Chapter 15 Structural Design Requirements for Design-Build Contracts — Contents

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Chapter 15 Structural Design Requirements for Design-Build Contracts

15.1 Manual Description

15.1.1 Purpose

This chapter provides the contractual requirements for structural design of WSDOT projects that supersede AASHTO LRFD *Bridge Design Specifications* (LRFD-BDS) and AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (LRFD-SGS).

15.1.2 Specifications

This manual and the following AASHTO Specifications are the foundation design criteria and design practice documents used to design highway bridges and structures in Washington State:

- AASHTO LRFD *Bridge Design Specifications* (LRFD-BDS)
- AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (LRFD-SGS)
- WSDOT *Standard Plans* – A-50 Series Standard Plans for Embankment Geometry at Bridge Ends

15.2 Bridge Configuration Criteria

15.2.1 General

15.2.1.A Structure Conceptual Plan

The Structure Conceptual Plan is part of the Design-Build project Request For Proposal (RFP) Appendix M. The purpose of the Structure Conceptual Plan is to present a baseline structural concept where bridges or Buried Structures are assumed by those preparing the RFP to be appropriate based on the criteria and requirements specified in the RFP. The Structure Conceptual Plan is developed to be consistent with the overall baseline civil roadway concept of the RFP Appendix M. The content of the Structure Conceptual Plan includes the items listed in the Conceptual Plan Checklist of Appendix 15.2-A1.

15.2.1.B Bridge Redundancy

For Bridge Redundancy requirements see Section 2.3.1.H.

15.2.1.C Bridge Deck Drainage

Roadway and bridge deck profiles shall be adjusted as much as possible to avoid having bridge drains on the bridge. If bridge geometry is such that drains are required, the number of drains should be minimized as much as possible while still providing a bridge deck drainage design that meets required standards. The bridge drain assembly and system shall be designed for low maintenance.

15.2.2 Railroad Crossings

15.2.2.A Clearances

For railroad overcrossings, minimum horizontal and vertical clearances shall be as described in the *Union Pacific Railroad-BNSF Railway Guidelines for Railroad Grade Separation Projects*.

15.2.2.B Crash Walls

Crash walls, when required, shall be designed to conform to the criteria of the *AREMA Manual*. To determine when crash walls are required, consult the following:

- *Union Pacific Railroad-BNSF Railway Guidelines for Railroad Grade Separation Projects.*
- *AREMA Manual*
- HQ Design Office Railroad Liaison

15.2.2.C Substructure

For highway over railway grade separations, the top of footings for bridge piers or retaining walls within 25 feet of the nearest existing or future track shall be 6 feet or more below the bases of rail. The footing face shall not be closer than 12 feet (measured perpendicular) to the centerline of the track. Consult with the *Union Pacific Railroad-BNSF Railway Guidelines for Railroad Grade Separation Projects* for additional requirements.

15.2.3 Temporary Bridges

Temporary bridges shall be designed to meet the requirement of BDM Section 10.13 and other BDM Sections as applicable.

15.2.4 Inspection and Maintenance Access

15.2.4.A General

Bridges shall be configured to allow inspectors direct access to bearings, and access to within 3-feet of superstructure surfaces. See also Figure 2.3.11-1 for under-bridge-inspection-truck clearance requirements.

15.2.4.B Bearings

Adequate clearance for maintenance and inspection of bearings shall be provided. The clearance shall be adequate to inspect, remove and replace the bearings.

Jacking points shall be provided for bearing replacement. Jacking points shall be designed to support 200 percent of the calculated lifting load.

15.2.4.C Safety Cables, Handrails, and Anchors

Built-up plate girder bridges with girders 5-feet deep or greater in depth shall be detailed with safety handrails for inspectors walking the bottom flanges. At large gusset plate locations on truss bridges (3-feet wide or wider), cables or lanyard anchors shall be placed on the inside face of the truss so inspectors can utilize bottom lateral gusset plates to stand on while traversing around the main truss gusset plates.

15.2.4.D Abutment Slopes

Slopes in front of abutments shall provide enough overhead clearance to the bottom of the superstructure to access bearings for inspection and possible replacement (3-feet minimum for girder type bridges and 5-feet minimum for concrete slabs).

15.2.4.E Access and Lighting

1. **Concrete Box and Prestressed Concrete Tub Girders**

   See Section 5.2.6 for design criteria.

2. **Composite Steel Box Girders**

   See Section 6.4.9 for design criteria.

3. **Access Doors, Lighting, Receptacles and Penetrations**

   All Access doors shall have a minimum 2′-6″ diameter or 2′-6″ square clear opening. Lock box latches shall be installed on all access doors accessible from ground level. Access hatches shall swing into the box girders and shall be placed at locations that do not impact traffic. Lighting and receptacle requirements shall be in accordance with WSDOT *Design Manual* Chapter 1040. Air vents shall be in accordance with Figures 5.2.6-1 and 5.2.6-2.

   Box girder penetrations (vents and drain holes) greater than one inch in diameter through the exterior shall be covered with galvanized wire mesh screen to prevent vermin and birds from accessing the interior of the box girder. The wires shall have a maximum spacing of ¼ inch in both directions.

15.2.5 Bridge Types

Bridges superstructure types shall conform to the requirements in Section 2.4.1.

15.2.6 Aesthetic Design Elements

The primary goal of the aesthetic design is to build visual compatibility between the new elements and their current surroundings. The existing elements, along with proposed and existing structures, typically establish an identifiable visual characteristic. These existing elements such as lighting fixtures, railings street hardware, construction materials, colors, and finishes, are to be included as an integral part of the new construction program. Examples of new elements may include but are not limited to:

- Bridge structure type
- Bridge structure major elements such as pier and crossbeam form
- Bridge structure minor elements such as railings and light standards
- Retaining wall materials, configuration and finishes
- Noise wall material, configuration and finishes: as viewed from the corridor
- Noise walls: as viewed from the neighborhoods
- Vista view points
- Median and roadside planting areas
- Color selection
- Opportunities for community funded art in accordance with *Design Manual* Chapter 950

Some design elements are planned to be functionally and visually consistent with features in the Project's adjacent structures. Other elements benefit by retaining flexibility within a consistent palette of materials, colors, and design forms in order to provide the design-build- process flexibility in developing potential solution.

The final design and configuration of these Project features and other functional components will require ongoing communication, review, and coordination between the design-build Contractor's design team and WSDOT's review team.

15.2.7 Architectural Design Standards

The RFP documents will include architectural standards. These will accommodate the functional requirements of the preferred design solution as well as address the contextual conditions of the Project area. They will be sensitive to corridor continuity as well as the scale, character and texture of the area.

The standards will provide a reference for the Project's context sensitive design and pass along the findings of the urban design analysis conducted during the Project's planning and early design phases. The standards describe the Project's urban design features and aide in the creation of an attractive facility that will be functional, maintainable over time, as well as add to the area's visual character.

Depending on the project complexity, the standards may be highly detailed and prescriptive or they may be more general in nature, such as a simple list of criteria.

Typical and Minimum Color and Finish Standards shall conform to requirements in Section 2.5.4D.

15.2.8 Methods

The design-builder shall comply with the architectural standards. The design-builder shall also be responsive to the existing urban design documents in the adjacent corridors.

The design-builder shall employ the highest standard of care by implementing national best practices in urban design. The methods shall include, but not limited to, such techniques a Context Sensitive Design (CSS) and Crime Prevention Through Environmental Design (CPTED).

15.2.9 Design-Builder Urban Design Team

Where required in the RFP, the design-builder shall include aesthetic design team member resources. This shall include an experienced urban designer capable of working with other team members and to address final context sensitive design issues, construction details, and special project design features. The urban designer shall be an architect with urban project experience.

The architect shall be licensed in the State of Washington and be responsible for the coordination and development of the project's architectural components. Preference shall be given to a team with an architect experienced in bridge architecture. Preference shall also be given to teams where the architect has a current standing with professional organizations such as the American Institute of Architects (AIA) or the American Institute of City Planners (AICP). The architect shall seal the applicable design documents.

When required by the RFP, and in order to assure consistency with the RFP architectural design standards, the design-builder shall form an Urban Design Team. The team shall consist of, as a minimum, a design builder project urban design manager, a WSDOT Bridge and Structures Office Structures Engineer, the WSDOT State Bridge and Structures Architect and the Region or HQ Principal Landscape Architect.

15.2.10 Analysis and Design Criteria for Structural Widenings and Modifications

The widening of a bridge shall be of a similar superstructure type as the existing. The overall appearance and geometrical dimensions of the widening shall be the same or as close as possible to those of the existing structure. Materials used in the construction of the widening shall have the same thermal and elastic properties as the materials in the original structure. Prestressed concrete girders may be used to widen existing cast-in-place concrete structures.

The members of the widening shall be proportioned to provide similar longitudinal and transverse load distribution characteristics as the existing structure.

Differential settlement between the new and existing structures shall be taken into account.

The design of the widening shall conform to current standards and not the standards used to design and construct the existing structure. The strength of the existing structure shall be checked utilizing current design standards. Existing components shall be strengthened as necessary so that their capacity/demand ratios are not worsened. Seismic design of bridge widenings shall be in accordance with Section 4.3.

Diaphragms for the widening shall coincide with and be parallel to the existing diaphragms.

Falsework for the widening shall be supported from the existing structure if the widening does not require additional girders or substructure. Otherwise, falsework for the widening shall not be supported from the existing structure.

If the widening requires additional girders or substructure, a closure strip shall be provided. All falsework supporting the widening shall be released prior to placing concrete in the closure strip. Formwork supporting the closure strip shall be supported from the existing structure and the widening.

15.2.11 Bridge Security

15.2.11.A General

Where required in the RFP, new bridges shall be designed for security. Bridge abutments in particular shall be designed to deter inappropriate public use and access by illegal urban campers.

The Design-Build Contractor shall coordinate with the project urban design team to identify deterrence strategies. The principles of CPTED (Crime Prevention Through Environmental Design) shall be employed with two strategic options. The first strategy employs natural surveillance and territorial reinforcement. For conditions where the first strategy is not feasible, then a second strategy shall be provided. The second strategy provides hard armoring, such as security fences.

15.2.11.B Natural Surveillance and Territorial Reinforcement

The natural surveillance and territorial reinforcement strategy shall be provided through the following:

1. The distance from the top of abutment wall to the finished grade at the face of abutment shall not be less than 10 feet in height and,
2. Horizontal graded landform shelves at the abutment face beneath superstructures shall be omitted and,
3. Alcove spaces within the abutment-superstructure interface shall be omitted and,
4. Unobstructed views for law enforcement surveillance shall be provided.

15.2.11.C Hard Armoring

The hard armoring strategy shall consist of one of the following or a combination of both:

1. A security fence system with an anti-cut, anti-climb, galvanized steel welded wire mesh fabric. The steel welded wire mesh fabric shall have a minimum wire spacing of ½ inch for horizontal elements and 3 inches vertical elements. The minimum wire diameter shall be 0.162 inch (8 gauge) steel welded wire mesh. The fence system shall have the components shown in Figure 2.8.3-2. The bridge security fence shall not be connection to the bridge superstructure. The security fence may be attached to the bridge abutment, curtain walls, girder seats or retaining walls.
2. Curtain walls may be used in lieu of a security fence system. Cast in place concrete, precast concrete, or concrete masonry unit materials may be constructed as curtain walls provided they meet the project urban design goals. Figure 2.8.3-1 shows a schematic view of the curtain wall option.

15.3 Load Criteria

15.3.1 Scope

LRFD-BDS shall be the minimum design criteria used for all projects. Additional requirements, exceptions, and deviations from LRFD-BDS requirements are contained herein.

15.3.2 Load Factors and Load Combinations

A value of 1.0 shall be used for η_i in Equation 3.4.1-1 of LRFD-BDS except for the design of columns when a minimum value of γ_i is required by Article 3.4.1 of LRFD-BDS. In such a case, η_i shall be 0.95.

Strength IV load combination shall not be used for foundation design. For foundation design, loads shall be factored after distribution through structural analysis or modeling.

The design live load factor for the Service III Limit State load combination shall be as follows:

γ_{LL} = 0.8 when the requirements of Sections 5.6.1 and 5.6.2 are satisfied and stress analysis is based on gross section properties.

γ_{LL} = 1.0 when the requirements of Sections 5.6.1 and 5.6.2 are satisfied and stress analysis is based on transformed section properties.

In special cases that deviate from the requirements of Sections 5.6.1 and 5.6.2 and have been approved by the WSDOT State Bridge Design Engineer, γ_{LL}, shall be as specified in the LRFD-BDS.

The Service III live load factor for load rating shall be 1.0.

The live load factor for Extreme Event-I Limit State shall be 0.5. The base construction temperature shall be taken as 64°F for the determination of Temperature Load.

The Load Factors for Permanent Loads Due to Superimposed Deformations are provided in Table 3.53. Table 3.5-3 replaces Table 3.4.1-3 of LRFD-BDS.

15.3.3 Permanent Loads

The design unit weights of common permanent loads shall be as shown in Table 3.8-1.

15.3.3.A Future Deck Overlay Requirement

All new bridge designs with a concrete driving surface, excluding modified concrete overlays, shall be designed for a 35 psf future wearing surface load. The future wearing surface load does not apply to girder deflection, “A” dimension, creep, or profile grade calculations.

Concrete bridge deck protection systems shall be in accordance with Section 5.7.4 for new bridge construction and widening projects.

15.3.4 Live Loads

15.3.4.A Design Live Load

The design live load shall be:

- For new bridges and bridges that are modified in such a way to include new substructure elements – Live load in accordance with LRFD-BDS
- For bridges modified in such a way that do not include new substructure elements – Live load criteria of the original design
- For bridges used for temporary detour or other temporary purposes – minimum 75 percent of HL-93 live load in accordance with LRFD-BDS
- The application of design vehicular live loads shall be as specified in LRFD-BDS Section 3.6.1.3. The design tandem, or "low boy", defined in LRFD-BDS Section C3.6.1.1 shall be included in the design vehicular live load.
- The effect of one design tandem combined with the effect of the design lane load specified in LRFD-BDS Article 3.6.1.2.4 and, for negative moment between the points of contraflexure under a uniform load on all spans and reactions at interior supports, shall be investigated a dual design tandem spaced from 26.0 feet to 40.0 feet apart, measured between the trailing axle of the lead vehicle and the lead axle of the trailing vehicle, combined with the design lane load. For the purpose of this article, the pairs of the design tandem shall be placed in adjacent spans in such position to produce maximum force effect. Axles of the design tandem that do not contribute to the extreme force effect under consideration shall be neglected.

15.3.4.B Live Load Deflection Evaluation

Article 2.5.2.6.2 of the LRFD-BDS is mandatory in its entirety.

15.3.4.C Distribution to Superstructure

15.3.4.C.1 *Cross sections a, b, c, e, k, and also i and j if sufficiently connected to act as a unit from LRFD-BDS Table 4.6.2.2.1-1*

The live load distribution factor shall be as follows:

- For exterior girder design with slab cantilever length equal or less than 40 percent of the adjacent interior girder spacing, use the live load distribution factor for the adjacent interior girder. The slab cantilever length is defined as the distance from the centerline of the exterior girder to the edge of the slab.
- For exterior girder design with slab cantilever length exceeding 40 percent of the adjacent interior girder spacing, use the lever rule with the multiple presence factor of 1.0 for single lane to determine the live load distribution. The live load used to design the exterior girder shall not be less than the live load used for the adjacent interior girder.
- The rigid cross section analysis for steel beam-slab bridge cross sections described in LRFD-BDS Section C4.6.2.2.2d shall not be used to determine live load distribution unless it can be demonstrated that the effectiveness of diaphragms on the lateral distribution of vehicular live load causes the cross section of the structure to deflect and rotate as a rigid cross section.

15.3.4.C.2 *Cross section Type d from LRFD-BDS Table 4.6.2.2.1-1*

This type of cross section shall be designed as a single unit. The live load force effects shall be that of a single lane of live load multiplied by the product of the live load distribution factor for interior girders computed in accordance with LRFD-BDS and the total number of webs in the cross section. The correction factor for live load distribution for skewed supports as specified in LRFD-BDS Tables 4.6.2.2.2e-1 and 4.6.2.2.3c1 for shear shall apply.

