Design Memorandum

TO: All Design Section Staff
FROM: Bijan Khaleghi
DATE: March 5, 2020
SUBJECT: BDM Section 8.3 - Buried Structures

This design memorandum specifies WSDOT's policy for Buried Structures. This memorandum replaces the design memorandum issued on December 25, 2019.

Bridge Design Manual Revisions:

8.3   Buried Structures

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

8.3.1   *General Policy*

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

Span length for Buried Structures shall be the widest opening from interior face to interior face as measured parallel to roadway centerline. When not supporting a roadway, the span length shall be the widest opening from interior face to interior face measured perpendicular to the Buried Structure centerline.

Consideration must be given to the degradation of Buried Structure materials resulting from corrosive and abrasive conditions. For culverts, 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 commonly used for metal structures, and additional

concrete cover or protective coatings over reinforcing steel are common approaches used in concrete structures to ensure the service life criteria is met.

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

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

8.3.2 WSDOT Buried Structure Templates

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

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

- Precast Reinforced Concrete Split Box Culvert (PRCSBC)

    o SB20 through SB25, see Bridge Standard Drawing 8.3.2-A3

    o SBS20 through SBS25, see Bridge Standard Drawing 8.3.2-A13

- Precast Reinforced Concrete Three-Sided Structures (PRCTSS)

    o FC20 through FC40, see Bridge Standard Drawing 8.3.2-A3

    o VC45 through VC50, see Bridge Standard Drawing 8.3.2-A4

    o VC55 through VC60, see Bridge Standard Drawing 8.3.2-A5

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

8.3.3 General Design Requirements

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

A. Design Delivery Methods

1. Span lengths less than 30.0 feet

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

2. Span lengths equal to or greater than 30.0 feet

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

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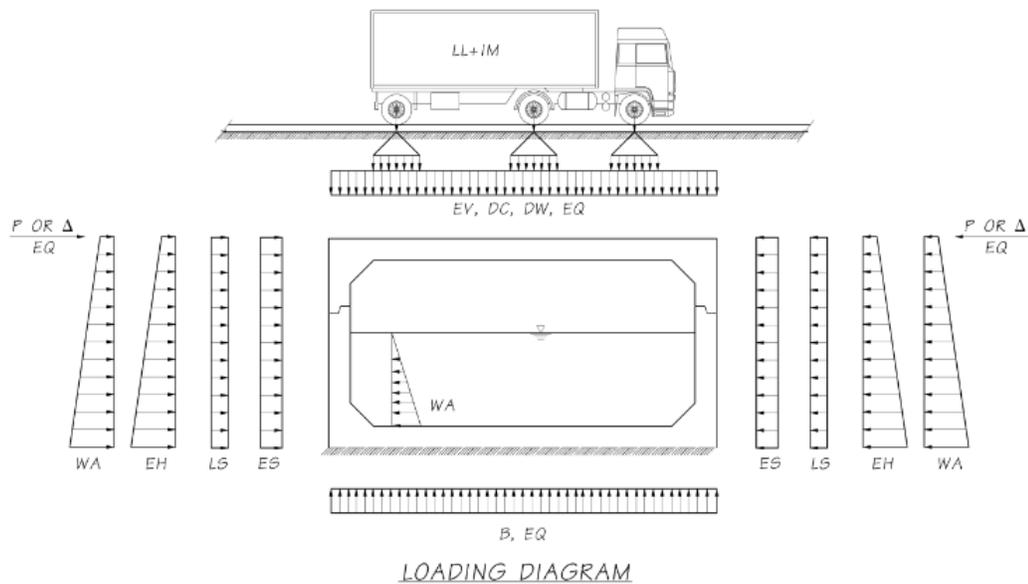
When a contractor supplied design is not selected:

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

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

c. The design may be completed by one of the following;

    • WSDOT or Consultant engineering staff.

    • Proprietary supplier identified as a sole source by WSDOT.

