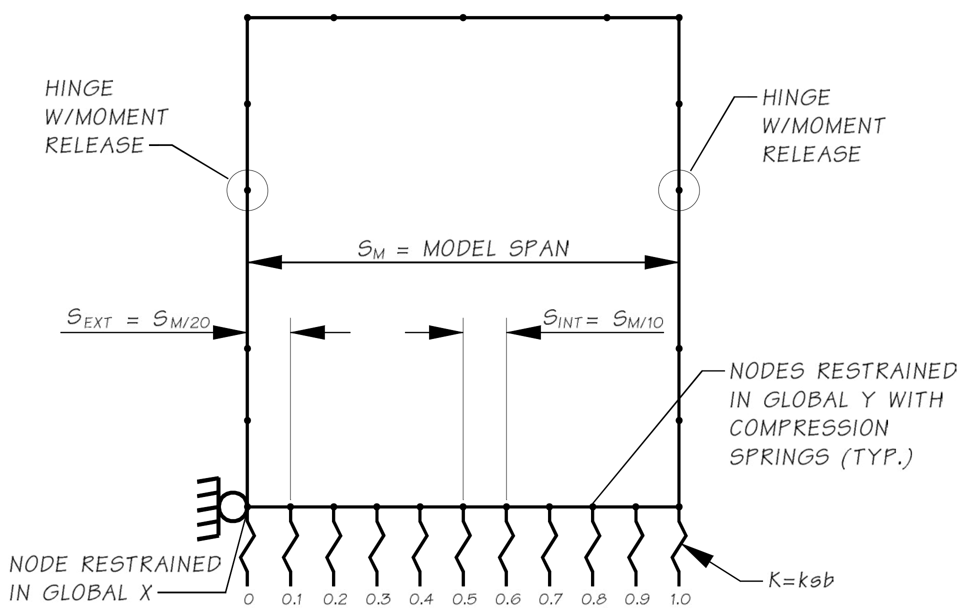
8.3.5.E.2 *Split Box Structures*

When utilizing an elastic foundation method for analyzing split boxes, the model should be developed 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.

When utilizing a uniform or trapezoidal support condition, the springs should be replaced with upward vertical distributed loads equal and opposite to the downward vertical loads.

The hinges shown in the model corresponds to the joints between the top and bottom units. When using a shiplap joint, the joint is only capable of transferring shear in one direction and no moment, therefore, the shear output from both joints should be added together and applied to a single joint for design. As an alternative way to resist shear in both directions, steel brackets (like the WT restrainers detailed on Standard Plan E-20.10) may be bolted across the joint to provide shear restraint in the unrestrained direction. Additionally, due to Dead Loads from the top unit and the fill above the Structure, friction forces are developed between the top and bottom units at the joints. A maximum Friction Factor, μ, of 0.4 (concrete to concrete) shall be used when accounting for friction forces to aid in shear resistance at the joints.

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

8.3.5.E.3 *Split Box ~ Slab Structures*

Can be modeled as described in Section 8.3.5.E.2, and 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 top unit should be replaced with a flat slab unit.

Alternatively, the top slab unit (superstructure) may be analyzed using *PGSuper* with the bearing reactions applied to the bottom unit as an external axial load on a per foot basis. Additionally, in lieu of a racking analysis, earthquake loads, and bearing shear forces shall be applied to the bottom unit as described in Section 7.5.4.D, and 7.5.4.E respectively.

8.3.6 *Provisions for Structure Type*

8.3.6.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 top unit, and a rigid three-sided frame base or bottom unit.

Concrete box Structures shall be constructed in accordance with *Standard Specifications*, Section 6-20.

8.3.6.A.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 = Design Span Length, shall be varied in 1'-0" increments from a minimum of 8'-0" to a maximum of 35'-0".

TW_F = Thickness of the Wall at the Fillet, is typically either 10.0" or 12.0", and tapers to TW_J = Thickness of the Wall at the Joint, the typical taper is 2H:103V.

H_T = Height of the Top Unit's Wall, and H_B = Height of the Bottom Unit's Wall, shall be varied in 1'-0" increments from a minimum of 0'-0" to a maximum of 10'-0" (includes the fillet height). 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".