15.3.4.C.3 *Distribution to Substructure*

The number of traffic lanes to be used in the substructure design shall be determined by dividing the entire roadway slab width by 12-feet. No fractional lanes shall be used. Bridge deck widths of less than 24 feet shall have a minimum of two design lanes.

15.3.4.C.4 *Distribution to Crossbeam*

The design and load rating shall be distributed to the substructure by placing wheel line reactions in a lane configuration that generates the maximum force effects in the substructure. Live loads are considered to act directly on the substructure without further distribution through the superstructure as illustrated in Figure 3.9-1.

For steel and prestressed concrete superstructure where the live load is transferred to substructure through bearings, cross frames or diaphragms, the girder reaction may be used for substructure design.

15.3.5 Noise Barrier Walls

Wind on Noise Walls shall be as specified in LRFD-BDS Sections 3.8.1, 3.8.1.2.4, and 15.8.2.

15.4 Seismic Design and Retrofit

15.4.1 General

This chapter, Chapter 4 and the LRFD-SGS are the foundation seismic design criteria documents used to design highway bridges in Washington State.

This chapter supplements and supersedes Chapter 4 and the LRFD-BDS by providing WSDOT seismic design criteria, policy and practice.

15.4.2 WSDOT Additions and Modifications to AASHTO Guide Specifications for LRFD Seismic Bridge Design

WSDOT modifications to the LRFD-SGS are as follows:

15.4.2.A Definitions

LRFD-SGS Article 2

Revise existing definitions and add new definitions as follows:

- **Owner**
  Person or agency having jurisdiction over the bridge. For WSDOT projects, regardless of delivery method, the term "Owner" in these Guide Specifications shall be the WSDOT State Bridge Design Engineer and/or the WSDOT State Geotechnical Engineer.

15.4.2.B Earthquake Resisting Systems (ERS) Requirements for Seismic Design Categories (SDCs) C and D

LRFD-SGS Article 3.3

WSDOT Global Seismic Design Strategies:

- **Type 1**
  Ductile Substructure with Essentially Elastic Superstructure. This category is permissible.
- **Type 2**
  Essentially Elastic Substructure with a Ductile Superstructure. This category is not permissible.
- **Type 3**
  Elastic Superstructure and Substructure with a Fusing Mechanism between the two. This category is not permissible.

For Type 1 ERS for SDC C or D, if columns or pier walls are considered an integral part of the energy-dissipating system but remain elastic at the demand displacement, the forces to use for capacity design of other components shall be a minimum of 1.2 times the elastic forces resulting from the demand displacement in lieu of the forces obtained from overstrength plastic hinging analysis. Because maximum limiting inertial forces provided by yielding elements acting at a plastic mechanism level is not effective in the case of elastic design, the following constraints are imposed.

1. Unless an analysis that considers redistribution of internal structure forces due to inelastic action is performed, all substructure units of the frame under consideration and of any adjacent frames that may transfer inertial forces to the frame in question shall remain elastic at the design ground motion demand.

2. Effective member section properties shall be consistent with the force levels expected within the bridge system. Reinforced concrete columns and pier walls shall be analyzed using cracked section properties. For this purpose in absence of better information or estimated by LRFD-SGS Figure 5.6.2-1, a moment of inertia equal to one-half that of the uncracked section shall be used.
3. Foundation modeling shall be established such that uncertainties in modeling will not cause the internal forces of any elements under consideration to increase by more than 10 percent.
4. When site-specific ground response analysis is performed, the response spectrum ordinates shall be selected such that uncertainties will not cause the internal forces of any elements under consideration to increase by more than 10 percent.
5. Thermal, shrinkage, prestress or other forces that may be present in the structure at the time of an earthquake shall be considered to act in a sense that is least favorable to the seismic load combination under investigation.
6. P-Delta effects shall be assessed using the resistance of the frame in question at the deflection caused by the design ground motion.
7. Joint shear effects shall be assessed with a minimum of the calculated elastic internal forces applied to the joint.
8. Detailing as normally required in either SDC C or D, as appropriate, shall be provided.

Use of expected material strengths for the determination of member strengths except shear for elastic response of members is permitted.

The use of elastic design in lieu of overstrength plastic hinging forces for capacity protection described above shall only be considered if designer demonstrates that capacity design of Article 4.11 of the AASHTO *Guide Specifications for LRFD-SGS* is not feasible due to geotechnical or structural reasons.

If the columns or pier walls remain elastic at the demand displacement, shear design of columns or pier walls shall be based on 1.2 times elastic shear force resulting from the demand displacement and normal material strength shall be used for capacities. The minimum detailing according to the bridge seismic design category shall be provided.

Type 3 ERS may be considered only if Type 1 strategy is not suitable and Type 3 strategy has been deemed necessary for accommodating seismic loads. Isolation bearings shall be designed in accordance with the AASHTO *Guide Specifications for Seismic Isolation*. Isolation bearings shall conform to Section 9.3.

Limitations on the use of ERS and ERE are shown in BDM Figures 4.2.2-1, 4.2.2-2, 4.2.2-3, and 4.2.3-4.

- Figure 4.2.2-2 Type 6, connection with moment reducing detail should only be used at column base if proved necessary for foundation design. A fixed connection at base of column remains the preferred option for WSDOT bridges.
- The design criteria for column base with moment reducing detail shall consider all applicable loads at service, strength, and extreme event limit states.
- 4.2.2-3 Types 6 and 8 are not permissible for non-liquefied configuration and permissible for liquefied configuration.

15.4.2.C Seismic Ground Shaking Hazard

LRFD-SGS Article 3.4

For bridges that are considered Critical or Recovery or Ordinary bridges with a site Class F, the seismic ground shaking hazard shall be determined in accordance with the site specific procedure in Article 3.4.3 of the LRFD-SGS.

15.4.2.D Temporary and Staged Construction

LRFD-SGS Article 3.6

For bridges that are designed for a reduced seismic demand, the contract plans shall either include a statement that clearly indicates that the bridge was designed as temporary using a reduced seismic demand or show the Acceleration Response Spectrum (ARS) used for design. No liquefaction assessment required for temporary bridges.

15.4.2.E Balanced Stiffness Requirements and Balanced Frame Geometry Recommendation

LRFD-SGS Articles 4.1.2 and 4.1.3

Balanced stiffness between bents within a frame and between columns within a bent and balanced frame geometry for adjacent frames are required for bridges in both SDCs C and D.

15.4.2.F Selection of Analysis Procedure to Determine Seismic Demand

LRFD-SGS Article 4.2

Minimum requirements for the selection of the analysis procedure to determine seismic demand shall be as specified in Tables 4.2-1 and 4.2-2 of the Guide Specifications, except Procedure 1 (Equivalent Static Analysis) shall not be used for WSDOT Bridges.

15.4.2.G Member Ductility Requirement for SDCs C and D

LRFD-SGS Article 4.9

In-ground hinging for drilled shaft and pile foundations may be considered for the liquefied configuration if allowed by the RFP Criteria.

15.4.2.H Longitudinal Restrainers

LRFD-SGS Article 4.13.1

Longitudinal restrainers shall be provided at the expansion joints between superstructure segments. Restrainers shall be designed in accordance with the FHWA *Seismic Retrofitting Manual for Highway Structure* (FHWAHRT06032), Article 8.4, the Iterative Method. Restrainers shall be detailed in accordance with the requirements of Guide Specifications Article 4.13.3 and *Bridge Design Manual* Section 4.4.4. Restrainers may be omitted for SDCs C and D where the available seat width exceeds the calculated support length specified in LRFD-SGS Equation C4.13.1-1.

Longitudinal restrainers shall not be used at the end piers (abutments).

15.4.2.I Abutments

LRFD-SGS Article 5.2 – Diaphragm Abutment type shown in Figure 5.2.3.2-1 shall not be used for WSDOT bridges.

LRFD-SGS Article 5.2

Abutments are revised as follows:

15.4.2.I.1 *4.2.11.1 - General*

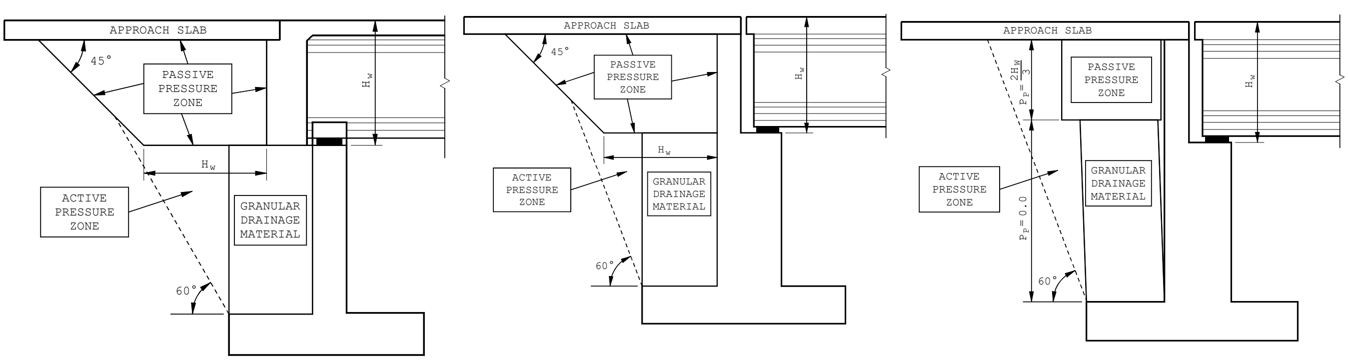
The participation of abutment walls in providing resistance to seismically induced inertial loads may be considered in the seismic design of bridges either to reduce column sizes or reduce the ductility demand on the columns. Damage to backwalls and wingwalls during earthquakes may be considered acceptable when considering no collapse criteria, provided that unseating or other damage to the superstructure does not occur. Abutment participation in the overall dynamic response of the bridge system shall reflect the structural configuration, the load transfer mechanism from the bridge to the abutment system, the effective stiffness and force capacity of the wall-soil system, and the level of acceptable abutment damage. The capacity of the abutments to resist the bridge inertial loads shall be compatible with the soil resistance that can be reliably mobilized, the structural design of the abutment wall, and whether the wall is permitted to be damaged by the design earthquake. The lateral load capacity of walls shall be evaluated on the basis of a rational passive earth-pressure theory.

The participation of the bridge approach slab in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically induced inertial loads may be considered if allowed by the RFP Criteria.

15.4.2.I.2 *4.2.11.2 - Longitudinal Direction*

The abutment may be considered as part of the ERS for a continuous superstructure. If the abutment is considered as part of the longitudinal ERS, the abutment stiffness and capacity shall be determined as illustrated schematically in Figure 4.2.11-1a for semi-integral abutments, Figure 4.2.11-1b for L-shaped abutments with backwall fuse, and Figure 4.2.11-1c for L-shaped abutments without backwall fuse.

Figure 4.2.11-1 Abutment Stiffness and Passive Pressure Estimate



(a) Semi-integral Abutment (b) L-shape Abutment Backwall Fuses (c) L-shape Abutment Backwall Does Not Fuse

Abutments shall be designed to sustain the design earthquake displacements. The passive abutment resistance shall be limited to 70 percent of the value obtained using the procedure given in Article 4.2.11.2.1.

15.4.2.I.3 *4.2.11.3 - Transverse Direction*

Transverse stiffness of abutments may be considered in the overall dynamic response of bridge systems if allowed by RFP Criteria. The transverse abutment stiffness used in the elastic demand models shall be taken as 50-percent of the elastic transverse stiffness of the adjacent bent.

Girder stops are expected to fuse at the design event earthquake level of acceleration to limit the demand and control the damage in the abutments and supporting piles/shafts. The forces generated with elastic demand assessment models shall not be used to size the abutment girder stops. Girder stops for abutments supported on a spread footing shall be designed to sustain the lesser of the acceleration coefficient, A_s, times the superstructure dead load reaction at the abutment plus the weight of abutment and its footing or sliding friction forces of spread footings. Girder stops for pile/shaft-supported foundations shall be designed to sustain the sum of 75 percent total lateral capacity of the piles/shafts and shear capacity of one wingwall.

The stiffness of fusing or breakaway abutment elements such as wingwalls (yielding or non-yielding), elastomeric bearings, and sliding footings shall not be relied upon to reduce displacement demands at intermediate piers.

Unless fixed bearings are used, girder stops shall be provided between all girders regardless of the elastic seismic demand. The design of girder stops shall accommodate unequal forces that may develop in each stop.

When fusing girder stops, transverse shear keys, or other elements that potentially release the restraint of the superstructure are used, then adequate support length meeting the requirements of Article 4.12 of the LRFD-SGS shall be provided in the transverse direction as well as the longitudinal direction. Additionally, the expected redistribution of internal forces in the superstructure and other bridge system element shall be considered. Bounding analyses considering incremental release of transverse restraint at each end of the bridge shall also be considered.

15.4.2.I.4 *4.2.11.4 - Curved and Skewed Bridges*

The passive pressure resistance in soils behind semi-integral or L-shape abutments shall be based on the projected width of the abutment wall normal to the centerline of the bridge. Abutment springs shall be included in the local coordinate system of the abutment wall.

15.4.2.J Foundation – General

LRFD-SGS Article 5.3.1

The required Foundation Modeling Method (FMM) and the requirements for estimation of foundation springs for spread footings, pile foundations, and drilled shafts shall be Modeling Method II as defined in Table 5.3.1-1.

15.4.2.K Foundation – Spread Footing

LRFD-SGS Article C5.3.2

Foundation springs for spread footings shall be determined in accordance with Section 7.2.7 and *Geotechnical Design Manual* Section 6.5.1.1.

15.4.2.L Procedure 3: Nonlinear Time History Method

LRFD-SGS Article 5.4.4

The time histories of acceleration used to describe the earthquake loads shall be selected in accordance with *Geotechnical Design Manual* Section 6-A.6.

15.4.2.M Drilled Shafts

LRFD-SGS Article C6.5

For WSDOT bridges, the scale factor for p-y curves or subgrade modulus for large diameter shafts shall not be used.

15.4.2.N Liquefaction Design Requirements

LRFD-SGS Article 6.8

Soil liquefaction assessment shall be based on *Geotechnical Design Manual* Section 6.4.2.6.

15.4.2.O Reinforcing Steel

Longitudinal reinforcement for ductile members in SCD's B, C & D, including foundations where in-ground-hinging is considered as part of the ERS, shall conform to ASTM A706 Grade 60. See Section 5.1.2 for other requirements.

For SDCs B, C, and D, the moment-curvature analyses based on strain compatibility and nonlinear stress strain relations shall be used to determine the plastic moment capacities of all ductile concrete members. The properties of reinforcing steel, as specified in Table 8-4.2-1, shall be used.

Deformed welded wire fabric shall not be used.

15.4.2.P Concrete Modeling

LRFD-SGS Article 8.4.4

Where in-ground plastic hinging is part of the ERS, the confined concrete core shall be limited to a maximum compressive strain as specified in AASHTO Guidelines for Performance-Based Seismic Table 3.2-4, and the member ductility demand shall be limited to 4 maximum.

15.4.2.Q Lateral Confinement for Non-Oversized Strengthened Pile Shaft for SDCs C and D

LRFD-SGS Article 8.8.13

Non-oversized column-shaft (the cross section of the confined core is the same for both the column and the pile shaft) is not permissible unless allowed by the RFP Criteria.

15.4.2.R Requirements for Capacity Protected Members

LRFD-SGS Article 8.9

For SDCs C and D where liquefaction is identified, pile and drilled shaft inground hinging may be considered as an ERE.

Bridges shall be analyzed and designed for the non-liquefied condition and the liquefied condition in accordance with Article 6.8. The capacity protected members shall be designed in accordance with the requirements of Article 4.11. To ensure the formation of plastic hinges in columns, oversized pile shafts shall be designed for an expected nominal moment capacity, M_{ne}, at any location along the shaft, that is, equal to 1.25 times moment demand generated by the overstrength column plastic hinge moment and associated shear force at the base of the column. The safety factor of 1.25 may be reduced to 1.0 depending on the soil properties.