B. Application of Loads

Buried Structures shall be designed for force effects in accordance with *LRFD Specifications*, Section 12.6.1 (as shown in Figure 8.3.3.B), except that exemption from seismic loading shall not apply for Buried Structures with spans of 20.0 feet or greater.

Figure 8.3.3.B   Typical Split Box Loading

LL+IM

EV, DC, DW, EQ

P OR Δ / EQ

P OR Δ / EQ

WA   EH   LS   ES

ES   LS   EH   WA

WA

B, EQ

LOADING DIAGRAM

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

- Maximum Vertical, Maximum Horizontal

- Maximum Vertical, Minimum Horizontal

- Minimum Vertical, Maximum Horizontal

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

The decrease in live load effect due to increase in fill depth shall be considered in both design and load rating of Buried Structures.

C. Buried Structure Foundation Design

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

D. Buried Structure Wingwall, Headwall, and Cutoff Wall Design

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

The term 'headwall' is an integral retaining wall at the inlet and outlet of Buried Structures as a means to retain the roadway fill directly above the structure.

The term 'cutoff wall' is a physical barrier employed to seal off water flow into the structural backfill below hydraulic structures, and protects structures placed over erodible deposits.

Wingwalls, headwalls, and cutoff walls for Buried Structures shall be designed in accordance with the current versions of the *Geotechnical Design Manual*, and Chapter 11 of the *LRFD Specifications*. The bottom of wingwall foundations, headwalls and cutoff walls shall be located a minimum of 2.0 feet below the scour elevation in accordance with the *Geotechnical Design Manual*, Section 15.4.5, unless a greater depth is otherwise specified. The structure shall be designed for the effects of scour as described in Section 8.1.9.

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

Headwalls, wingwalls, and railings shall be designed for vehicular collision and pedestrian or worker fall protection forces where applicable in accordance with Section 10.2 and Section 10.5.

E.   Fall Protection

Fall protection shall be provided on headwalls and wingwalls in accordance with Section 8.1.10.B for exposed wall heights equal to or greater than 4.0 feet.

F.   Buried Structure Seismic Design

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

Seismic design need not be considered for Buried Structures with span lengths of less than 20.0 feet.

Buried Structures greater than or equal to 20.0 feet shall be designed for seismic effects in accordance with Section 13.5 *Seismic Considerations* in the *AASHTO, Technical Manual for Design and Construction of Road Tunnels – Civil Elements* (hereafter referred to as the *Technical Manual*), with current interims.

1.   Seismic Loading Effects

Buried Structures with span lengths equal to or greater than 20.0 feet shall be designed in accordance with *AASHTO LRFD Road Tunnel Design and Construction Guide Specifications*, 1st Edition, 2017, Sections 10.8.3 and 10.8.4 respectively to accommodate the effects resulting from two types of seismic loading:

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

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

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

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

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

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

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

2. Attenuation of Peak Ground Motion Parameters

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

Table 8.3.3.F-2   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
≤ 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 ratio can be estimated by linear interpolation – larger ratio corresponding to shallower depth.

G. Usage of Buried Structure Design Software and/or Spreadsheets

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

8.3.4 Materials

A. Concrete

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

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

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

B. Steel

Reinforcing steel shall be in accordance with Section 5.1.2.

Steel reinforcing bars shall be deformed, and shall conform to either ASTM A706 or AASHTO M 31 (ASTM A615). The nominal yield strength for reinforcement bar shall be limited to a maximum of 80 ksi.

Welded wire reinforcement may be used to replace steel reinforcing bars in Buried Structures. Welded wire reinforcement shall be deformed and shall conform to the requirements of AASHTO M 336 (ASTM A1064). The specified minimum yield

strength of welded wire reinforcement shall be limited to a maximum of 75 ksi per Section 5.1.2.I.

Prestressing steel shall be in accordance with Section 5.1.3.

C. Bedding Material

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

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

1. Precast Reinforced Concrete Three-Sided Structures (PRCTSS)

    Crushed Surfacing Base Course.