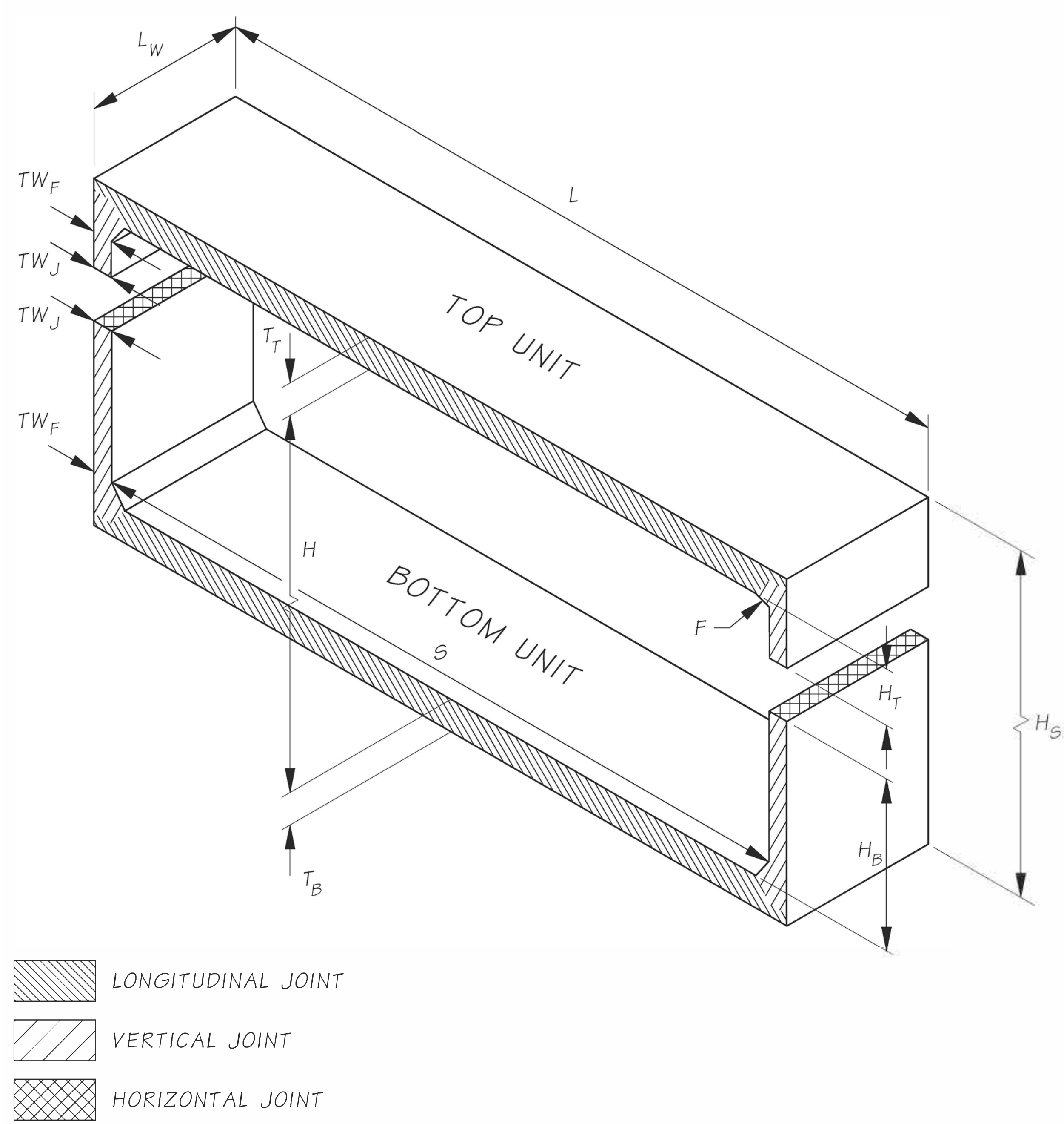
F = Fillet dimensions are fixed and shall be 17.0" x 17.0".