The design moments below ground for extended pile shaft may be determined using the nonlinear static procedure (pushover analysis) by pushing them laterally to the displacement demand obtained from an elastic response spectrum analysis. The point of maximum moment shall be identified based on the moment diagram. The expected plastic hinge zone shall extend 3D above and below the point of maximum moment. The plastic hinge zone shall be designated as a “no splice” zone and the transverse steel for shear and confinement shall be provided accordingly.

15.4.2.S Superstructure Capacity Design for Transverse Direction (Integral Bent Cap) for SDCs C and D

LRFD-SGS Article 8.11

For SDCs C and D, the longitudinal flexural bent cap beam reinforcement shall be continuous. Splicing of cap beam longitudinal flexural reinforcement shall be accomplished using mechanical couplers that are capable of developing the tensile strength of the spliced bar. Splices shall be staggered at least 2 feet. Lap splices shall not be used.

15.4.2.T Integral Bent Cap Joint Shear Design

LRFD-SGS Article 8.13.4.1.1

In addition to the T-joints listed in Article 8.13.4.1.1, the exterior column joints for box girder superstructure and other superstructures if the cap beam extends the joint far enough to develop the longitudinal cap reinforcement shall be considered T-joints for joint shear analysis in the transverse direction.

15.4.2.U Cast-in-Place and Precast Concrete Piles

LRFD-SGS Article 8.16.2

Minimum longitudinal reinforcement of 0.75 percent of A_g shall be provided for CIP piles in SDCs B, C, and D. Longitudinal reinforcement shall be provided for the full length of pile.

15.4.3 Seismic Design Requirements for Bridge Modifications and Widening Projects

15.4.3.A Seismic Analysis and Retrofit Policy

The Seismic Analysis and Retrofit Policy for Bridge Modifications and Widening Projects shall conform to Sections 4.3.1, 4.3.2, 4.3.3, and 4.3.4.

Specific seismic requirements for widening of Recovery and Critical bridges are provided in RFP Section 2.13 and Section 4.3.3.

The spectral response parameters shall be as defined in Section 4.2.3.

15.4.3.B Design and Detailing Considerations

15.4.3.B.1 *Differential Settlement*

The designer shall evaluate the potential for differential settlement between the existing structure and widening structure. Additional geotechnical measures may be required to limit differential settlements to tolerable levels for both static and seismic conditions. The bridge designer shall evaluate, design, and detail all elements of new and existing portions of the widened structure for the differential settlement warranted by the Geotechnical Engineer. Angular distortions between adjacent foundations shall not exceed 0.008 (RAD) in simple spans and 0.004 (RAD) in continuous spans.

The horizontal displacement of pile and shaft foundations shall be estimated using procedures that consider soil structure interaction (see *Geotechnical Design Manual* Section 8.12.2.3). Horizontal movement criteria shall be established at the top of the foundation based on the tolerance of the structure to lateral movement with consideration of the column length and stiffness. Tolerance of the superstructure to lateral movement will depend on bridge seat widths, bearing type(s), structure type, and load distribution effects.

15.4.3.B.2 *Isolation Bearings*

Isolation bearings may be used for bridge widening projects to reduce the seismic demand through modification of the dynamic properties of the bridge. Isolation bearings shall be designed in accordance with AASHTO *Guide Specifications for Seismic Isolation* and shall conform to Section 9.3.

15.4.4 Seismic Retrofitting of Existing Bridges

15.4.4.A Seismic Analysis Requirements

The multi-mode spectral analysis of *Seismic Retrofitting Manual* Section 5.4.2.2 (as a minimum) shall be used to determine the seismic displacement and force demands to identify seismically deficient elements of the existing structure. Prescriptive requirements, such as support length, shall be considered mandatory and shall be included in the analysis. Seismic capacities shall be determined in accordance with the requirements of the *Seismic Retrofitting Manual*. Displacement capacities shall be determined by the Method D2 – Structure Capacity/Demand (Pushover) Method of *Seismic Retrofitting Manual* Section 5.6.

The seismic retrofit of Ordinary, Recovery and Critical bridges shall be in accordance with the requirements of the Seismic Retrofitting Manual, and WSDOT BDM Section 4.4. Specific requirements for the seismic retrofit of Recovery and Critical bridges are provided in RFP Section 2.13.

15.4.4.B Seismic Retrofit Design

Table 111, Chapters 8, 9, 10, 11, and Appendices D thru F of the *Seismic Retrofitting Manual* shall be used in selecting and designing the seismic retrofit measures.

15.4.4.C Isolation Bearings

Isolation bearings may be used for seismic retrofit projects to reduce the demands through modification of the dynamic properties of the bridge as a viable alternative to strengthening weak elements of non-ductile bridge substructure members of existing bridge. Isolation bearings shall be designed in accordance with the requirement of the AASHTO *Guide Specifications for Seismic Isolation* and shall conform to Section 9.3.

15.5 Concrete Structures

15.5.1 General

Design of concrete structures for roadway elements such as bridges, lids, retaining walls, noise walls, three-sided structures, traffic barrier, pedestrian barrier, sign structures, and bridge approach slabs, etc., shall be based on the requirements cited herein and in the current LRFD-BDS, LRFD-SGS, WSDOT Special Provisions and the WSDOT *Standard Specifications*.

15.5.2 Materials

15.5.2.A Concrete

15.5.2.A.1 Cast-in-place (CIP) Concrete

Cast-in-place (CIP) concrete shall meet the requirements of Table 15.5.2-1:

Table 15.5.2-1

Component or Application	Minimum Numerical Class and Minimum Compressive Strength at 28 days (psi)	Letter Suffix	Compressive Strength for use in Design = f'c (psi)
Commercial Concrete; Non-structural Concrete; Sidewalks; Curbs; Gutters	3000	-	Numerical Class
General Structural Concrete including Spread Footings; Walls; Columns; Crossbeams; Box Girders; Slabs; Barriers; etc.	4000	-	Numerical Class
Bridge Approach Slabs	4000	A	Numerical Class
Bridge Decks	4000	D	Numerical Class
Piles and Shafts	4000/5000	P	Numerical Class
Underwater Seals	4000	W	0.6 times Numerical Class

15.5.2.A.2 Modulus of Elasticity

For calculation of the modulus of elasticity, the unit weight of plain concrete (w_c) shall be taken as 0.155 kcf for prestressed concrete girders and 0.150 kcf for normal-weight concrete unless project specific data is available. The correction factor (K_1) shall be taken as 1.0 unless project specific data is available.

15.5.2.A.3 Shrinkage and Creep

Shrinkage and creep shall be calculated with relative humidity (H) taken as 75 percent unless project specific data is available. The maturity of concrete (t) shall be taken as 2000 days. In determining the age of concrete at time of load application (t_i) one day of accelerated curing by steam or radiant heat shall be taken as equal to seven days of normal curing.

15.5.2.A.4 Grout

Grout pads with thickness exceeding 4″ shall be reinforced with steel reinforcement. Non-shrink grout conforming to *Standard Specifications* Section 9-20.3(2) shall be used in keyways between prestressed concrete girders.

15.5.2.A.5 *Mass Concrete*

Concrete placements with a least dimension of greater than 6-feet shall be considered mass concrete, except that shafts need not be considered mass concrete.

The temperature of mass concrete during placement and curing shall not exceed 160°F. The temperature difference between the geometric center of the mass concrete and the center of nearby exterior surfaces during placement and curing shall not exceed 35°F.

A thermal control plan shall be submitted by the Design-Builder for review and comment for mass concrete placements. The thermal control plan may include such things as: a thermal analysis; temperature monitors and equipment; insulation; concrete cooling before placement; concrete cooling after placement, such as by means of internal cooling pipes; use of smaller, less frequent placements; or other methods proposed by the Design-Builder and accepted by the WSDOT Bridge Technical Advisor (BTA).

Concrete mix designs may be optimized (such as by using low-heat cement, fly ash or slag cement, low-water/cement ratio, low cementitious materials content, larger aggregate, etc.) as long as the concrete mix meets other project requirements.

15.5.2.A.6 *Shotcrete*

Shotcrete shall not be used for permanent structures, including exterior wall fascia surfaces, unless allowed by RFP Criteria. Shotcrete may be used for temporary applications.

15.5.2.A.7 *Lightweight Aggregate Concrete*

Lightweight aggregate concrete shall not be used on bridge decks or other components exposed to traffic wheel loads in service. The absorption of the lightweight coarse aggregate for prestressed elements shall not exceed 10 percent when tested in accordance with AASHTO T85.

15.5.2.B Reinforcing Steel

15.5.2.B.1 *Grades*

Steel reinforcing bars shall conform to Section 5.1.2 and Section 15.4.2.T.

15.5.2.B.2 *Compressive Development Length*

The minimum compressive development length shall be 1′-0″.

15.5.2.B.3 *Splices*

Minimum lap splice lengths, for both tension and compression, shall be 2′-0″. When two bars of different diameters are lap spliced, the length of the lap splice shall be the larger of the lap splice for the smaller bar or the development length for the larger bar.

15.5.2.B.4 *Welded Wire Reinforcement in Prestressed Concrete Girders, Walls, Barriers and Deck Panels*

Welded wire reinforcement may be used to replace steel reinforcing bars in prestressed concrete girders, walls, barriers, and deck panels.

Welded wire reinforcement shall be deformed.

Longitudinal wires and welds shall be excluded from regions with high shear demands, including girder webs. Longitudinal wires for anchorage of welded wire reinforcement shall have an area of 40 percent or more of the area of the wire being anchored as described in ASTM A497 but shall not be less than D4.

15.5.2.B.5 *Reinforcing Bar Dowels and Resin Bonded Anchors*

Allowable tensile loads and minimum required embedment for reinforcing bar dowels shall be in accordance with Section 5.5.4.A.4. If it is not possible to obtain this embedment, the allowable load on the dowel shall be reduced by the ratio of the actual embedment divided by the required embedment.

Before core drilling, existing reinforcement shall be located by non-destructive methods or by chipping if existing reinforcement cannot be damaged. Core drilled holes shall be roughened.

15.5.2.C Prestressing Steel

Prestressing steel shall be AASHTO M 203 Grade 270 low relaxation for strands and AASHTO M 275 Type II for bars.

The refined estimate for computing time-dependent losses shall be used.

Partial prestressing is not permitted.

15.5.2.D Post-Tensioning Systems

Multistrand grouted tendons with steel strand systems shall be used for post-tensioned concrete bridge superstructures, spliced girders, and bridge components. For post-tensioned concrete bridge decks, unbonded single strand post tensioning systems may be used.

Multistrand and grouted post-tensioning systems for permanent construction shall be designed and constructed in accordance with Protection Level 2 (PL-2) practices, as defined by the requirements of PTI/ASBI M50.3-19 *Specification for Multistrand and Grouted Post-Tensioning* and PTI M55.1-19 *Specification for Grouting of Post-Tensioned Structures*. Unbonded single strand post-tensioning systems shall be designed and constructed in accordance with PTI M10.2-17 *Specification for Unbonded Single Strand Tendons*.

15.5.3 Design Considerations

15.5.3.A Service and Fatigue Limit States

The exposure factor for LRFD-BDS Section 5.6.7 “Control of Cracking by Distribution of Reinforcement” shall be based upon a Class 2 exposure condition.

Concrete stresses in prestressed members shall be limited to the allowable stresses shown in Table 5.2.1-1.

15.5.3.B Strength Limit State

The shear design of prestressed members shall be based on the general procedure of LRFD-BDS Section 5.7.3.4.2. The shear design of non-prestressed members shall be based on either the general procedure, or the simplified procedure of LRFD-BDS Section 5.7.3.4.1. LRFD-BDS Section 5.8.3.4.3 “Simplified Procedure for Prestressed and Non-prestressed Sections” shall not be used.

The maximum spacing of shear and torsion reinforcement shall be 18 inches.

15.5.3.C Post-Tensioning

Dead end anchorages shall be avoided.

A 2″ minimum clearance shall be provided between post-tensioning ducts.

Confinement reinforcement shall be provided to confine curved post-tensioning tendons in accordance with Section 5.8.1.F.

Structure shortening effects due to post-tensioning shall be included in the design.

The camber shall be shown on the plans and shall include the effect of both dead load and final prestressing.

All post-tensioning anchorages in webs of box girder or multi-stem superstructure should be vertically aligned. Tendons adjacent to post-tensioning anchorages shall meet the minimum tangent length and minimum tendon radii requirements of Section 5.8.1.D.

15.5.4 Superstructures

15.5.4.A Reinforced Concrete Superstructures

The use of CIP reinforced concrete bridge superstructures without post-tensioning shall be restricted to widening existing reinforced concrete bridge superstructures. Longitudinal post-tensioning shall be provided for all new CIP reinforced concrete bridge superstructures.

15.5.4.B Box Girder Superstructures

15.5.4.B.1 *Intermediate Diaphragms for Curved Concrete Box Girder Bridges*

Intermediate diaphragms shall be provided for curved concrete box girder bridges with centerline radius, R, less than 800 feet. Minimum diaphragm spacing shall be as follows:

For 600 feet ≤ R < 800 feet - at midspan.

For 400 feet ≤ R < 600 feet - at ⅓ points of span.

For R < 400 feet - at ¼ points of span.

15.5.4.B.2 *Temperature Effects*

Thermal stresses shall be investigated in design using the following criteria:

1. A mean temperature 50°F with rise 45°F and fall 45°F for longitudinal analysis using one-half of the modulus of elasticity (Maximum Seasonal Variation.)
2. The superstructure box girder shall be designed transversely for a temperature differential between inside and outside surfaces of ±15°F with no reduction in modulus of elasticity (Maximum Daily Variation).
3. The superstructure box girder shall be designed longitudinally for a top slab temperature increase of 20°F with no reduction in modulus of elasticity.

15.5.4.B.3 *Drains*

Drains shall be placed in the bottom slab at the low points of each cell. Drain hole details shall be in accordance with Figure 5.3.8-1.

15.5.4.C Prestressed Concrete Girder Superstructures

15.5.4.C.1 *WSDOT Standard Girder Types and Construction Sequences*

Prestressed concrete girders shall be a WSDOT standard girder type in accordance with Bridge Standard Drawing 5.6-A1-10 through 5.6-A1-13.

Prestressed concrete girder superstructures shall follow a construction sequence in accordance with Bridge Standard Drawing 5.6-A2-1 through 5.6-A2-3.

15.5.4.C.2 *Superstructure Continuity*

Prestressed concrete girder superstructures shall be designed for the envelope of simple span and continuous span loadings for all permanent and transient loads. Loads applied before establishing continuity (typically before placement of continuity diaphragms) need only be applied as a simple span loading. Continuity reinforcement shall be provided at supports for loads applied after establishing continuity.

15.5.4.C.3 *Continuous Structure Configuration*

Girder type, depth and number of lines shall be identical in adjacent spans over intermediate piers. Girder type, depth and number of lines may be changed at expansion joints.

15.5.4.C.4 *Girder Ends*

Prestressed concrete girders shall have a standard end type in accordance with Section 5.6.2.E. Prestressing strands at girder ends shall be extended into diaphragms and made continuous in accordance with Section 5.1.3.D.

Girder end skew angles for trapezoidal tub, slab, wide flange deck, wide flange thin deck and deck bulb-tee prestressed concrete girders shall be limited to 30 degrees. Girder end skew angles for all other prestressed concrete girders shall be limited to 45 degrees.

The splitting resistance of pre-tensioned anchorage zones shall be as described in LRFD-BDS Section 5.9.4.4.1. The end vertical reinforcement shall not be larger than #5 bars and spacing shall not be less than 2½″. The remaining splitting reinforcement not fitting within the h/4 zone may be placed beyond the h/4 zone at a spacing of 2½″.

15.5.4.C.5 *Diaphragms*

Diaphragms for prestressed concrete girder superstructures shall be cast-in-place concrete.

Diaphragms shall be oriented parallel to girder support skew. On curved bridges, diaphragms shall be placed on radial lines. Intermediate and end diaphragms shall be in accordance with Bridge Standard Drawings.

Except for Prestressed Concrete Wide Flange Deck Girder and Prestressed Concrete Slab Girder bridges, intermediate diaphragms shall be provided for all prestressed concrete girder bridges in the following situations:

- Spans crossing a roadway with a minimum vertical clearance of 20'-0" or less.
- Spans crossing a railway with a minimum vertical clearance of 23'-4" or less from the top of rail.
- Spans crossing a water body or waterway with a minimum vertical clearance of 6'-0" of less from the 100-year MRI water surface level.
- Spans that will possibly or likely have vehicular traffic under the span in the future with a minimum vertical clearance of 20'-0" or less.