2. Precast Reinforced Concrete Split Box Culverts (PRCSBC)

    Standard Specifications, Section 9-03.12(3), or to AASHTO Grading No. 57 as specified in *Standard Specifications*, Section 9-03.1(4)C.

3. Precast Wingwalls

    Crushed Surfacing Base Course.

D. Joint Sealant and External Sealing Bands

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

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

See Section 8.4 Bridge Standard Drawings for joint sealing details.

E. Corrosion

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

1. Metal Structural Plate Structures

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

2. Concrete Structures

    Additional concrete cover or protective coatings over reinforcing steel may be utilized.

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

8.3.5 *Limit States and Design Methodologies*

A. Service Limit State

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

1. Total and Differential Settlement

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

2. Deflection

   Concrete structures with less than 2.0 feet of fill and top slabs that are thinner than specified in *LRFD Specifications*, 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 Specifications*, Table 2.5.2.6.3-1 are required unless computation of deflection indicates that lesser depths may be used without adversely affecting the strength or serviceability of the structure. The vehicular deflection limits for concrete structures in accordance with *LRFD Specifications*, Section 2.5.2.6.2 may be used to meet these requirements.

   For concrete structures where the top slab is less than two feet from the roadway surface, the design shall also provide a method of shear transfer between adjacent units to equalize deflections by incorporating at least one of the following:

   - Provide a joint between adjacent units capable of transferring the imposed shear.

   - Provide a 5.0 inch minimum composite cast-in-place concrete topping, meeting the requirements of a Type 4 Protection System in accordance with Section 5.7.4.

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

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

3. Control of Cracking

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

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

B. Strength Limit State

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

C. Extreme Limit State

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

D. Boundary Conditions

Rigid frames are statically indeterminate structures, and as such, require more rigorous analysis than statically determinate structures. This has led to some simplifying assumptions to facilitate rapid computations, which do not capture the inherent advantages of rigid frames, such as continuity, stiffness, and economy, creating inefficiencies in the design.

Rigid frame members designed assuming simply supported boundary conditions, produce conservative reinforcement requirements in positive moment regions, and leave negative moment regions with deficiencies in reinforcement requirements. This is not an efficient design approach, because the assumed boundary conditions do not capture the essence of the structure's behavior. Therefore, to accurately capture a rigid frame's behavior and eliminate possible reinforcement deficiencies, concrete three-sided structures and box structures shall be analyzed as a rigid frame, applying appropriate boundary conditions.

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

compression springs along the bottom slab of the structure. To effectively capture this soil-structure interaction behavior, the bottom slab of box structures shall be analyzed on an elastic foundation.

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

E. Structural Modeling

1. Precast Reinforced Concrete Three-Sided Structures (PRCTSS)

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

2. Precast Reinforced Concrete Split Box Culverts (PRCSBC)

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

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

The hinges shown in the model corresponds to the joints between the upper and lower segments. Due to the behavior of a shiplap joint, capable of transferring shear in only one direction but not moment, the shear output from both joints should be added together and applied to a single joint.

Figure 8.3.5.E-2 Split Box on an Elastic Foundation Model
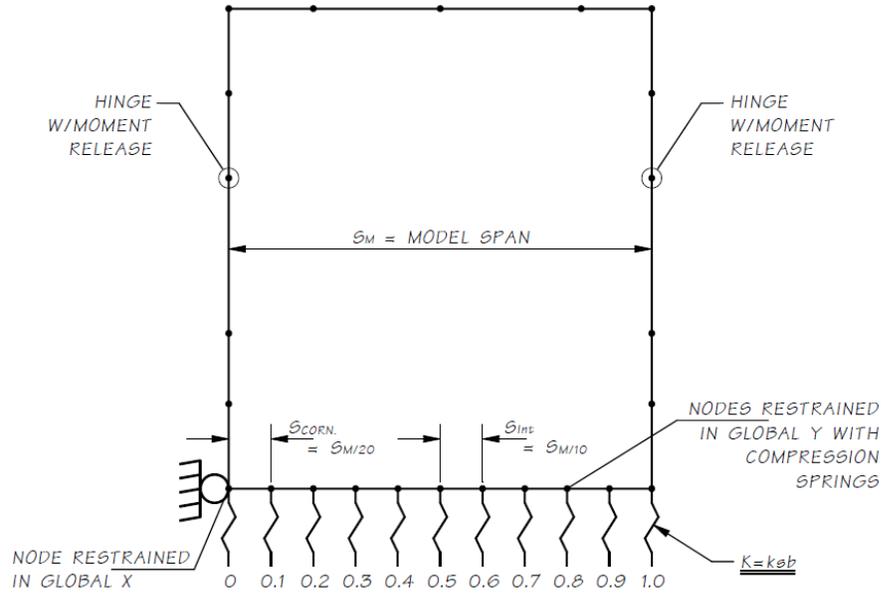
Nodes should be placed at points of interest such as corners, fillet/uniform cross-section interface locations, and midspan of members. Additional nodes should be placed along the bottom element of the model, and restrained using compression springs in the global y direction.