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 Frame Elements



L	=	Length of Structure	H_B	=	Height of Bot. Unit Walls
H_s	=	Height of Structure	T_T	=	Thickness of Top Slab
L_W	=	Lay Width or ‘Lay Length’	T_B	=	Thickness of Bot. Slab
S	=	Design Span Length	TW_F	=	Wall Thickness at Fillet
H	=	Design Height of Segment	TW_J	=	Wall Thickness at Joint
H_T	=	Height of Top Unit Walls	F	=	Fillet (Height & Width)

8.3.6.A.2 *Distribution of Live Load through Earth Fill*

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

Where the Fill Depth is:

1. Less than 2.0 feet, the distribution of axle load to the top slab specified in LRFD-BDS, Article 4.6.2.10.2 shall not apply. Axle load shall be distributed directly to the top slab for determining moment, thrust, and shear, ignoring the effects of Live Load distribution through the Fill Depth.
2. 2.0 feet or greater, Live Load shall be distributed to the top slab in accordance with LRFD-BDS, Article 3.6.1.2.6 as wheel loads and shall consider interaction effects.

8.3.6.A.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.
- **Horizontal Joints** (See Figure 8.3.6.A-1):

  Joints between the top units and bottom units.
- **Longitudinal Joints** (See Figure 8.3.6.A-1):

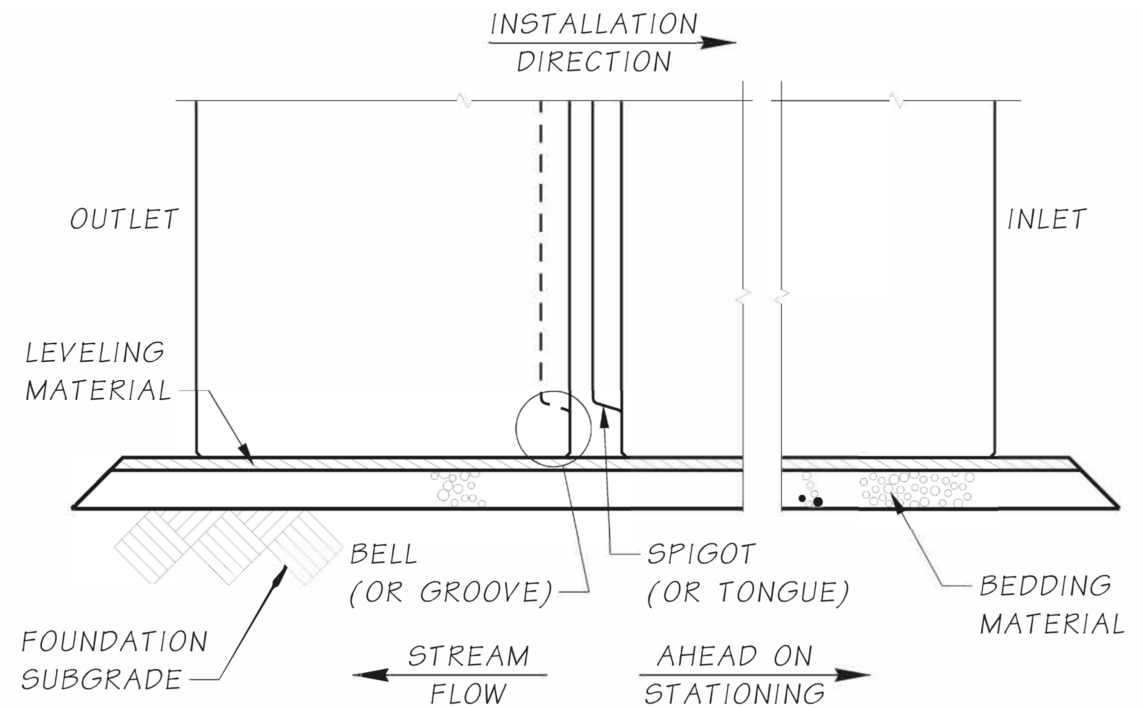
  Top and bottom joints between adjacent units parallel to the Design Span.

  Top slab longitudinal joints shall address shear transfer as specified in Section 8.3.5.A.2.

WSDOT acceptable joint types can be found on Standard Plan E-20.10. 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. See Bridge Standard Drawing 8.3-A2-7 for joint sealing details.

Joints employing a bell & spigot (a.k.a. a tongue & groove, or shiplap), as shown in Figure 8.3.6.A-3, shall be detailed such that the laying of units on the prepared bedding and leveling 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 in this aligned configuration should be such that the bottom units 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.

8.3.6.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 (comprising a footing, or a footing with an integral stem wall).