Intermediate diaphragms shall be equally spaced between bearing centerlines at a spacing not to exceed 50'.

Intermediate diaphragms shall be full depth for structures crossing over roads with average daily traffic (ADT) greater than 50,000, in accordance with Section 5.6.4.C.4.

15.5.4.C.6 *Barrier and Sidewalk Load Distribution*

The dead load of one traffic barrier or sidewalk shall not be distributed over more than three girder webs.

15.5.4.C.7 *Composite Action*

Composite section properties including effective flange width of the composite deck shall be in accordance with Section 5.6.2.B.

15.5.4.C.8 *Dead Loads*

The bridge deck dead load to be applied to a girder shall be based on the full bridge deck thickness. The pad/haunch weight due to the maximum pad/haunch height shall be added to that load over the full length of the girder.

When the depth of the pad/haunch between the top of the prestressed concrete girder and the underside of the deck at the centerline of the girder exceeds 6″, reinforcement shall be provided in the pad in accordance with Figure 5.6.4-2.

15.5.4.C.9 *Girder Stirrups*

Girder stirrups shall be field bent over the top mat of reinforcement in the bridge deck unless pre-bent hooks are allowed by the WSDOT standard girder type or Additional reinforcement is provided in conformance with Section 5.6.2.H.

15.5.4.C.10 *Transformed Section Properties*

Transformed section properties shall not be used for design of prestressed concrete girders. Gross section properties shall be used.

15.5.4.C.11 *Deck Shrinkage*

The elastic gain in prestressing strands due to slab shrinkage shall be computed in accordance with LRFD-BDS Section 5.9.3.4.3.d. Deck shrinkage shall be considered as an external force applied to the composite section for the Service I, Service III, and Fatigue I limit states. The deck shrinkage strain shall be computed as 50-percent of the strain determined by LRFD-BDS Equation 5.4.2.3.3-1.

15.5.4.C.12 *Deck Girder Superstructures*

The term “deck girder” refers to a prestressed concrete girder whose top flange or surface is the driving surface, with or without an overlay, including slab and deck bulb-tee girders.

Unless noted otherwise deck girders that are not connected to adjacent girders shall use a Type 1 deck protection system; girders that only have shear connections with adjacent girders shall use a Type 3 or Type 4 deck protection system; and girders that have moment connections with adjacent girders shall use Type 2 or Type 3 deck protection systems.

Deck girders without a composite CIP deck slab shall have a minimum concrete cover of 2″ over the top mat. The top mat of reinforcement in the top flange shall be epoxy-coated.

15.5.4.C.13 *Slab Girders*

Slab girder spans between centerline bearings shall be limited to the prestressed concrete girder height multiplied by 30. A minimum 5″ composite CIP bridge deck shall be placed over slab girders directly supporting traffic loadings. The CIP concrete bridge deck shall at a minimum be Class 4000D concrete with one layer of #5 epoxy coated reinforcement in both the transverse and longitudinal directions. The longitudinal reinforcement shall be spaced at 12 inches maximum and the transverse reinforcement shall be spaced at 6 inches maximum.

15.5.4.C.14 *Deck Bulb-Tee Girders*

Deck bulb-tee girders shall be limited to pedestrian bridges, temporary bridges and to widening existing similar structures.

15.5.4.C.15 *Wide Flange Deck Girders*

Wide flange deck girders shall be limited to single-span bridges.

The top flanges shall be connected with UHPC. These bridges shall have a polyester polymer concrete overlay or, if the approach roadways are paved with HMA, an HMA overlay is also acceptable. A waterproofing membrane shall be provided with an HMA overlay.

15.5.4.C.16 *Wide Flange Thin Deck Girders*

Two mats of transverse reinforcement in the CIP bridge deck shall be designed to resist live loads and superimposed dead loads using a Type 1 Deck Protection System. The longitudinal reinforcement shall be spaced at 12 inches maximum and the transverse reinforcement shall be spaced at 6 inches maximum.

15.5.4.C.17 *Tub Girders*

Drains shall be placed in the centerline of the bottom flange at the low points of each cell. Drain hole details shall be in accordance with Bridge Standard Drawing 5.6-A9-3.

15.5.4.C.18 *Spliced Prestressed Concrete Girders*

Closure joints shall be CIP concrete with a minimum length of 2′-0″. The sequence of placing concrete for the closure joints and deck shall be specified in the plans.

Concrete cover to web stirrups at the CIP closure at pier diaphragms shall not be less than 2½″. If intermediate diaphragm locations coincide with CIP closures between precast segments, then the concrete cover at the CIP closures shall not be less than 2½″.

Girders shall be post-tensioned prior to deck placement, unless otherwise noted in the RFP.

Ducts for longitudinal post-tensioning shall be kept below the bridge deck.

15.5.5 Concrete Bridge Decks

Concrete bridge decks shall be designed using the Traditional Design of LRFD-BDS Section 9.7.3.

For web spacing in excess of 12 feet or cantilever overhang in excess of 6 feet, transverse post-tensioning shall be provided in the deck.

For structures that include sidewalks, the construction joint between the sidewalk and the deck shall be a smooth surface.

Longitudinal expansion or isolation joints in bridge decks are not permitted.

15.5.5.A Bridge Deck Requirements

The minimum bridge deck thickness shall be 5″ for slab and deck bulb-tee prestressed concrete girder superstructures, 7.5″ for other concrete superstructures, 8.0″ for steel girder superstructures, and 8.5″ (including 3.5″ stay-in-place deck panel and 5″ CIP concrete deck) for superstructures with SIP deck panels. This minimum thickness may be reduced by 0.5″ for bridges with Deck Protection Systems 2, 3 and 5. For bridge deck overhangs that support traffic barriers, the minimum thickness shall be 8”.

The distance from the top of the bridge deck to the top of the girder at centerline bearing at centerline of girder is represented by the “A” Dimension.

A roughened surface or a shear key shall be provided at deck construction joints.

15.5.5.B Bridge Deck Reinforcement

Top transverse reinforcement shall be hooked at the deck slab edge unless a traffic barrier is not used.

Longitudinal deck slab reinforcement shall be provided in accordance with Section 5.7.2.B.

The minimum clearance between top and bottom reinforcing mats shall be 1″.

The minimum cover over the top layer of reinforcement shall be in accordance with the appropriate Deck Protection System. The minimum cover below the bottom layer reinforcement shall be 1.0″.

The minimum amount of reinforcement in each direction shall be 0.18 in^2/feet for the top mat and 0.27 in^2/feet for the bottom mat.

The maximum bar spacing in both transverse and longitudinal directions for the top mat, and transverse direction of the bottom mat shall not exceed 12″. The maximum bar spacing for the bottom longitudinal direction within the effective length, as specified in LRFD-BDS Section 9.7.2.3, shall not exceed the deck thickness.

15.5.5.C Stay-in-Place (SIP) Deck Panels

SIP deck panels shall be precast concrete and their details shall be in accordance with Bridge Standard Drawing 5.6-A10-1. SIP steel deck forms are not permitted.

SIP deck panels shall not be used in longitudinal negative moment regions of continuous superstructures, unless the deck is longitudinally post-tensioned.

For a bridge widening or phased construction, SIP deck panels shall not be used in the bay adjacent to the existing structure.

SIP deck panels shall not be used on prestressed concrete girders with flanges less than 12″ wide.

SIP deck panels shall not be used on steel girder bridge superstructures.

15.5.5.D Bridge Deck Protection

All new bridge decks, precast or cast-in-place slabs, and deck girder structures shall utilize a deck protection system in accordance with this section and Section 5.7.4.A. Widening of existing bridge decks and slab bridges shall be in accordance with Section 5.7.4.B.

15.5.5.E Bridge Deck HMA Paving

Asphalt resurfacing including bituminous surface treatment (BST) on bridge decks and slab bridges shall be in accordance with Section 5.7.5 and the technical requirements in RFP 2.7. Bridge Condition Report (BCR) shall be provided as reference information for each structure.

All structures shall be evaluated for HMA removal and paving train loads in accordance with Section 5.7.5.B.9.

Construction shall be in accordance with the bridge paving specifications included in the RFP.

15.6 Steel Structures

Except for the following items, refer to Chapter 6 for Steel Structure requirements. Where Chapter 6 requires approval from the Bridge Design Office, Bridge Design Specialist, or State Bridge Design Engineer, the request shall not be allowed unless otherwise approved through the RFP Technical Requirements.

15.6.1 Preliminary Girder Proportioning

Steel bridges shall be redundant, with three or more girders lines for I girders and two or more boxes for box girders, except as otherwise allowed by the RFP Technical Requirements.

15.6.2 Bridge Steels

AASHTO M 270 grade HPS 70W and HPS100W may only be used if allowed by the RFP Technical Requirements.

15.6.3 Fracture Critical Superstructures

Non-redundant, fracture critical single tub superstructures, and twin I-girder systems, may only be used if allowed by RFP Technical Requirements.

15.7 Substructure Design

15.7.1 General Substructure Considerations

15.7.1.A Foundation Seals

Requirements from Section 7.1.6 shall be followed.

15.7.1.B Scour

Requirements from Section 7.1.7 shall be followed. The hydraulic engineer of record replaces the Hydraulic Office where mentioned.

15.7.1.C Combination of Extreme Event Effects

15.7.1.C.1 *Downdrag*

Seismic soil liquefaction induced downdrag forces shall be included in the Extreme Event I limit state. Downdrag loads may be decoupled from the inertial and overstrength load effects.

15.7.1.C.2 *Lateral Ground Displacement*

Where lateral ground displacement (e.g. lateral spreading and lateral flow) is expected, the ground displacement may be decoupled from the inertial and overstrength load effects. See WSDOT *Geotechnical Design Manual* Sections 6.4.2.7 and 6.5.4 for additional guidance on combining loads when lateral ground displacement occurs.

15.7.1.C.3 *Scour*

The effects of local scour shall be combined with earthquake loading. At the Extreme Event I limit state, the design shall consider a scour depth equal to 50 percent of the total scour at scour design flood depth.

15.7.2 Foundation Modeling for Seismic Loads

15.7.2.A General

Bridge modeling for seismic events shall be in accordance with requirements of the LRFD-SGS Section 5.

If liquefaction is a design condition, the bridge shall be analyzed using both the static and liquefied soil conditions in accordance with LRFD-SGS Section 6.8.

15.7.2.B Bridge Model Section Properties

In general, gross section properties may be assumed for all members, except concrete columns and other ductile reinforced concrete members. Seismic response analysis for deep foundations shall be based on a bracketed approach using a stiff substructure response and a soft substructure response.

15.7.2.B.1 *Cracked Properties for Columns*

Effective section properties shall be in accordance with the LRFD-SGS Section 5.6.

15.7.2.B.2 *Shaft Properties*

The shaft concrete strength and construction methods lead to significant variation in shaft stiffness described as follows:

For a stiff substructure response:

1. Use f'_c to calculate the modulus of elasticity.
2. Use I_g based on the maximum oversized shaft diameter allowed by *Standard Specifications* Section 6-19.
3. When permanent casing is used, increase shaft I_g using the transformed area of the casing.

For a soft substructure response:

1. Use f'_c to calculate the modulus of elasticity.
2. Use I_g based on the nominal shaft diameter. Alternatively, Ie may be used when it is reflective of the actual load effects in the shaft.
3. When permanent casing is used, increase shaft I_g using the transformed area of the casing.

15.7.2.B.3 *Cast-in-Place Pile Properties*

For a stiff substructure response:

1. Use 1.5 f'_c to calculate the modulus of elasticity.
2. Use the pile I_g plus the transformed casing moment of inertia.

For a soft substructure response:

1. Use 1.0 f'_c to calculate the modulus of elasticity.
2. Use pile I_g, neglecting casing properties.

15.7.2.C Spread Footing Modeling Methods

The method for calculating footing springs is given in Section 7.2.7.

15.7.2.D Deep Foundation Modeling Methods

The method used to model deep foundations shall conform to LRFD-SGS Section 5.3.

15.7.2.D.1 *Group Effects*

The reduction factors for lateral resistance due to the interaction of deep foundation members is provided in GDM Section 8.12.2.3.

15.7.2.D.2 *Shaft Caps and Pile Footings*

In areas prone to scour or lateral spreading, the passive resistance of caps and pile-supported footings shall be neglected.

15.7.2.E Design of Deep Foundations for Lateral Forces

15.7.2.E.1 *Determination of Tip Elevations*

A parametric study or analysis shall be performed to evaluate the sensitivity of the depth of the shaft or pile to the ground level displacement of the structure in order to determine the depth required for stable, proportionate lateral response of the structure.

15.7.2.E.2 *Design for Lateral Loads*

The structural design of shafts and piles shall consider the following conditions at the applicable limit state:

1. Static soil properties with both stiff and soft shaft or pile properties.
2. Dynamic or degraded soil properties with both stiff and soft shaft or pile properties.
3. Liquefied soil properties with both stiff and soft shaft or pile properties. When lateral spreading is possible, additional loading conditions will need to be analyzed.
4. Scour condition with stiff and soft shaft or pile properties.

15.7.3 Column Design

15.7.3.A Shear Design

At Strength limit states, shear design shall follow the “Simplified Procedure for Nonprestressed Sections” in LRFD-BDS Section 5.7.3.4.1.

15.7.3.B Column Silos

Due to the construction and inspection complications of column silos, the Design-Builder shall attempt to meet balanced stiffness and frame geometry requirements by the other methods suggested in Section 4.1.4 of the LRFD-SGS prior to use of column silos. Column silos shall meet the requirements of Section 7.3.4.

15.7.3.C Longitudinal Reinforcement

The maximum reinforcement ratio shall be 0.04 in SDCs A, B, C and D. The minimum reinforcement ratio shall be 0.007 for SDC A, B, and C and shall be 0.01 for SDC D.

For bridges in SDC A, if oversized columns are used for architectural reasons, the minimum reinforcement ratio of the gross section may be reduced to 0.005, provided all loads can be carried on a reduced section with similar shape and the reinforcement ratio of the reduced section is equal to or greater than 0.01 and $0.133f'_c/f_y$. The column dimensions are to be reduced by the same ratio to obtain the similar shape.

The reinforcement shall be evenly distributed and symmetric within the column.

15.7.3.D Longitudinal Reinforcement Splices

No splices are allowed when the required length of longitudinal reinforcement is less than the conventional mill length of 60-feet. Splicing of longitudinal reinforcement shall be outside the plastic hinge regions. But in SDC A, splices need only be located a minimum of 1.5 times the column diameter from the top and bottom of the column.

For bridges in SDC A and SDC B, no lap splices shall be used for #14 or #18 bars (such splices shall be mechanical splices conforming to *Standard Specifications* Section 6-02.3(24)C). Either lap or mechanical splices may be used for #11 bars and smaller. Lap splices shall be detailed as Class B splices. The spacing of transverse reinforcement over the length of a lap splice shall not exceed 4-inches or one-quarter of the minimum member dimension.

For bridges in SDC C and SDC D, bars shall be spliced using mechanical splices conforming to *Standard Specifications* Section 6-02.3(24)F. Splices shall be staggered. The distance between splices of adjacent bars shall be greater than the maximum of 20-bar diameters or 24-inches.

15.7.3.E Longitudinal Reinforcement Development

15.7.3.E.1 *Crossbeams*

Development of longitudinal reinforcement shall be in accordance with LRFD-SGS Section 8.8.4. Column longitudinal reinforcement shall be extended into crossbeams as close as practicably possible to the opposite face of the crossbeam.

15.7.3.E.2 *Footings*

Longitudinal reinforcement at the bottom of a column should extend into the footing and rest on the bottom mat of footing reinforcement with standard 90 degree hooks. In addition, development of longitudinal reinforcement shall be in accordance with LRFD-SGS Section 8.8.4 and LRFD-BDS Section 5.10.8.2.1.

15.7.3.E.3 *Drilled Shafts*

Embedment shall be specified using TRAC Report WA-RD 417.1 titled "Noncontact Lap Splices in Bridge Column-Shaft Connections". The requirements of the LRFD-SGS Section 8.8.10 for development length of column bars extended into oversized pile shafts for SDC C and D shall not be used.