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

$$K = k \, s \, b$$

Where:

K = Compression Spring Constant

k = Modulus of Subgrade Reaction

s = Tributary Length Associated with a Node

b = Unit Slab Width

3. Precast Reinforced Concrete Split Box Culverts (PRCSBC) with Top Slab

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

Alternatively, the upper flat slab unit (superstructure) may be analyzed using PGSuper with the bearing reactions applied to the lower unit as an external axial load on a per foot basis. Additionally, in lieu of a racking analysis,

earthquake loads, and bearing shear forces could be applied to the lower unit as described in Section 7.5.4.D, and 7.5.4.E respectively.

8.3.6   *Provisions for Structure Type*

A. Reinforced Concrete Box and Split Boxes

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

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

1. Precast Geometric Limitations

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

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

W1 = Wall Thickness at the Fillet, is typically either 10.0" or 12.0", and tapers to W2 = Wall Thickness at the Joint of upper unit, or W3 = Wall Thickness at the Joint of lower unit respectively, typical taper is 2H:103V.

H1 = Height of Tapered Wall of upper unit, and H2 = Height of Tapered Wall of lower unit, shall be varied in 1'-0" increments from a minimum of 0'-0" to a maximum of 10'-0". If project limitations require a dimension between 1'-0" increments, the designer is encouraged to contact precast manufacturers for available options.

TT = Thickness of Top Slab, and TB = Thickness of Bottom Slab, shall be varied in 2.0" increments from a minimum of 10.0" to a maximum of 24.0".

F = Fillet, shall be 17.0" x 17.0".

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

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

If the lay width of the individual precast segments is not critical on a project, the designer is encouraged to provide a minimum lay width of the segments, along with an overall lay width of structure to allow fabricators to choose lay

widths of individual segments based on their form capabilities and shipping requirements (typical shipping weight limits are 60-65 kips).