Three-Sided Structures shall be constructed in accordance with *Standard Specifications*, Section 6-20.

8.3.6.B.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-A1-1. If project limitations require alternate dimensions, the designer is encouraged to contact precast manufacturers for available options.

8.3.6.B.2 *Distribution of Live Load through Earth Fill*

The distribution of wheel loads shall be as specified in Section 8.3.6.A.2.

For Structures with skews greater than 15°, the effects of the skew shall be considered in the analysis in accordance with LRFD-BDS, Article 12.14.5.3.

8.3.6.B.3 *Joint Design and Details*

Refer to Section 8.3.6.A.3 for joint requirements, except as noted:

- **Horizontal Joints** (See Figure 8.3.6.A-1):

  Joints between the top units and foundation units.

  Foundation units consist of either a footing, or a footing with an integral stem wall. See Bridge Standard Drawing 8.3-A3-4 for connection detail options. See Bridge Standard Drawing 8.3-A3-5 for joint sealing details.

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

Design and construction of metal structural plate Structures shall conform to the LRFD-BDS, Article 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.

When required, reinforced concrete splash walls shall extend from the top of footing or cap to Hydraulic Design Flood Elevation. The splash wall shall meet the following requirements:

1. The wall shall have a minimum thickness of 6.0 inches.
2. The wall shall be constructed from structural concrete with a minimum compressive strength of 4000 psi.
3. The wall may be constructed as a cantilever wall integral with the footing or a wall positively connected to the galvanized metal Structure. A positive connection shall consist of headed bolts, shear studs, or other means to physically connect the wall to the Structure. If connected to the footing, the designer should consider short dowels extending from the footing lap spliced to the remaining wall reinforcement. This allows the metal Structure to be installed and bolted without conflicting with wall reinforcement.

4. The wall shall be reinforced for temperature, shrinkage, and crack control as a minimum per the LRFD-BDS.
5. The exposed wall surface shall be finished with a Class 2 surface finish.
6. The wall shall not be used for attaching appurtenances such as root wads, cables etc. unless specifically designed for.
7. The designer should consult with metal plate supplier on the sequence of construction of the splash wall in relation to backfilling and indicate in the Plans any required sequences. If none are required, the sequence shall be left up to the Contractor.

Minimum backfill cover, over the top and on the sides of the 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 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 the *Geotechnical Design Manual*. In addition to earth pressures, water tables, seismic design, and uplift, special consideration should be given to ensure differential settlement either does not occur or is included in the calculations for forces, crack control and water stops.

Buoyant forces from high ground water conditions should be investigated for permanent as well as construction load cases so the vault does not float. Controlling loading conditions may include backfilling an empty vault, filling the vault with stormwater before it is backfilled, or seasonal maintenance that requires draining the vault when there is a high-water table. In all Limit States, the buoyancy force (*WA*) load factor shall be taken as γ_{WA} = 1.25 in LRFD-BDS, Table 3.4.1-1. In the Strength Limit State, the load factors that resist buoyancy (γ_{DC}, γ_{DW}, γ_{ES}, Etc.) shall be their minimum values, in accordance with 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, Article 3.4.2.

In certain situations, tie-downs may be required to resist buoyancy forces. The resisting force (R_n) and resistance factors (ø) for tie-downs shall be provided by the Geotechnical Engineers. The buoyancy check shall be as follows:

For Buoyancy without Tie-Downs:

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

For Buoyancy with Tie-Downs:

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

Where:

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

$R_{UPLIFT} = |\, \gamma_{WA}\, WA \,|$

ACI 350-06 has stricter criteria for cover and spacing of joints than the LRFD-BDS. Cover is not to be less than 2.0 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, Article 5.6.7 with γ_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 ties spaced at 4′-0″ on center. Any modifications to these described joints shall be justified with calculations. 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′-0" in height, the minimum wall thickness is 12.0 inches. This minimum thickness is generally good practice for all external walls, regardless of height, to allow for 2.0 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 1.0- 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 1,200 gallons of water to facilitate maintenance and cleanout operations. Baffles shall be watertight. 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 36.0-inches 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 State 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 1,500-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 2.0-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