The modification factor in LRFD-BDS Section 5.10.8.2.1 that allows l_d to be decreased by the ratio of (A_s required)/(A_s provided), shall not be used.

15.7.3.F Transverse Reinforcement

15.7.3.F.1 *General*

All transverse reinforcement in columns shall be deformed. Columns in SDC A and B may use spirals, circular hoops, or rectangular hoops and crossties. Columns in SDC C and D shall use circular hoop reinforcement. However, rectangular hoops with ties may be used when large, odd shaped column sections are required.

15.7.3.F.2 *Spiral Splices and Hoops*

Welded laps shall be used for splicing and terminating spirals. Spirals or butt-welded hoops are required within plastic hinge regions. Splices shall be staggered. Also, where interlocking hoops are used in rectangular or non- circular columns, the splices shall be located in the column interior. Circular hoops for columns shall be shop fabricated using a manual direct butt weld or resistance butt weld. Field welded splices and termination welds of spirals of any size bar are not permitted in the plastic hinge region, including a zone extending 2'-0" into the connected member.

15.7.3.G Reduced Column Section

Columns with overstrength force reducing details shall be designed in accordance with Section 7.3.7.

15.7.4 Crossbeam

Two-stage integral non-prestressed crossbeams shall be designed in accordance with Section 7.4.1.

15.7.5 Abutment Design and Details

15.7.5.A General

15.7.5.A.1 *Bent-Type and Isolated Abutments*

Bent-type and isolated abutments shall be designed in accordance with Section 7.5.1.

15.7.5.A.2 *Abutments on Structural Earth (SE) Walls and Geosynthetic Walls*

Bridge abutments may be supported on structural earth walls and geosynthetic walls. Abutments supported on these walls shall be designed in accordance with the requirements of this RFP and the following documents (listed in order of importance):

- BDM Section 7.5.2
- WSDOT *Geotechnical Design Manual* Section 15.5.3.5
- LRFD-BDS
- *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*, Volume I and II, FHWA-NHI-10-024, FHWA-NHI-10-025

Walls directly supporting bridge abutment spread footings shall be 30 feet or less in total height, measured from the top of the fascia leveling pad to the bottom of the bridge abutment footing. Fall protection or security fencing shall be required at the top of the wall when the wall supports a spread footing abutment.

Structural Earth Walls shall follow the requirements in Section 15.8.1.E.

15.7.5.B Embankment and Backfill

15.7.5.B.1 *General Clearances*

The minimum clearances for the embankment at the front face of abutments shall be as indicated on Standard Plans A-50.10.00 through A-50.40.00.

The minimum clearance between the bottom of the superstructure and the embankment below shall be 3'-0" for girder bridges and 5'-0" for non-girder, slab, and box girder bridges.

15.7.5.B.2 *Abutments on SE Walls and Geosynthetic Walls*

Clearances around bridge abutments shall be provided as shown in Figure 7.5.2-3. Concrete slope protection shall be provided.

15.7.5.B.3 *Drainage and Backfill*

3″ diameter weep holes shall be provided in all bridge abutment walls. These shall be located 6″ above the finish ground line at 12′ on center. In cases where the vertical distance between the top of the footing and the finish groundline is greater than 10′, additional weep holes shall be provided 6″ above the top of the footing.

Gravel backfill for walls shall be provided behind all bridge abutments. A 3′ width of gravel backfill shall be provided behind the cantilever wing walls. An underdrain pipe and gravel backfill for drain shall be provided behind all bridge abutments except abutments on fills with a stem wall height of 5′ or less.

15.7.5.C Abutment Loading

15.7.5.C.1 *Earthquake Load*

EQ

For bearing pressure and abutment stability checks, the seismic inertial force of the abutment, P_{IR}, shall be combined with the seismic lateral earth pressure force, P_{AE}, as described in LRFD-BDS Section 11.6.5.1. For structural design of the abutment, the seismic inertial force, P_{IR}, shall be combined with the seismic lateral earth pressure force, P_{AE}, as described in LRFD-BDS Section 11.6.5.1 for stability checks.

15.7.5.C.2 *Bearing Forces*

TU

For strength design, the bearing shear forces shall be based on ½ of the annual temperature range.

15.7.5.D Abutment Details

15.7.5.D.1 *Bearing Seats*

The bearing seats shall have a minimum edge dimension of 3″ from the bearings shall and satisfy the requirements of LRFD-BDS Section 4.7.4.4. On L abutments, the bearing seat shall be sloped away from the bearings to prevent ponding at the bearings.

15.7.5.D.2 *Transverse Girder Stops*

All superstructures shall be restrained against lateral displacement at the abutments and intermediate expansion piers. All prestressed girder bridges in Western Washington (within and west of the Cascade mountain range) shall have girder stops between all girders at abutments and intermediate expansion piers. Girder stops shall be full width between girder flanges except to accommodate bearing replacement requirements as specified in Chapter 9. Girder stops are designed using a shear strength resistance factor shall be $\phi_s = 0.9$.

15.7.5.D.3 *Abutment Walls*

When construction joints are located in the middle of the abutment wall, a pour strip or an architectural reveal should be used for a clean appearance. LRFD-BDS Section 5.10.6 shall be followed for temperature and shrinkage reinforcement requirements near concrete surfaces exposed to daily temperature changes and in structural mass concrete. The minimum cross tie reinforcement in abutment walls shall be #4 tie bars with 135° hooks, spaced at approximately 2′-0″ center-to-center vertically and horizontally.

15.7.6 Abutment Wing Walls and Curtain Walls

Wall footing thickness shall not be less than 1’-6”

15.7.7 Footing Design

15.7.7.A General Footing Criteria

See Figure 7.7.1-1 for footing cover requirements. Footings supported on SE walls or geosynthetic walls shall have a minimum of 6″ of cover.

15.7.7.B Spread Footing Design

15.7.7.B.1 *Foundation Design*

1. Bearing Stress
   The maximum effective width for calculating uniform bearing stress is limited to $C+2D$ as shown in Figure 7.7.4-3.

15.7.7.B.2 *Structural Design*

1. **Footing Thickness and Shear**
   The minimum footing thickness shall be 1′-0″. The minimum plan dimension shall be 4′-0″.
2. **Vertical Reinforcement**
   Vertical reinforcement shall be developed into the footing to adequately transfer loads to the footing. Vertical rebar shall be bent 90° and extend to the top of the bottom mat of footing reinforcement. Bars in tension shall be developed using 1.25 L_d. Bars in compression shall develop a length of 1.25 L_d, prior to the bend. Where bars are not fully stressed, lengths may be reduced in proportion, but shall not be less than ¾ L_d.
3. **Bottom Reinforcement**
   Reinforcement shall not be less than #6 bars at 12″ centers to account for uneven soil conditions and shrinkage stresses.
4. **Top Reinforcement**
   Top reinforcement for footings designed for two-way action shall not be less than #6 bars at 12″ centers, in each direction while top reinforcement for bearing wall designed for one-way action shall not be less than #5 bars at 12″ centers in each direction.

15.7.7.C Pile-Supported Footing Design

The minimum footing thickness shall be 2′-0″. The minimum plan dimension shall be 4′-0.

15.7.7.C.1 *Pile Embedment, Clearance, and Rebar Mat Location*

Cast-in-place concrete piles with reinforcing extending into footings are embedded a minimum of 6″. The clearance for the bottom mat of footing reinforcement shall be 1½″ between the reinforcing and the top of the pile for CIP pile footings. See Figure 7.7.5-2 for the minimum pile clearance to the edge of footing.

15.7.7.C.2 *Concrete Design*

In determining the proportion of pile load to be used for calculation of shear stress on the footing, any pile with its center 6″ or more outside the critical section shall be taken as fully acting on that section. Any pile with its center 6″ or more inside the critical section shall be taken as not acting for that section. For locations in between, the pile load acting shall be proportioned between these two extremes.

15.7.8 Shafts

15.7.8.A Axial Resistance

Axial Resistance Group Reduction Factors

The group reduction factors for axial resistance of shafts for the strength and extreme event limit states shall be taken as shown in Table 7.8.1-1. These reduction factors presume that good shaft installation practices are used to minimize or eliminate the relaxation of the soil between shafts and caving. If this cannot be adequately controlled due to difficult soils conditions or for other constructability reasons, lower group reduction factors shall be used as recommended by the Geotechnical Engineer of record. These group reduction factors apply to both strength and extreme event limit states. For the service limit state the influence of the group on settlement as required in the LRFD-BDS and the LRFD-SGS are still applicable.

1. Shafts with permanent casing, installed by drilling and not driven, require reduced side resistance specified by the Geotechnical Engineer. Side resistance shall be limited to 10 percent of the nominal (ultimate) side resistance unless otherwise approved by the WSDOT State Geotechnical Engineer.

15.7.8.B Structural Design and Detailing

1. For shaft foundation supporting columns in SDC C or D, the shaft nominal moment capacity shall be designed to resist 1.25 times the moment demand generated in the shaft by the overstrength column plastic hinge moment at the base of the column.
2. Concrete Class 5000P shall be specified for the entire length of the shaft for wet or dry conditions of placement.
3. When shafts are constructed in water, the concrete specified for the casing shoring seal shall be Class 4000W.
4. The assumed concrete compressive strength may be taken as $0.85f'_c$ for structural design of shafts. For seismic design, the expected compressive strength may be increased by 1.3 in accordance with AASHTO Seismic Section 8.4.4.

5. The presence of permanent steel casing shall be taken into account in the shaft design (i.e. for stiffness, and etc.), but the structural capacity of permanent steel casing shall not be considered for structural design of drilled shafts unless the design conforms to Section 15.7.10.
6. Minimum cover requirements shall be as specified below:
   - Diameter less than or equal to 3′-0″ = 3″
   - Diameter greater than 3′-0″ and less than 5′-0″ = 4″
   - Diameter greater than or equal to 5′-0″ = 6″
7. The clear spacing between spirals and hoops shall not be less than 6″ or more than 9″. The clear spacing between spirals or hoops may be reduced in the splice zone in single column/single shaft connections if the concrete is vibrated.
8. The volumetric ratio and spacing requirements of the LRFD-SGS for confinement need not be met.
9. #7 through #9 welded lap spliced hoops are acceptable to use provided they are not located in possible plastic hinge regions. Welded splices in hoops for shafts shall be completed prior to assembly of the shaft steel reinforcing cage. When hoops are used, the plans shall show a staggered splice pattern around the perimeter of the shaft so that no two adjacent splices are located at the same location.
10. In single column/single shaft configurations, the spacing of the shaft transverse reinforcement in the splice zone shall meet the requirements of the TRAC Report titled, "*Noncontact Lap Splices in Bridge Column-Shaft Connections*". The factor *k* represents the ratio of column tensile reinforcement to total column reinforcement at the nominal resistance. In the upper half of the splice zone, *k* shall be taken as 1.0. In the lower half of the splice zone, this ration could be determined from a column moment-curvature analysis.
11. Longitudinal reinforcement shall be provided for the full length of drilled shafts. The minimum longitudinal reinforcement in the splice zone of single column/single shaft connections shall be the larger of 0.75 percent A_g of the shaft or 1.0 percent A_g of the attached column. The minimum longitudinal reinforcement beyond the splice zone shall be 0.75 percent A_g of the shaft. The minimum longitudinal reinforcement in shafts without single column/single shaft connections shall be 0.75 percent A_g of the shaft.
12. The clear spacing between longitudinal reinforcement shall not be less than 6″ or more than 9″. If a shaft design is unable to meet this minimum requirement, a larger diameter shaft shall be considered.
13. Mechanical splices in longitudinal bars shall be placed in low stress regions and staggered 2′-0″ minimum.
14. Where undersized permanent slip casing is used, provide a minimum of concrete cover of 3″ for shafts with a diameter of 4′-0″ and larger and 1½″ for shafts with a diameter less than 4′-0″.
15. Reinforcing bar centralizers shall be detailed in the plans as shown in Section 7.8.2-4.

15.7.9 Piles and Piling

15.7.9.A Pile Types

Piles for new permanent bridges shall be CIP concrete piles, precast, prestressed concrete piles, structural steel pipe piles, structural steel H piles, CFSTs, or RCFSTs. Precast, prestressed concrete piles shall only be used in SDC A or B. Steel H piles shall only be used at bridge abutments, and the connections into the cap shall develop the strength required for design and to prevent pull out during uplift.

15.7.9.B Pile Groups

The minimum center-to-center spacing of piles shall be 30″ or 2.5 pile diameters.

15.7.9.C Battered Piles

Battered piles shall not be used to resist lateral loads for permanent bridge foundations.

15.7.9.D Structural Design and Detailing of CIP Concrete Piles

1. Concrete Class 5000P shall be specified for CIP concrete piles. The top 10′ of concrete in the pile shall be vibrated.
2. For structural design, the reinforcement alone shall be designed to resist the total moment throughout the length of pile without considering strength of the steel casing. The minimum reinforcement shall be 0.75 percent A_g for SDC B, C and D and shall be provided for the full length of the pile. Minimum clearance between longitudinal bars shall meet the requirements in Appendix 5.1-A2.
3. If the pile to footing/cap connection is not a plastic hinge zone longitudinal reinforcement need only extend above the pile into the footing/cap a distance equal to 1.0 l_d (tension). If the pile to footing/cap connection is a plastic hinge zone longitudinal reinforcement shall extend above the pile into the footing/cap a distance equal to 1.25 l_d.
4. Transverse spiral reinforcement shall be designed to resist the maximum shear in the pile. The minimum spiral shall be a #4 bar at 9″ pitch. If the pile to footing/cap connection is not a plastic hinge zone the volumetric requirements of LRFD-BDS Section 5.11.4.5 need not be met.

15.7.9.E Structural Steel Pipe Piles

Structural steel pipe piles shall follow the current Special Provisions in addition to the requirement in the *Standard Specifications*. Additionally, the design wall thickness shall be reduced for corrosion over a 75-year minimum design life. Minimum corrosion rates are specified in Section 6.7.1.

15.7.9.F Pile Resistance

The bridge plans shall include the Ultimate Bearing Capacity (Nominal Driving Resistance, R_{ndr}, for driven piles) in tons as shown in Figure 7.9.11-1.

15.7.10 Concrete-Filled Steel Tubes

15.7.10.A Design Requirements

Concrete-filled steel tubes (CFST), reinforced concrete-filled steel tubes (RCFST) and their connections shall be designed in accordance with Section 7.10. The use of CFST and RCFST requires approval from the WSDOT Bridge Design Engineer when used as a ductile element as part of an earthquake-resisting system. Additionally, the plastic hinge modeling parameters and methods must be approved by the WSDOT Bridge Design Engineer.

15.8 Walls and Buried Structures

15.8.1 Retaining Walls

15.8.1.A General

Refer to Section 8.1.3 for design criteria.

15.8.1.B Loads

Refer to Section 8.1.3 for design criteria.

15.8.1.C Design of Reinforced Concrete Retaining Walls

Refer to Section 8.1.4 for design criteria.

15.8.1.D Design of Cantilever Soldier Pile and Soldier Pile Tieback Walls

15.8.1.D.1 *Ground Anchors (Tiebacks)*

Refer to Section 5.1.5.A for design criteria.

15.8.1.D.2 *Design of Soldier Pile*

Refer to Section 8.1.5.B for design criteria.

15.8.1.D.3 *Design of Lagging*

Refer to Section 8.1.5.C for design criteria.

15.8.1.D.4 *Design of Fascia Panels*

Refer to Section 8.1.5D for design criteria.

Use of shotcrete in lieu of cast-in-place conventional concrete for the soldier pile fascia shall require the approval of the WSDOT State Bridge and Structures Engineer.

15.8.1.E Design of Structural Earth Walls

15.8.1.E.1 *Pre-approved Proprietary Structural Earth Walls*

Refer to Section 8.1.6 for design criteria.

15.8.1.F Design of Standard Plan Geosynthetic Walls

Details for construction are given in the *Standard Plans* Section D.

15.8.1.G Design of Soil Nail Walls

Refer to Section 8.1.8 for design criteria.

15.8.1.H Scour of Retaining Walls

Refer to Section 8.1.10 for design criteria.

15.8.1.I Miscellaneous Items

Refer to Section 8.1.11 for design criteria.