Figure 8.3.6.A-1 Typical Split Box, Comprising Two Precast Three-Sided Frames
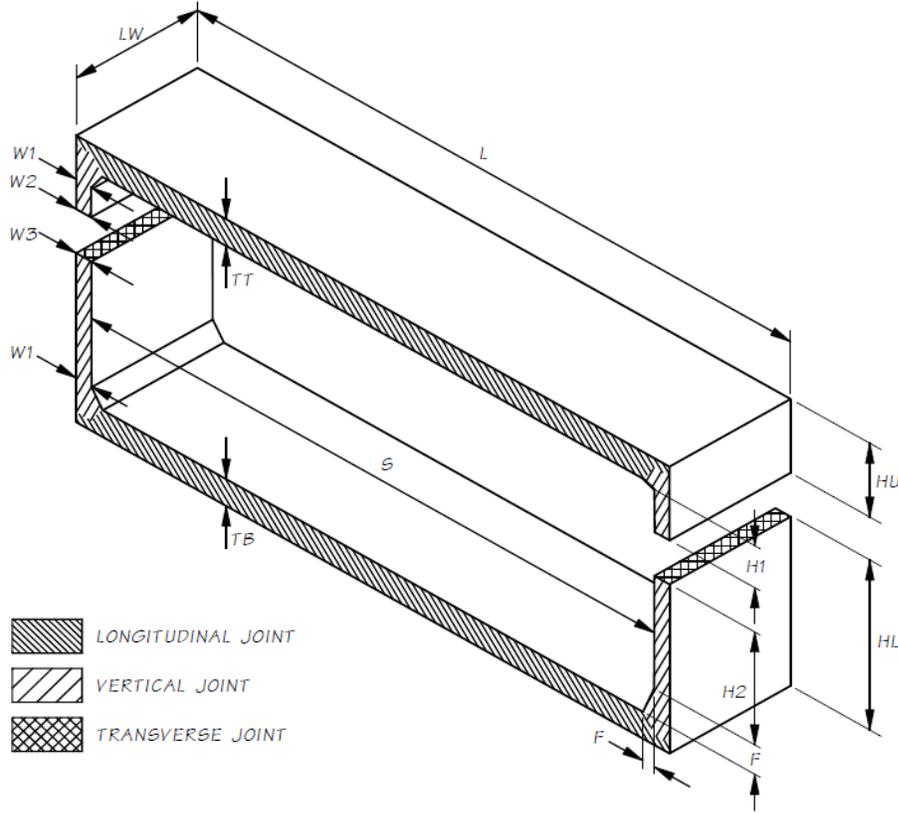
L	=	Length (Out To Out)	W1	=	Wall Thickness At Fillet
LW	=	Lay Width Or 'Lay Length'	W2	=	Wall Thickness At Joint
S	=	Span Length	W3	=	Wall Thickness At Joint
HU	=	Height Of Upper Unit	TT	=	Thickness Of Top Slab
HL	=	Height Of Lower Unit	TB	=	Thickness Of Bot. Slab
H1	=	Height Of Tapered Wall	F	=	Fillet Height & Width
H2	=	Height Of Tapered Wall			

2. Distribution of Live Load through Earth Fill

The distribution of wheel loads through earth fill shall be in accordance with *LRFD Specifications,* Section 3.6.1.2.6a except the 5th paragraph shall be modified as follows:

5th Paragraph – Where the live load and dynamic load allowance moment in concrete slabs, based on the distribution of the wheel load through earth fills, exceeds the live load and dynamic load allowance moment calculated according to

Article 4.6.2.10, the latter moment shall be used.

Where the depth of fill is:

- Less than 2.0 feet, live load shall be distributed to the top slab in accordance with *LRFD Specifications*, Section 4.6.2.10.

- 2.0 feet or greater, live load shall be distributed to the top slab in accordance with *LRFD Specifications*, Section 3.6.1.2.6.

The effects of Live Load may be neglected for;

- <u>Single-Span Concrete Box Structures</u>, when the depth of fill is greater than 8.0 feet and exceeds the span length.

- <u>Multiple-Span Concrete Box Structures</u>, when the depth of fill exceeds the distance between inside faces of end walls.

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 <u>Figure 8.3.6.A-1</u>):

  Joints between adjacent units shall be designed for the applied lateral forces identified in <u>Section 8.3.3.B.</u>

- Horizontal Joints (See <u>Figure 8.3.6.A-1</u>):

  Transverse joints between the upper units and lower units shall be designed for the applied lateral forces identified in <u>Section 8.3.3.B.</u>

  Longitudinal joints in the bottom slab shall be designed for the applied vertical forces identified in <u>Section 8.3.3.B</u>, in addition to any differential settlement as warranted by the geotechnical engineer.

  Longitudinal joints in the top slab shall be designed for the applied vertical forces identified in <u>Section 8.3.3.B</u>, in addition to any differential settlement as warranted by the geotechnical engineer, and shall incorporate a method of shear transfer between adjacent units in accordance with <u>Section 8.3.5.A-2</u> as applicable.