Structural Earth Walls

8.1-A2-1 SEW Wall Elevation (PDF 50KB) (DWG 36KB)
8.1-A2-2 SEW Wall Section (PDF 90KB) (DWG 51KB)

Tie Back Walls

8.1-A3-1 Soldier Pile/Tieback Wall Elevation (PDF 61KB) (DWG 44KB)
8.1-A3-2 Soldier Pile/Tieback Walls Details A (PDF 194KB) (DWG 200KB)
8.1-A3-3 Soldier Pile/Tieback Walls Details B (PDF 200KB) (DWG 210KB)
8.1-A3-4 Soldier Pile/Tieback Walls Details (PDF 144KB) (DWG 146KB)
8.1-A3-5 Soldier Pile/Tieback Walls Fascia Panel Details (PDF 127KB) (DWG 136KB)
8.1-A3-6 Soldier Pile/Tieback Wall Perm Ground Anchor Details (PDF 153KB) (DWG 128KB)

Soil Nail Wall

8.1-A4-1 Soil Nail Wall Elevation (PDF 75KB) (DWG 81KB)
8.1-A4-2 Soil Nail Wall Typical Section (PDF 103KB) (DWG 115KB)
8.1-A4-3 Soil Nail Wall Fascia Epoxy Coated (PDF 85KB) (DWG 60KB)
8.1-A4-4 Soil Nail Wall Fascia Encapsulated (PDF 111KB) (DWG 96KB)

Noise Barrier

8.1-A5-1 Noise Barrier on Bridge (PDF 48KB) (DWG 39KB)

Cable Fence

8.1-A6-3 Cable Fence Details 1 of 3 (PDF 74KB) (DWG 68KB)
8.1-A6-4 Cable Fence Details 2 of 3 (PDF 74KB) (DWG 68KB)
8.1-A6-5 Cable Fence Details 3 of 3 (PDF 166KB) (DWG 160KB)

Buried Structures (Precast)

8.3-A1-1 Geometrics Table (PDF 74KB) (DWG 68KB)
8.3-A1-2 Split Box & Three-Sided Structure General Notes (PDF 94KB)(DWG 228KB)
8.3-A1-3 Split Box General Load Combinations (PDF 80KB) (DWG 78KB)
8.3-A1-4 Three-Sided Structure General Load Combinations (PDF 80KB) (DWG 78KB)

Split Boxes Structures (Precast)

8.3-A2-1 Split Box SB20 – SB25 Series (PDF 74KB) (DWG 68KB)
8.3-A2-2 Split Box Segment Layout Plan and Elevation (PDF 82KB) (DWG 78KB)
8.3-A2-3 Split Box Typical Section (PDF 803KB) (DWG 91KB)
8.3-A2-4 Split Box ~ Slab Typical Section (PDF 107KB) (DWG 235KB)
8.3-A2-5 Split Box Unit Geometric Details (PDF 68KB) (DWG 75KB)
8.3-A2-6 Split Box Unit Reinforcement Details (PDF 68KB) (DWG 76KB)
8.3-A2-7 Split Box Joint Sealing Details (PDF 88KB) (DWG 67KB)
8.3-A2-8 Split Box Tie Plate Connection Details (PDF 74KB) (DWG 68KB)

Three-Sided Structures (Precast)

8.3-A3-1	Three-Sided FC20 – FC40 Series (PDF 74KB) (DWG 68KB)
8.3-A3-2	Three-Sided VC45 – VC50 Series (PDF 74KB) (DWG 68KB)
8.3-A3-3	Three-Sided VC55 – VC60 Series (PDF 74KB) (DWG 68KB)
8.3-A3-4	Three-Sided Foundation Connection Joint Details (PDF 78KB) (DWG 81KB)
8.3-A3-5	Three-Sided Longitudinal Joint Details (PDF 68KB) (DWG 88KB)

Buried Structure Design Examples

8.3-A4-1	Split Box Buried Structure (PDF 254KB)
8.3-A4-2	Three-Sided Buried Structure (PDF 275KB)
8.3-A4-3	Soil-Structure Interaction (Racking) Analysis (PDF 275KB)