15.8.2 Noise Barrier Walls

15.8.2.A General

Refer to Section 8.2.1 for design criteria.

15.8.2.B Loads

Refer to Section 8.2.2 for design criteria.

15.8.2.C Design

15.8.2.C.1 *Standard Plan Noise Barrier Walls*

Refer to Section 8.2.3.A for design criteria.

15.8.2.C.2 *Non-Standard Noise Barrier Walls*

Refer to Section 8.2.3.B for design criteria.

15.8.3 Buried Structures

15.8.3.A General Policy

Refer to Section 8.3.1 for design criteria.

If a Design-Builder intends to use alternate materials, other than reinforced concrete, composite concrete filled arch, or metal structural plate, they shall submit an Alternative Technical Concept.

15.8.3.B General Design Requirements

Refer to Section 8.3.3 for design criteria.

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

15.8.3.C Application of Loads

Refer to Section 8.3.3.B for design criteria.

Provisions for Vehicular Live Load shall be in accordance with Section 8.3.3.B.1.

Provisions for Vehicular Collision Force shall be in accordance with Section 8.3.3.B.2.

15.8.3.D Foundations and Scour

Refer to Section 8.3.3.D for design criteria.

15.8.3.E Corrosion

Refer to Section 8.3.4.E for design criteria.

15.8.3.F Worker, Pedestrian and Bicycle Fall Protection

Refer to Section 8.3.3.F for design criteria.

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

When Standard Plan C-20.41 or Standard Plan C-20.43 is attached to a Buried Structure, refer to Section 8.3.3.G for applicable design criteria.

15.8.3.H Headwalls and Wingwalls

Refer to Section 8.3.3.E for design criteria.

15.8.3.I Control of Cracking

Refer to Section 8.3.5.A.3 for design criteria.

15.8.3.J Joints

Joints shall be designed to carry the applied horizontal and vertical forces resulting from, but not limited to, differential settlement between segments, live load deflection, and shear transfer. Joints shall be so formed that they can be assembled to transmit those forces and provide joint tightness consistent with tolerances outlined in the Contract Documents. Each joint shall be sealed to prevent exfiltration or infiltration of soil fines and/or water.

15.8.3.K Deck Protection and Approach Slabs

Refer to Section 8.3.3.C for design criteria.

15.8.3.L Metal Structural Plate Structures

Refer to Section 8.3.6.C for design criteria.

15.8.3.M Design of Detention Vaults

Design of completely enclosed buried detention vaults shall not be permitted. Refer to Section 8.3.7 for design criteria specific to detention vaults.

15.8.3.N Design of Tunnels

Refer to Section 8.3.8 for design criteria specific to tunnels.

15.9 Bearings and Expansion Joints

15.9.1 Expansion Joints

15.9.1.A General Considerations

Bridges shall be designed to accommodate movements from all sources, including thermal fluctuations, concrete shrinkage, prestressing creep, and elastic post-tensioning shortening.

Where seismic isolation bearings are used, expansion joints shall be designed to accommodate seismic movements to allow the isolation bearings to function properly.

Expansion joints shall be designed to accommodate movement while minimizing imposition of secondary stresses in the structure. Expansion joint systems shall be sealed to prevent water, salt, and debris infiltration to substructure elements below. They shall also be designed to maximize durability while providing a relatively smooth riding surface.

Semi-integral construction shall be subject to the bridge length limitations stipulated below. In semi-integral construction, concrete end diaphragms are cast monolithically with the bridge deck. Girders are supported on elastomeric bearings, which are supported on a stub or cantilever abutment. Approach slab anchors, in conjunction with a compression seal, shall connect the monolithic end diaphragm to the bridge approach slab.

15.9.1.A.1 *Concrete Bridges*

Semi-integral construction shall be used for prestressed concrete girder bridges under 450 feet long and for post-tensioned spliced concrete girder and cast-in-place post-tensioned concrete box girder bridges under 400 feet long. Stub “L” and cantilever “L” type abutments with expansion joints at the bridge ends shall be used where bridge length exceeds these values.

15.9.1.A.2 *Steel Bridges*

“L” type abutments shall be used with expansion joints at the ends for multiple span bridges.

The use of intermediate expansion joints shall be avoided wherever possible.

For the purposes of this section, expansion joints are broadly classified into three categories based upon their total movement range as follows:

Small Movement Range Joints	Total Movement Range <= 1¾ inch
Medium Movement Range Joints	1¾ inch < Total Movement Range < 5 inch
Large Movement Range Joints	Total Movement Range >= 5 inch

15.9.1.B General Design Criteria

Expansion joints and bearings shall be designed interdependently and in conjunction with the anticipated behavior of the overall structure.

Shrinkage and uniform thermal variation movements shall be calculated as follows:

15.9.1.B.1 *Shrinkage Effects*

The shrinkage strain used for sizing expansion joints that are installed 30 to 60 days following concrete deck placement shall be no less than 0.0002. This value shall be corrected for restraint conditions imposed by various superstructure types as follows:

$$\Delta_{shrink} = \beta \times \mu \times L_{trib} \qquad (9.1.2\text{-}1)$$

Where:

- L_{trib} = Tributary length of the structure subject to shrinkage
- β = Ultimate shrinkage strain after expansion joint installation; estimated as 0.0002 in lieu of more refined calculations
- μ = Restraint factor accounting for the restraining effect imposed by superstructure elements installed before the concrete slab is cast 0.0 for steel girders, 0.5 for precast prestressed concrete girders, 0.8 for concrete box girders and T-beams, 1.0 for concrete flat slabs

15.9.1.B.2 *Thermal Effects*

Uniform thermal movement range shall be calculated using the maximum and minimum anticipated bridge superstructure average temperatures in accordance with LRFD-BDS Article 3.12.2.1 Procedure A. Most of western Washington shall be classified as a moderate climate. Eastern Washington and higher elevation areas of western Washington having more than 14 days per year with an average temperature below 32°F shall be classified as a cold climate. Factored thermal effects shall be calculated using the load factors stipulated in LRFD-BDS Article 3.4.

Total unfactored uniform thermal movement range shall be calculated as:

$$\Delta_{temp} = \alpha \times L_{trib} \times \delta T \qquad (9.1.2\text{-}2)$$

Where:

- L_{trib} = Tributary length of the structure subject to thermal variation
- α = Coefficient of thermal expansion; 0.000006 in./in./°F for concrete and 0.0000065 in./in./°F for steel
- δT Bridge superstructure average temperature range as a function of bridge type and climate as determined using AASHTO BDS Article 3.12.2.1 Procedure A

In accordance with *Standard Specifications*, contract drawings shall state dimensions at a normal temperature of 64°F unless specifically noted otherwise. Construction and fabrication activities at structure average temperatures other than 64°F require the Contractor or fabricator to adjust lengths of structural elements and concrete forms accordingly.

Strip seal and modular expansion joint systems are typically installed in preformed concrete blockouts after the bridge deck concrete has been cast. In these instances, concrete shall be placed in the blockout with the expansion joint device set at a gap that corresponds to the temperature of the already constructed bridge deck at the time concrete is placed in the blockout. To accomplish this, expansion device gap settings shall be specified on the contract drawings as a function of superstructure ambient average temperature. Generally, these settings shall be specified for temperatures of 40°F, 64°F, and 80°F.

15.9.1.C Small Movement Range Joints

Elastomeric compression seals shall be used for all small movement range applications for new bridges. Elastomeric compression seals or poured silicone sealant may be used for rehabilitation of existing small movement range expansion joints and widenings.

15.9.1.C.1 *Compression Seals*

Compression seals shall be designed and installed to effectively seal a joint against all water and debris infiltration. Compression seals shall extend continuously across the full roadway width and up into traffic barriers. No field splices of compression seals are allowed.

Compression seals shall be installed against smooth, straight vertical concrete faces. Concrete surfaces may be either formed or sawcut. Polyester or elastomeric concrete nosing material shall be used for rehabilitation of existing compression seal joints in accordance with Section 15.9.1.3C below.

For design purposes, the minimum and maximum working widths of the seal shall be 40 percent and 85 percent of the uncompressed width. These measurements are taken perpendicular to the joint axis. Compressed seal width at the normal construction temperature of 64°F may be taken as 60 percent of the seal's uncompressed width. For skewed joints, bridge deck movements shall be separated into components perpendicular and parallel to the joint axis. Shear displacement of the seal over the full expected temperature range shall be limited to 22 percent of its uncompressed width.

15.9.1.C.2 *Rapid-Cure Silicone Sealants*

Rapid-cure silicone sealants may be installed against either concrete or steel. Concrete or steel substrate surfaces shall be thoroughly cleaned before the sealant is installed.

Rapid-cure silicone sealants shall be designed and installed based upon the manufacturer's recommendations.

15.9.1.C.3 *Asphaltic Plug Joints*

Asphaltic plug joints are not allowed.

15.9.1.C.4 *Headers*

Expansion joint headers for new construction shall be the same class structural concrete as used for the bridge deck and shall be cast integrally with the deck.

Expansion joint headers installed as part of a rehabilitative and/or overlay project shall be either polyester concrete or elastomeric concrete. Expansion joint headers shall be in accordance with General Special Provisions in the RFP Appendix.

Concrete headers shall be constructed on each side of an expansion joint when an HMA overlay is installed atop an existing concrete bridge deck.

For bridge overlays, modified concrete overlay (MCO) material may provide rigid side support for an elastomeric compression seal or a rapid cure silicone sealant bead without the need for separately constructed elastomeric concrete or polyester concrete headers. Such modified concrete overlay headers may utilize welded wire fabric as reinforcement.

15.9.1.D Medium Movement Range Joints

15.9.1.D.1 *Steel Sliding Plate Joints*

Steel sliding plates shall be limited to the following specific applications:

1. sidewalks and crosswalks
2. modular expansion joint upturns at traffic barriers
3. bridge deck applications involving unusual movements (translation and large rotations) not readily accommodated by modular expansion joints.

All applications subject to pedestrian traffic shall meet ADA requirements and shall include a non-skid surface. Non-pedestrian traffic applications shall be galvanized or painted to provide corrosion resistance.

15.9.1.D.2 *Strip Seal Joints*

An elastomeric strip seal expansion joint shall consist of a preformed elastomeric gland mechanically locked into steel edge rails embedded into the concrete deck on each side of an expansion joint gap. Unfolding of the elastomeric seal accommodates movement. Edge rails shall be anchored to the concrete deck. The system shall be designed and detailed to accommodate the replacement of damaged or worn seals with minimal traffic disruption.

Either a standard anchorage or a special anchorage may be used for a strip seal expansion joint. The special anchorage incorporates steel reinforcement bar loops welded to intermittent steel plates, which in turn are welded to the steel shape. The special anchorage shall be used for very high traffic volumes or applications subject to snowplow hits. In applications highly susceptible to snowplow hits and concomitant damage, the intermittent steel plates shall be detailed to protrude ¼″ above the bridge deck surface to launch the snowplow blade and prevent it from catching on the forward extrusion.

The standard anchorage requires a minimum 7-inch-deep block out. The special anchorage requires a minimum 9-inch-deep block out.

The use of preformed silicone strip seal systems, in lieu of conventionally armored strip seal systems, shall be restricted as noted in Section 9.1.4.B.

15.9.1.D.3 *Bolt-down Panel Joints*

Bolt-down panel joints are not allowed.

On bridge overlay and expansion joint rehabilitation projects, existing bolt-down panel joints shall be replaced with rapid-cure silicone sealant joints or strip seal expansion joints.

15.9.1.E Large Movement Range Joints

15.9.1.E.1 *Steel Finger Joints*

Steel finger joints may only be used where modular expansion joints are incapable of accommodating the movements or are otherwise not feasible. Elastomeric or metal troughs shall be installed beneath steel finger joints to catch and redirect runoff water.

The steel fingers shall be designed to support traffic loads with sufficient stiffness to preclude excessive vibration. In addition to longitudinal movement, finger joints shall accommodate rotation and differential vertical deflection across the joint. Finger joints shall be fabricated with a slight downward taper toward the ends of the fingers in order to minimize potential for snowplow blade damage.

15.9.1.E.2 *Modular Expansion Joints*

Modular expansion joints shall provide watertight wheel load transfer across expansion joint openings. Modular expansion joints are generally shipped in a completely assembled configuration. Modular expansion joints longer than 40 feet. may be shipped in segments to accommodate construction staging and/or shipping constraints.

1. **Operational Characteristics**

   Modular expansion joints shall comprise a series of steel center beams oriented parallel to the expansion joint axis. Elastomeric strip seals or box-type seals shall attach to adjacent center beams, preventing infiltration of water and debris. The center beams shall be supported on support bars, which span in the primary direction of anticipated movement. The support bars shall be supported on sliding bearings mounted within support boxes. Polytetrafluoroethylene (PTFE) - stainless steel interfaces shall be used between elastomeric support bearings and support bars.

   Modular expansion joint systems shall meet the fatigue resistance characterization requirements specified in the Special Provision for modular expansion joints at time of contract award.

   Center beam field splices shall be carefully designed and constructed to mitigate fatigue susceptibility in accordance with the Special Provisions.

2. **Movement Design**

   Modular expansion joints shall be sized to accommodate 115 percent of calculated total movement range. Contemporary modular expansion joints permit approximately 3 inches of service movement per elastomeric seal element. Extreme event movement ranges of up to 5 inches per elastomeric seal element are allowed provided that support bars and support boxes are sized and detailed to accommodate the larger cumulative movement without structurally damaging the modular expansion joint or detaching any elastomeric strip seal elements. To minimize impact and wear on bearing elements, the maximum gap between adjacent center beams shall be limited to 3½ inch.

   To facilitate the installation of a modular joint at temperatures other than the 64°F normal temperature, the plans shall specify expansion gap distances face-to-face of edge beams as a function of the superstructure temperature at the time of installation.

3. **Review of Shop Drawings and Structural Design Calculations**

   Modular expansion joints shall be designed, tested, fabricated, QA\QC inspected, and installed in accordance with the General Special Provision in the RFP Appendix, including submittal of design calculations, fatigue testing results, weld procedures, and shop drawings.

   The expansion joint system shall be designed to ensure complete concrete consolidation underneath all support boxes. A minimum vertical clearance of 2 inches shall be provided between the bottom of each support box and the top of the concrete block out. Alternatively, when vertical clearance is minimal, grout pads may be placed underneath support boxes before casting the concrete within the blockout.

4. **Construction Considerations**

   Temperature adjustment devices shall be removed as soon as possible after concrete placement in the block out.

15.9.2 Bearings

15.9.2.A General Considerations

Bearings and expansion joints shall be designed interdependently and in conjunction with the anticipated behavior of the overall structure.

15.9.2.B Force Considerations

Bridge bearings shall be designed to transfer all anticipated loads from the superstructure to the substructure. Bearing design calculations shall be based upon the relevant load combinations and load factors stipulated in the LRFD-BDS. Impact need not be applied to live load forces in the design of bearings.

15.9.2.C Movement Considerations

The movement restrictions imposed by a bearing shall be compatible with the movements allowed by an adjacent expansion joint. Both bearings and expansion joints shall be designed consistent with the anticipated load and deformation behavior of the overall structure. Design rotations shall be calculated as follows:

15.9.2.C.1 *Elastomeric and Fabric Pad Bearings*

The maximum service limit state rotation for bearings that do not have the potential to achieve hard contact between metal components shall be taken as the sum of unfactored dead and live load rotations plus an allowance for fabrication and construction uncertainties of 0.005 radians.

15.9.2.C.2 *HLMR Bearings*

High-load multi-rotational (HLMR) bearings include spherical bearings, disc bearings, cylindrical bearings and pot bearings.

Both service and strength limit state rotations shall be used in the design of HLMR bearings. These rotations shall be shown on the plans to allow the manufacturer to properly design and detail a bearing.

Deformable elements such as polyether urethane discs and PTFE shall be designed for service limit state loads and rotations. The service limit state rotation shall include an allowance for uncertainties of ±0.005 radians.

The maximum strength limit state rotation shall be used to assure that potential hard contact (metal-to-metal or metal-to-concrete) is prevented. For disc bearings, the strength limit state rotation shall include an allowance of ±0.005 radians for uncertainties. For other HLMR bearings the strength limit state rotation shall include an allowance of ±0.005 radians for fabrication and installation tolerances and an additional allowance of ±0.005 radians for uncertainties in accordance with the LRFD-BDS.