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

Alternate joint types may be proposed addressing applied forces, differential settlement between segments, live load deflection, shear transfer, and prevention of water and soil migration through the joint. If an alternate joint design is accepted it

shall have equal or greater capacity as the joint shown on the Contract Plans. The designer shall provide a note on the Plans stating the capacity of the joint detailed.

Example Note:

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

Each joint shall be sealed to prevent exfiltration or infiltration of soil fines or water. Field tests may be required by the Engineer whenever there is a question regarding compliance. See Bridge Standard Drawing 8.3.2-A8 for joint sealing details.

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

Figure 8.3.6.A-3   Precast Box Joint Details to Facilitate Installation
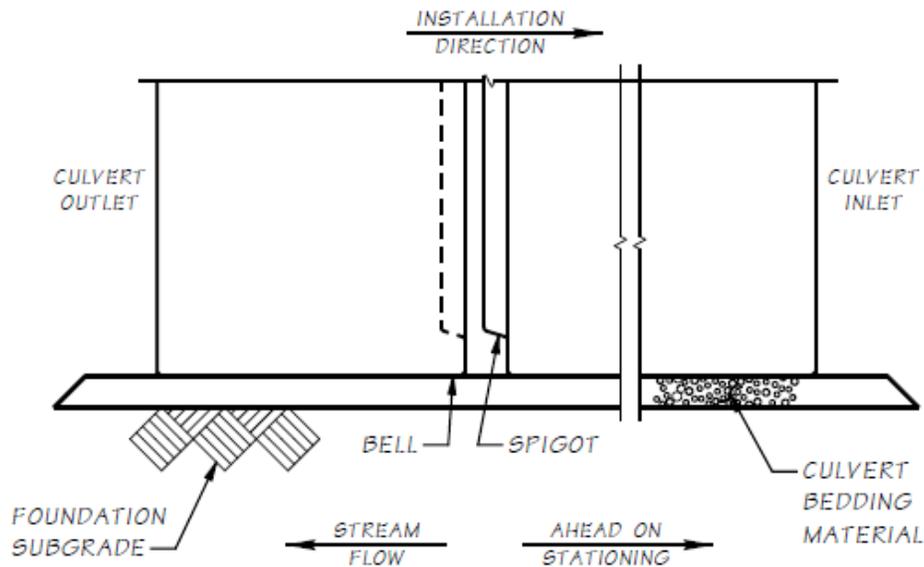
The orientation of the bell and spigot should alternate between the bottom units and the top units. This alternating orientation is preferred, as it provides the most flexibility to facilitate staged construction requirements, and has an inherent placement advantage during construction.

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

trying to slide a bell end under a spigot.

B. Design of Reinforced Concrete Three-Sided Structures

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

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

1. Precast Geometric Limitations

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

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

2. Distribution of Live Load through Earth Fill

    If soil cover is not provided, the top of precast reinforced three-sided structures shall be designed for direct application of vehicular loads.

    The distribution of wheel loads through earth fill shall be in accordance with *LRFD Specifications*, Section 3.6.1.2.6a except the 5th paragraph shall be modified as follows:

    5th Paragraph – Where the live load and dynamic load allowance moment in concrete slabs, based on the distribution of the wheel load through earth fills, exceeds the live load and dynamic load allowance moment calculated according to ~~Articles 4.6.2.1 and 4.6.3.2~~ Article 4.6.2.10, the latter moment shall be used.

    Where the depth of fill is:

    - Less than 2.0 feet, live load shall be distributed to the top slab in accordance with *LRFD Specifications*, Equation 4.6.2.10.2-1, regardless of the direction of travel.

    - 2.0 feet or greater, live load shall be distributed to the top slab in accordance with *LRFD Specifications*, Section 3.6.1.2.6.

    - The effects of Live Load may be neglected for;

      Single-Span Buried Structures, when the depth of fill is greater than 8.0 feet and exceeds the span length.

> > Multiple-Span Buried Structures, when the depth of fill is exceeds the distance between inside faces of end walls.

> 3. Joint Design and Details

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

> - Vertical Joints (See Figure 8.3.6.A-1):

>   Joints between adjacent units shall be designed for the applied lateral forces identified in Section 8.3.3.B.