8.5 Appendices

Appendix 8.1-A1 Summary of Design Specification Requirements for Walls

Wall Types		Design Specifications
Preapproved Proprietary Structural Earth Walls	General	Design shall be based on current editions, including current interims, of the following documents: AASHTO LRFD Bridge Design Specifications, WSDOT *Geotechnical Design Manual* (GDM), and WSDOT BDM.
Preapproved Proprietary Structural Earth Walls	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 BDM Chapter 4.
Preapproved Proprietary Structural Earth Walls	Traffic Barrier	Moment slab barrier shall be designed in accordance with the WSDOT BDM and the AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load, unless otherwise specified in the Contract Plans or Contract Special Provisions.
Non-Preapproved Proprietary Structural Earth Walls	General	Design shall be based on current editions, including current interims, of the following documents: AASHTO LRFD Bridge Design Specifications, WSDOT GDM, and WSDOT BDM.
Non-Preapproved Proprietary Structural Earth Walls	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 BDM Chapter 4.
Non-Preapproved Proprietary Structural Earth Walls	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 Geosynthetic Walls are designed in accordance with the design criteria listed in the GDM.
Standard Plan Geosynthetic Walls	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).
Standard Plan Geosynthetic Walls	Traffic Barrier	Traffic 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.
Non-Standard Geosynthetic 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.
Non-Standard Geosynthetic Walls	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 BDM Chapter 4.
Non-Standard Geosynthetic Walls	Traffic Barrier	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.

Wall Types		Design Specifications
Standard Plan Reinforced Concrete Walls	General	Standard Plan D-10.10 thru D-10.45 walls are designed in accordance with AASHTO LRFD Bridge Design Specifications 4th Edition 2007 and interims through 2008 and the WSDOT GDM Nov. 2008. Standard Plan D-20.10 walls are designed in accordance with AASHTO LRFD Bridge Design Specifications 9th Edition 2020, WSDOT BDM June 2022, and the WSDOT GDM February 2022.
	Seismic	AASHTO Guide Specifications for LRFD Seismic Bridge Design using 2002 U.S. Geological Survey (USGS) National Seismic Hazard Model (NSHM).
	Traffic Barrier	Standard Plan D-10.10 thru D-10.45 walls are designed for TL-4 impact loading distributed over 48 ft at the base of wall.
Non-Standard Reinforced Concrete Walls	General	Non-standard reinforced concrete 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.
	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 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. F_t is distributed over L_t at the top of barrier. Load from top of barrier is distributed at a 45-degree angle into the wall.
Soldier Pile Walls With & Without Tiebacks	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 BDM Chapter 4.
	Traffic Barrier	AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load. Ft is distributed over Lt at the top of barrier. Load from top of barrier is distributed downward into the wall spreading at a 45-degree angle.
Standard Plan Noise Barrier Walls	General	Standard Plans D-2.36 and D-2.46 are designed in accordance with AASHTO LRFD Bridge Design Specifications 6th Edition 2012 and interims through 2013.
	Seismic	Standard Plans D-2.36 and D-2.46 are designed in accordance with AASHTO LRFD Bridge Design Specifications 1000-year map design acceleration.
	Traffic Barrier	Standard Plan D-2.46 may accommodate a MASH compliant concrete barrier having a railing test level of TL-4 or less. The concrete barrier shall be located as shown in the Standard Plan.

Wall Types		Design Specifications
Non-Standard Noise Barrier Walls	General	Design shall be based on current editions, including current interims, of the following documents: AASHTO LRFD Bridge Design Specifications, WSDOT GDM, and WSDOT BDM.
	Seismic	AASHTO 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 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.
Soil Nail 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. 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 ensure 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. Design of the cantilevered portions of the soil nail wall facing which typically occurs at the top, bottom, or ends of the wall shall be in accordance with current editions, including current interims, of the following documents: AASHTO LRFD Bridge Design Specifications (See Article 11.6 for conventional retaining walls), 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 BDM Chapter 4.
	Traffic Barrier	Moment slab barrier shall be designed in accordance with the WSDOT *Bridge Design Manual* and the AASHTO LRFD Bridge Design Specifications section A13.3 for Concrete Railings considering a minimum TL-4 impact load.
Non-Standard Non-Proprietary Walls Gravity Blocks, Gabion Walls	General	Design shall be based on current editions, including current interims, of the following documents: AASHTO LRFD Bridge Design Specifications, WSDOT GDM, and WSDOT BDM.
	Seismic	AASHTO 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 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.

8.99 References

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