15.9.2.D Detailing Considerations

HLMR bearings shall be designed, detailed, fabricated, and installed to facilitate inspection, maintenance, and eventual replacement. Jacking points shall be identified in the contract drawings so that bearings can be reset, repaired, or replaced.

Prestressed concrete girder bridges having end Type A (semi - integral) need not be detailed to accommodate elastomeric bearing replacement at abutments. Prestressed concrete girder bridges having end Type B (L-type abutments) shall be designed and detailed to accommodate elastomeric bearing replacement at abutments. Specifically, girder stops and end diaphragms shall be detailed to accommodate the placement of hydraulic jacks. The standard end diaphragms for long-span girders may not have sufficient flexural and shear capacity to support jacking induced stresses. Sufficient steel reinforcement shall be provided to accommodate shear forces and bending moments induced by jacking. (Girder end Types A and B are depicted Chapter 5) Intermediate piers of prestressed concrete girder bridges having steel reinforced elastomeric bearings shall also be designed and detailed to facilitate bearing replacement.

15.9.2.E Bearing Types

15.9.2.E.1 *Elastomeric Bearings*

Steel reinforced elastomeric bearings shall be designed using the LRFD-BDS Method B procedure. Shear modulus shall be specified on the plans as 165 psi at 73˚ F without reference to durometer hardness.

Elastomeric bearings shall conform to the requirements of AASHTO M 251M/M - *Plain and Laminated Elastomeric Bridge Bearings*. Shims shall be fabricated from ASTM A 1011 Grade 36 steel unless noted otherwise on the plans. Bearings shall be laminated in ½-inch thick elastomeric layers with a minimum total thickness of 1 inch. For overall bearing heights less than 5 inches, a minimum of ¼ inch of side clearance shall be provided over the steel shims. For overall heights greater than 5 inches, a minimum of ½-inch of side clearance shall be provided. Live load compressive deflection shall be limited to ¹⁄₁₆-inch. Compressive dead load and live load shall be specified on the plans.

With respect to width, elastomeric bearings shall be designed and detailed as follows:

1. For all wide flange prestressed concrete girders, the edge of the bearing pad shall be set between 1 inch minimum and 9 inches maximum inside of the edge of the girder bottom flange.
2. For prestressed concrete I-girders, bulb-tee girders, and deck bulb-tee girders, the edge of the bearing pad shall be set 1 inch inside of the edge of the girder bottom flange.
3. For all prestressed concrete tub girders, the edge of the bearing shall be set 1 inch inside of the edge of the bottom flange. Bearing pads for prestressed concrete tub girders shall be centered close to the centerline of each web.
4. For all prestressed concrete slabs one bearing pad and corresponding grout pad is required for each end of the prestressed concrete slab. The centerline of the bearing and grout pad shall coincide with the centerline of the prestressed concrete slab. The need for steel shims shall be assessed during the bearing design.

In order to facilitate compressive load testing, future bearing replacement, and vertical geometry coordination, the following table shall be included in the Plans:

Bearing Design Table	
Service I Limit State	
Dead load reaction	--------- kips
Live load reaction (w/o impact)	--------- kips
Unloaded height	--------- inches
Loaded height (*DL*)	--------- inches
Shear modulus at 73° F	--------- psi

In the construction of precast prestressed concrete girder and steel girder bridges, elastomeric bearings need not be offset to account for temperature variation during erection of the girders. Girders may be set atop elastomeric bearings at temperatures other than the mean of the temperature range. This shall be statistically reconciled by assuming a maximum thermal movement in either direction of:

$$\Delta_{temp} = 0.75 \cdot \alpha \cdot L \cdot (T_{MaxDesign} - T_{MinDesign})$$

where $T_{MaxDesign}$ is the maximum anticipated superstructure average temperature and $T_{MinDesign}$ is the minimum anticipated superstructure average temperature during the life of the bridge.

For precast prestressed concrete girder bridges, the maximum thermal movement, Δ_{temp}, shall be added to shrinkage and long-term creep movements to determine total bearing height required. The shrinkage movement for this bridge type shall be half that calculated for a cast-in-place concrete bridge.

15.9.2.E.2 *Fabric Pad Sliding Bearings*

Fabric pad sliding bearings incorporate fabric pads with a polytetrafluoroethylene (PTFE) - stainless steel sliding interface to permit large translational movements. Fabric pads shall conform to the requirements of AASHOT Specification M 351 - *Cotton Duck Fabric Bridge Bearings*.

Unfilled PTFE shall be used for fabric pad sliding bearings.

Unfilled PTFE shall be recessed half its depth into a steel backing plate, which shall be bonded to the top of a fabric pad. The stainless steel sheet shall be seal welded to a steel sole plate attached to the superstructure.

1. **Fabric Pad Design**
   Maximum service load average bearing pressure for fabric pad bearing design shall be 1,200 psi. Maximum service load edge bearing pressure for fabric pad bearing design shall be 2,000 psi.

2. **PTFE – Stainless Steel Sliding Surface Design**
   PTFE having a maximum dimension less than or equal to 24 inches shall be at least 3⁄16-inch thick and recessed 3⁄32-inch into a minimum ½-inch thick steel plate that is bonded to the top of the fabric pad. PTFE having a maximum dimension greater than 24 inches shall be at least ¼ inch thick and shall be recessed ⅛-inch into a ½-inch thick steel plate that is bonded to the top of the fabric pad.

   Stainless steel sheet shall be finished to a No. 8 (Mirror) finish and shall be seal welded to the sole plate.

15.9.2.E.3 *Pin Bearings*

Steel pin bearings may be used to support heavy reactions with moderate to high levels of rotation about a single predetermined axis.

15.9.2.E.4 *Rocker and Roller Type Bearings*

Rocker bearings and steel roller bearings are not allowed for new bridges.

15.9.2.E.5 *Spherical Bearings*

Woven fabric PTFE shall be used on the curved surfaces of spherical bearings. When spherical bearings are detailed to accommodate translational movement, woven fabric PTFE shall be used on the flat sliding surface also. Woven fabric PTFE, which is mechanically interlocked over a metallic substrate, shall have a minimum thickness of 1⁄16-inch and a maximum thickness of ⅛-inch over the highest point of the substrate.

Spherical bearings shall be detailed with the concave surface oriented downward. Structural analysis of the overall structure shall recognize the center of rotation of the bearing not being coincident with the neutral axis of the girder above.

The contract drawings shall show the diameter and height of the spherical bearing in addition to all dead, live, and seismic loadings. Sole plate connections, base plate, anchor bolts, and any appurtenances for horizontal force transfer shall be detailed on the plans. The spherical bearing manufacturer shall submit shop drawings and detailed structural design calculations of spherical bearing components for review and comment by WSDOT.

15.9.2.E.6 *Disc Bearings*

Disc bearings composed of an annular shaped polyether urethane disk with a steel shear-resisting pin in the center may be used. A flat PTFE - stainless steel surface may be incorporated into the bearing to also provide translational movement capability.

15.9.2.E.7 *Seismic Isolation Bearings*

Seismic isolation bearings shall be designed in accordance with the requirements of the LRFD-BDS, LRFD-SGS and AASHTO Guide Specifications for Seismic Isolation Design and shall conform to the requirements in Section 9.3. Where Section 9.3 requires approval from the Bridge Design Office, Bridge Design Specialist, or State Bridge Design Engineer, the request shall not be allowed unless otherwise approved through the RFP Technical Requirements.

15.9.2.F Miscellaneous Details

15.9.2.F.1 *Temporary Support before Grouting Masonry Plate*

The masonry plate of an HLMR bearing shall be supported on a grout pad that is installed after the bearing and superstructure girders above have been erected. This sequence allows the Contractor to level and slightly adjust the horizontal location of the bearing before immobilizing it. Two methods for temporarily supporting the masonry plate are acceptable:

1. **Shim Packs**

   Multiple stacks of steel shim plates may be placed atop the concrete surface to temporarily support the weight of the girders on their bearings before grouting.

2. **Two-step Grouting with Cast Sleeves**

   A two-step grouting procedure with cast-in-place voided cores may be used for smaller HLMRs not generally subjected to uplift. Steel studs are welded to the underside of the masonry plate to coincide with the voided cores. With temporary shims installed between the top of the concrete surface and the underside of the masonry plate, the voided cores are fully grouted. Once the first stage grout has attained strength, the shims are removed, the masonry plate is dammed, and grout is placed between the top of the concrete surface and the underside of the masonry plate.

15.9.2.F.2 *Anchor Bolts*

Anchor bolts shall be designed to resist all horizontal shear forces and direct tension force due to uplift.

Anchor bolts shall be ASTM A 449 where strengths equal to ASTM A 325 are required and ASTM A 354, Grade BD, where strengths equal to ASTM A 490 are required. Anchor bolts shall be ASTM F 1554 bolts with supplemental Charpy test requirements in applications in which the bolts are subject to seismic loading.

15.9.2.G Contract Drawing Representation

High load multi-rotational bearings shall be depicted schematically in the contract drawings. Each bearing manufacturer has unique fabricating methods and procedures that allow it to fabricate a bearing most economically. Depicting the bearings schematically with loads and geometric requirements provides each manufacturer the flexibility to innovatively achieve optimal economy.

15.9.2.H Shop Drawing Review

High-load multi-rotational bearings shall be designed, tested, fabricated, QA/QC inspected, and installed in accordance with the Special Provisions in the RFP Appendix, including submittal of design calculations and shop drawings.

15.9.2.I Bearing Replacement Considerations

Bearings shall be designed and detailed to permit the replacement of all elements subject to wear. Superstructure and substructure elements shall be designed and detailed to accommodate lifting of the superstructure using hydraulic jacks to facilitate bearing element replacement.

For bearing replacements, the Design-Builder shall show anticipated lifting loads on the contract drawings. Limitations on lift height shall also be specified. Consideration shall be given to lift height as it relates to adjacent expansion joints elements and adjoining sections of railing. Restrictions on differential lift height between multiple jacks shall be specified to minimize stresses induced in adjacent structural elements.

Jacks shall be sized for 200 percent of the calculated lifting load.

15.10 Signs, Barriers, Bridge Approach Slabs, and Utilities

15.10.1 Sign and Luminaire Supports

The provisions of Section 10.1 are applicable and shall be required.

15.10.2 Bridge Traffic Barriers

15.10.2.A General Guidelines and Policy

The provisions of Section 10.2 are applicable and shall be required.

15.10.3 At Grade Concrete Barriers

The provisions of Section 10.3 are applicable and shall be required.

15.10.3.A Moment Slab Traffic Barrier Design

15.10.3.A.1 *Structural Capacity*

The structural capacity of the moment slab traffic barrier shall be designed for the required TL impact forces in accordance with LRFD-BDS Chapters 5 and 13 as modified by Section 10.2.4.A. The minimum Test Level shall be TL-4.

Any section along the moment slab shall not fail in shear, bending, or torsion when the barrier is subjected to the TL impact forces.

The moment slab reinforcement shall be designed to resist forces developed at the base of the barrier. Moment slab supporting concrete barrier shall be designed in accordance to Deck Overhang Design in accordance with LRFD-BDS Section A13.4 as modified by Section 10.2.4.A.

The torsion capacity of the moment slab shall be equal to or greater than the traffic barrier moment generated by the TL impact forces.

15.10.3.A.2 *Global Stability*

See Section 10.3.2.B.2 through Section 10.3.2.B.5.

15.10.3.A.3 *Geometry*

The minimum height of the traffic barrier portion of the moment slab shall be 42 inches above the finished roadway surface.

Moment slabs shall have a minimum width of 4.0 feet measured from the point of rotation to the heel of the slab and a minimum average depth of 0.83 feet.

15.10.3.A.4 *Soil Reinforcement*

Design of the soil reinforcement shall be in accordance with the *Geotechnical Design Manual* Chapter 15.

15.10.3.A.5 *Wall Panel*

The wall panels shall be designed to resist the dynamic pressure distributions as defined in the *Geotechnical Design Manual* Chapter 15.

The wall panel shall have sufficient structural capacity to resist the *maximum design rupture load for the wall reinforcement designed in accordance with the Geotechnical Design Manual* Chapter 15.

15.10.3.B Precast Concrete Barrier

Concrete barrier Type 2 and Type 4 shall be used in accordance to Section 10.3.4.

15.10.4 Bridge Traffic Barrier Rehabilitation

15.10.4.A General Guidelines and Policy

When identified in the RFP, deficient rails shall be improved or replaced within the limits of roadway resurfacing projects in accordance to Section 10.4.

Retrofit shall be an approved crash tested rail system or shall be designed to the strength requirements set forth by Section 2 of AASHTO *Standard Specifications for Highway Bridges*, 17^{th} edition.

See Section 10.4.4 and WSDOT *Design Manual* for replacement criteria.

See Section 10.4.5 and 10.4.6 for available bridge rail retrofit and bridge rail replacement designs.

This section is applicable to modifications of structures and structural components in the barrier load path associated with barrier retrofit, rehabilitation, modification, repair, and/or replacements other than complete replacements for structures within the applicable timeframe, those constructed prior to the year 2000.

15.10.4.A.1 *Design Criteria*

1. **Structural Capacity**

   A strength and geometric review shall be required for all bridge rail rehabilitation projects. The AASHTO LFD load of 10 kips shall be used in the retrofit of existing traffic barrier systems constructed prior to the year 2000. No load less than 10 kips shall be applied, and the strength of the system shall be assessed in the pre and post retrofit condition. No reduction in structural capacity of greater than or equal to 10% is permitted.

   If the strength of the existing bridge rail and their supporting elements (i.e. deck) are unable to resist a 10 kip barrier impact design load or has not been crash tested, then modifications or replacement will be required to improve its redirectional characteristics and strength.

   If the design of the bridge rehabilitation includes other bridge components that will be designed using LRFD-BDS then the following minimum equivalent Extreme Event (CT) traffic barrier loading can be used limited to a maximum of <10% reduction:

   Flexure = (1.3)*(1.67)*(10 kip) / (0.9) = 24.10 kip

   Shear = (1.3)*(1.67)*(10 kip) / (0.85) = 25.54 kip

2. **Geometry**

   Standard thrie beam guardrail post spacing is 6′-3″ except for the SL-1 Weak Post, which is at 8′-4″. Post spacing can be increased up to 10′-0″ if the thrie beam guardrail is nested (doubled up).

   Guardrail shall be continuous without gaps.

   Design F guardrail end sections shall be used at the approach and trailing end of these gaps.

   Standard Plan thrie beam guardrail transitions shall be used at each corner of the bridge.

   Placement of the retrofit system will be determined from the WSDOT *Design Manual*.

15.10.5 Bridge Railing

15.10.5.A General Guidelines and Policy

Pedestrian and bicycle/pedestrian railings shall be designed in accordance with LRFD-BDS Chapter 13 with the following supplemental guidelines.

Railings shall be designed for vehicular impact load or be successfully crash tested unless location is low speed, location is outside of Design Clear Zone as defined in *Design Manual* Chapter 1600, or location has minimal safety consequence from collapse of railing.

Minimum height of 54″ shall be provided for bicycle railings on structures.

Fall Protection railing shall meet the requirements of WAC 296-880 and as described in the *Design Manual* Chapter 1060.

See Section 10.5.2 for available bridge railing designs.

15.10.6 Bridge Approach Slabs

Bridge approach slabs are required for the following structures:

- New bridges
- Widened bridges (full roadway width)
- Class 1 and Class 2 Buried Structures without a full Roadways section (including HMA and CSBC) within 25 feet of each end of the Buried Structure

Bridge runoff at the abutments shall be carried off and collected at least 10 feet beyond the bridge approach slab.

15.10.6.A Bridge Approach Slab Design Criteria

The standard bridge approach slab design is based on the following criteria:

1. The bridge approach slab is designed as a slab in accordance with LRFD-BDS. (Strength Limit State, IM = 1.33, no skew).
2. The support at the roadway end is assumed to be a uniform soil reaction with a bearing length that is approximately ⅓ the length of the approach slab, or 25′/3 = 8′.
3. The Effective Span Length (Seff), regardless of approach length, is assumed to be:

   25′ approach – 8′ = 17′.