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

>   Transverse joints between the upper units and foundation units shall be designed for the applied lateral forces identified in Section 8.3.3.B, employing a shear key, block restrainer, or dowel bars. See Bridge Standard Drawing 8.3.2-A6 for connection details.

>   Longitudinal joints in the top slab shall be designed for the applied vertical forces identified in Section 8.3.3.B, in addition to any differential settlement as warranted by the geotechnical engineer, and shall incorporate a method of shear transfer between adjacent units accordance with Section 8.3.5.A-2 as applicable.

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

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

> Example Note:

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

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

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 by the AASHTO *LRFD Bridge Design Specification* and the following: Seismic design effects shall satisfy the requirements of ACI 350.3-06 "*Seismic Design of Liquid-Containing Concrete Structures*". Requirements for Joints and jointing shall satisfy the requirements of ACI 350-06. Two references for tank design are the PCA publications *Rectangular Concrete Tanks*, Revised 5th Edition (1998) and *Design of Liquid-Containing Structures for Earthquake Forces* (2002).

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

Buoyant forces from high ground water conditions should be investigated for permanent as well as construction load cases so the vault does not float. Controlling loading conditions may include: backfilling an empty vault, filling the vault with stormwater before it is backfilled, or seasonal maintenance that requires draining the vault when there is a high water table. In all Limit States, the buoyancy force (*WA*) load factor shall be taken as $\gamma_{WA} = 1.25$ in AASHTO LRFD Table 3.4.1-1. In the Strength Limit State, the load factors that resist buoyancy (γ_{DC}, γ_{DW}, γ_{ES}, Etc.) shall be their minimum values, in accordance with AASHTO LRFD Table 3.4.1-2 and the entire vault shall be considered empty.

During the vault construction, the water table shall be taken as the seal vent elevation or the top of the vault, if open at the top. In this case the load factors that resist buoyancy shall be their minimum values, except where specified as a construction load, in accordance with AASHTO LRFD Section 3.4.2.

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

For Buoyancy without tie-downs:

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

For Buoyancy with tie-downs:

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

Where:

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

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

ACI 350-06 has stricter criteria for cover and spacing of joints than the AASHTO LRFD. Cover is not to be less than 2 inches (ACI 7.7.1), no metal or

other material is to be within 1½ inches from the formed surface, and the maximum bar spacing shall not exceed 12 inches (ACI 7.6.5).

Crack control criteria is in accordance with AASHTO LRFD Section 5.6.7 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 details from WSDOT Precast Prestressed Slab Details or approved equivalent, with weld ties spaced at 4′-0″ on center. Modifications to the above joints shall be justified with calculations. Calculations shall be provided for all grouted shear connections. The width of precast panels shall be increased to minimize the number of joints between precast units.

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

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

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

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

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

Minimum vault dimensions shall be 4′-0″ wide and 7′-0″ tall; inside dimensions.

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

8.3.8   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, *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.* This document shall be used for all WSDOT tunnels. NFPA 502 – Standard for Road Tunnels, Bridges, and Other Limited Access Highways, uses tunnel length to dictate minimum fire protection requirements:

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

Some recent WSDOT tunnel projects are:

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

This 1500 foot long tunnel is part of the major improvement of Interstate 90. Work was started in 1983 and completed in 1988. The net interior diameter of the bored

portion, which is sized for vehicular traffic on two levels with a bike/pedestrian corridor on the third level, is 63.5 feet. The project is the world's largest diameter tunnel in soft ground, which is predominantly stiff clay. Construction by a stacked-drift method resulted in minimal distortion of the liner and insignificant disturbance at the ground surface above.

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

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

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

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

If you have any questions regarding this policy memorandum, please contact William Miller MilleWi@wsdot.wa.gov at (360) 705-7206, or Bijan Khaleghi Bijan.Khaleghi@wsdot.wa.gov at (360) 705-7181.

cc: Patrick Glassford, Construction Office – 47354

   Craig Boone, Bridge and Structures – 47340