4. Longitudinal reinforcing bars do not require modification for skewed approaches up to 30 degrees or for slab lengths greater than 25′.
5. The bridge approach slab is designed with a 2″ concrete cover to the bottom reinforcing.

15.10.6.B Bridge Approach Slab Detailing

The minimum dimension from the bridge is 25′.

AS1 bars shall be epoxy coated. Bending diagrams shall be shown for all custom reinforcement. All Bridge Approach Slab sheets will have the AP2 and AP7 bars. If there is a traffic barrier, then AP8, AS1, and AS2 bars shall be shown.

Longitudinal contraction joints are required on bridge approach slabs wider than 40 feet or where steps are used on skewed alignments. Joints shall be located at lane lines or median barrier and in accordance with Bridge Standard Drawing 10.6-A1-2.

15.10.6.C Skewed Bridge Approach Slabs

For all skewed abutments, the roadway end of the bridge approach slab shall be normal to the roadway centerline. Skews greater than 20-degrees require analysis to verify the bottom mat reinforcement, and may require expansion joint modifications.

The roadway end of the approach may be stepped to reduce the size or to accommodate staging construction widths. At no point shall the roadway end of the approach slab be closer than 25′ to the bridge. These criteria apply to both new and existing bridge approach slabs. If stepped, the design shall provide the absolute minimum number of steps and the longitudinal construction joint shall be located on a lane line. See Figure 10.6.4-1 for clarification.

In addition, for bridges with traffic barriers and skews greater than 20 degrees, the AP8 bars shall be rotated in the acute corners of the bridge approach slabs. Typical placement is shown in the flared corner steel detail, see Figure 10.6.4-2.

15.10.6.D Approach Anchors and Expansion Joints

For semi-integral abutments or stub abutments, the joint design shall be checked to ensure the available movement of the standard joint is not exceeded. For bridge approach slabs with barrier, the compression seal shall extend into the barrier.

L Type Abutments

Use a pinned connection in accordance with Section 10.6.5.

15.10.6.E Bridge Approach Slab Addition or Retrofit to Existing Bridges

Bridge approach slabs on existing bridges shall be pinned to the existing pavement seat, or attached with approach anchors.

The pinning option is only allowed on semi-integral abutments as a bridge approach slab addition or retrofit to an existing bridge. Figure 10.6.6-1 shows the pinning detail. As this detail eliminates the expansion joint between the bridge approach slab and the bridge, the maximum bridge superstructure length is limited to 150′. Additionally, if the roadway end of the bridge approach slab is adjacent to PCCP roadway, then the detail shown in Figure 10.6.6-2 applies. PCCP does not allow for as much movement as HMA and a joint is required to reduce the possibility of buckling.

When pinning is not applicable, then the bridge approach slab shall be attached to the bridge with approach anchors. If the existing pavement seat is less than 10 inches, the seat shall be modified to provide at least 10 inches of seat width.

When a bridge approach slab is added to an existing bridge, the final grade of the bridge approach slab concrete shall match the existing grade of the concrete bridge deck, including bridges with asphalt pavement. The existing depth of asphalt on the bridge shall be shown in the Plans and an equal depth of asphalt placed on a new bridge approach slab. If the existing depth of asphalt is increased or decreased, the final grade shall also be shown on the Plans.

15.10.6.F Bridge Approach Slab Staging

Ensure staging follows traffic control.

Add mechanical splice option as shown in Figure 10.6.6-3 when needed.

15.10.7 Traffic Barrier on Bridge Approach Slabs

A gap between the bridge approach slab and wingwall (or retaining wall) shall be shown in the details. The minimum gap is twice the long-term settlement, or 2 inches as shown in Figure 10.7-1.

When the traffic barrier is placed on the bridge approach slab,

- Barrier shall extend to the end of the bridge approach slab
- Conduit deflection or expansion fittings shall be called out at the joints
- Junction box locations shall start and end in the approach
- The transverse top reinforcing in the slab shall be sufficient to resist a traffic barrier impact load.

15.10.7.A Bridge Approach Slab over Wing Walls, Cantilever Walls or Geosynthetic Walls

All walls that are cast-in-place below the bridge approach slab shall continue the barrier soffit line to grade as shown in Figure 10.7.1-1.

15.10.7.B Bridge Approach Slab over SE Walls

The barrier soffit line shall match that for the SEW barrier starting at the bridge expansion joint as shown in Figures 10.7.2-1 and 10.7.2-2.

15.10.8 Utilities Installation on New and Existing Structures

15.10.8.A General Concepts

The utilities included under this section are those described in *Standard Specifications* Section 6-01.10. Bridge plans shall include all hardware specifications and details for the utility attachment as described in the RFP.

15.10.8.A.1 *Coating and Corrosion Protection*

When the bridge is to receive pigmented sealer, any exposed utility lines and hangers shall be painted to match the bridge. When a pigmented sealer is not required, steel utility conduits and hangers shall be painted or galvanized for corrosion protection. The RFP Criteria shall specify cleaning and painting procedures.

15.10.8.B Utility Design Criteria

All utilities shall be designed to resist Strength and Extreme Event Limits States. Utility support design calculations shall be stamped with a State of Washington Professional Engineer stamp, signed and dated.

Positive resistance to loads shall be provided in all directions perpendicular to and along the length of the utility as required by the utility engineer.

Dynamic fluid action due to loads shall be resisted off the bridge.

Where utilities are insulated, the insulation system shall be designed to allow the intended motion range of the hardware supporting the utility.

Conduit shall be rigid.

15.10.8.B.1 *Utility Location*

Utilities shall be located, such that a failure will not result in damage to the bridge, the surrounding area, or be a hazard to traffic. The utility shall be installed between girders. Utilities and supports shall not extend below the bottom of the superstructure. Utilities shall be installed no lower than 1 foot 0 inches above the bottom of the girders. Utilities shall not be attached above the bridge deck nor attached to the railings or posts.

15.10.8.B.2 *Termination at the Bridge Ends*

Utility conduit and encasements shall extend 10 feet minimum beyond the ends of the structure. Utilities off the bridge shall be installed prior to paving of approaches.

15.10.8.B.3 *Utility Expansion*

The utilities shall be designed with a suitable expansion system as required to prevent longitudinal forces from being transferred to bridge members.

15.10.8.B.4 *Utility Blockouts*

Blockouts shall be provided in all structural members that prohibit the passage of utilities, such as girder end diaphragms, pier crossbeams, and intermediate diaphragms. These blockouts shall be large enough to fit deflection fittings, and shall be parallel to the utility. For multiple utilities, a note shall be added to the plans that the deflection fittings shall be staggered such that no fitting is located adjacent to another, or the blockouts shall be designed to fit both fittings. Expansion fittings shall be staggered.

15.10.8.B.5 *Gas Lines or Volatile Fluids*

Pipelines carrying volatile fluids through a bridge superstructure shall be designed in accordance with WAC 480-93, *Gas Companies—Safety, and Minimum Federal Safety Standard*, Title 49 Code of Federal Regulations (CFR) Section part 192. WAC 468-34-210, *Pipelines - Encasement*, describes when casing is required for carrying volatile fluids across structures.

15.10.8.B.6 *Water Lines*

Transverse support or bracing shall be provided for all water lines to carry Strength and Extreme Event Lateral Loading. In box girders (closed cell), a rupture of a water line will generally flood a cell before emergency response can shut down the water main. This shall be designed for as an Extreme Event II load case, where the weight of water is a dead load (DC). Additional weep holes or open grating, or full length casing extending 10-feet beyond the end of the bridge approach slab shall be used to offset this Extreme Event (see Figure 10.8.3-1).

15.10.8.B.7 *Sewer Lines*

Sewer lines shall meet the same design criteria as waterlines. Encasement pipe is required for sewer lines on bridges that cross over water or roads.

15.10.8.B.8 *Electrical (Power and Communications)*

Telephone, television cable, and power conduit shall be galvanized Rigid Metal Conduit (RGS) or Rigid Polyvinyl Chloride Conduit (PVC). Where such conduit is buried in concrete curbs or barriers or has continuous support, such support is considered to be adequate. Where hangers or brackets support conduit at intervals, the maximum distance between supports shall be in accordance with Section 10.8.6.

15.10.8.C Box/Tub Girder Bridges

Utilities shall be located between girders or under the bridge deck soffit when the reinforced concrete box or tub girders are less than 4 feet inside clear height.

Special utilities (such as water or gas mains) in box girder bridges shall use concrete pedestals. Continuous supports shall not be used.

15.10.8.D Traffic Barrier Conduit

All new bridge construction shall install two 2-inch galvanized Rigid Metal Conduit (RGS) or Rigid Polyvinyl Chloride Conduit (PVC) in the traffic barriers. PVC conduit may be used only in stationary-form barriers, and will connect to RGS using a PVC adaptor when exiting the barrier. RGS conduit may be used in stationary-form barriers, but it shall be used in slipform barriers.

Each conduit shall be stubbed-out into its own concrete junction box at each corner of the bridge.

The galvanized steel conduit shall be wrapped with corrosion resistant tape at least one foot inside and outside of the concrete structure, and this requirement shall be so stated on the plans. The corrosion resistant tape shall be 3M Scotch 50, Bishop 5, Nashua AVI 10, or approved equal.

Pull boxes shall be provided within the barrier for each conduit at each end of the bridge and at a maximum spacing of 200 feet. For fiber optics only, spacing shall not exceed 360 feet. The pull box size shall conform to the specifications of the National Electric Code or be a minimum of 8 inches by 8 inches by 18 inches to facilitate pulling of wires. Galvanized steel pull boxes (or junctions boxes) shall meet the specifications of the "NEMA Type 4X" standard for stationary-form barrier, shall meet the specifications of the "NEMA 3R" and be adjustable in depth for slip form barrier, and the NEMA junction box type shall be stated on the plans. Stainless steel pull boxes may be used as an option to the galvanized steel.

In the case of existing bridges, an area 2 feet in width shall be reserved for conduit beginning at a point either 4 feet or 6 feet outside the face of usable shoulder.

15.10.8.E Conduit Types

All electrical conduits shall be galvanized Rigid Metal Conduit (RGS) or Rigid Polyvinyl Chloride Conduit (PVC).

15.10.8.F Utility Supports

All utility installations shall address temperature expansion in the design of the system or expansion devices.

Utility supports shall be designed so that any loads imposed by the utility installation do not overstress the conduit, supports, bridge structure, or bridge members.

Designs shall provide longitudinal and transverse support for loads from gravity, earthquakes, temperature, inertia, etc.

Vertical supports shall be spaced at 5 foot maximum intervals for telephone and power conduits, and at a spacing to resist design loads for all other utilities. For Schedule 40 steel conduit, 4" or greater, support spacing may be increased to 8 feet maximum if the design loads permit.

Drilling into prestressed concrete members for utility attachment shall not be allowed.

15.10.8.F.1 *Pipe Hangers*

For heavy pipes over traffic (10″ water main or larger), a Safety Factor of 1.5 shall be used to resist vertical loads for Strength design.

The cast-in-place insert shall be at least 6″ long and hot dipped galvanized in accordance with AASHTO M 111 or AASHTO M 232.

The insert shall not interfere with reinforcement in the bridge deck. The inserts shall be installed level longitudinally and transversely.

Transverse supports shall, at a minimum, be located at every other vertical support. Bridge Standard Drawings 10.8-A1-1 and 10.8-A1-2 depict typical utility support installations and placement at abutments and diaphragms.

15.10.8.F.2 *Surface Mounting*

Utilities to be installed on existing structures that cannot be located between girders may be mounted under the deck soffit. Adhesive anchor shall be design in accordance with Section 10.10.

Bridge Standard Drawing 10.8-A1-3 shows typical mounting locations for concrete beam of box girder bridges. Anchors shall be located 3" minimum from the edge of deck or other concrete surfaces.

15.10.9 Review Procedure for Utility Installations on Existing Structures

Utility installations on existing bridges shall be reviewed in accordance with Section 10.9.

15.10.10 Anchors for Permanent Attachments

The design procedure for cast-in-place and post-installed anchors shall be in accordance with LRFD-BDS 5.13. Adhesive and undercut anchors shall meet the assessment criteria in accordance with ACI 355.4 and ACI 355.2, respectively.

Fast set epoxy anchors shall not be used for adhesive anchors.

Undercut anchors shall be stainless steel.

15.10.11 Drainage Design

Drainage design shall meet the requirements in Section 10.11, except as otherwise noted herein.

15.11 Detailing Practices

Structural detailing and deliverable requirements shall be done in accordance with Chapter 11 of the *Bridge Design Manual* (LRFD). Anything not covered in Chapter 11 shall be done in accordance with the WSDOT *Electronic Engineering Data Standards* (EEDS) Division 3 - Symbology, Section 5 PS&E Standards as well as the *Plans Preparation Manual*.

15.12 Bridge Load Rating

15.12.1 General

Follow the requirements in Chapter 13. Final approval of the load rating of a bridge shall rest with WSDOT's Load Rating Engineer.

15.13 Appendices

Appendix 15.2-A1 Conceptual Plan Checklist

Project _______________ SR ______ Concept Plan by ___________ Check by _____ Date______

Plan

_____ Survey Lines and Station Ticks
_____ Survey Line Bearings
_____ Roadway and Median Widths
_____ Bridge Deck Lane and Shoulder Widths
_____ Bridge Deck Sidewalk Width
_____ Profile Grade and Pivot Point
_____ Roadway Superelevation Rate (if constant)
_____ Traffic Arrows
_____ Back to Back of Pavement Seats
_____ Span Lengths
_____ Existing utilities Type, Size, and Location
_____ Stream Flow Arrow
_____ R/W Lines and/or Easement Lines
_____ Points of Minimum Vertical Clearance
_____ Exist . Bridge No . (to be removed, widened)
_____ Section, Township, Range
_____ City or Town
_____ North Arrow
_____ SR Number
_____ Bearing of Piers, or note if radial

Typical Section

_____ Bridge Roadway Width
_____ Lane and Shoulder Widths
_____ Profile Grade and Pivot Point
_____ Superelevation Rate
_____ Survey Line
_____ BP/Pedestrian Rail dimensions
_____ Assumed Structure Depth/Prestressed Girder Type

Miscellaneous

_____ Assumed Structure Type
_____ Live Loading
_____ Undercrossing Alignment Profiles/Elevs .
_____ Bridge Deck Superelevation Diagrams
_____ Bridge Deck Alignment Curve Data
_____ Names

Elevation

_____ Full Length Reference Elevation Line
_____ Existing Ground Line x ft . Rt of Survey Line
_____ Pier Stations
_____ Profile Grade Vertical Curves
_____ BP/Pedestrian Rail
_____ Minimum Vertical Clearances
_____ Water Surface Elevations and Flow Data
_____ Datum
_____ Grade elevations shown are equal to …

15.99 References

AASHTO LRFD *Bridge Design Specifications* (LRFD-BDS), Latest Edition and Interims.

AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (LRFD-SGS), Latest Edition and Interims.

AASHTO Guidelines for Performance-Based Seismic Design of Highway Bridges, Latest Edition.

AASHTO Guide Specifications for Seismic Isolation Design, Latest Edition.

AASHTO *Standard Specifications for Highway Bridges*, 17th edition

AASHTO *Manual for Bridge Evaluation*

ACI 355.4-11 (2014) *"Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary,"* American Concrete Institute, Farmington Hills, MI.

WSDOT *Bridge Inspection Manual* M 36-64

WSDOT *Geotechnical Design Manual* M 46-03

WSDOT *Design Manual* M 22-01

WSDOT *Construction Manual* M 41-01

WSDOT *Local Agency Guidelines* M 36-63

WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (*Standard Specifications*) M 41-10

WSDOT *Standard Plans* M 21.01

WSDOT *Hydraulics Manual* M 23-03

Washington Utilities and Transportation Commission *Clearance Rules and Regulations Governing Common Carrier Railroads*

American Railway Engineering and Maintenance Association (AREMA) *Manual for Railroad Engineering*. Note: This manual is used as the basic design and geometric criteria by all railroads. Use these criteria unless superseded by FHWA or WSDOT criteria.

The Union Pacific Railroad *"Guidelines for Design of Highway Separation Structures over Railroad (Overhead Grade Separation)"*