

Chapter 15 Structural Design Requirements for Design-Build Contracts

Contents

15.1	Manual Description	15-1
15.1.1	Purpose	15-1
15.2	Bridge Configuration Criteria	15-2
15.2.1	General	15-2
15.2.1.A	Structure Conceptual Plan	15-2
15.2.1.B	Bridge Redundancy	15-2
15.2.1.C	Bridge Deck Drainage	15-2
15.2.2	Railroad Crossings	15-2
15.2.2.A	Clearances	15-2
15.2.2.B	Crash Walls	15-2
15.2.2.C	Substructure	15-2
15.2.3	Temporary Bridges	15-3
15.2.4	Inspection and Maintenance Access	15-3
15.2.4.A	General	15-3
15.2.4.B	Bearings	15-3
15.2.4.C	Safety Cables, Handrails, and Anchors	15-3
15.2.4.D	Abutment Slopes	15-3
15.2.4.E	Access and Lighting	15-3
15.2.5	Bridge Types	15-4
15.2.6	Aesthetic Design Elements	15-4
15.2.7	Architectural Design Standards	15-4
15.2.8	Methods	15-5
15.2.9	Design-Builder Urban Design Team	15-5
15.2.10	Analysis and Design Criteria for Structural Widening and Modifications	15-5
15.2.11	Bridge Security	15-6
15.2.11.A	General	15-6
15.2.11.B	Natural Surveillance and Territorial Reinforcement	15-6
15.2.11.C	Hard Armoring	15-6

15.3	Load Criteria	15-7
15.3.1	Scope	15-7
15.3.2	Load Factors and Load Combinations	15-7
15.3.3	Permanent Loads	15-7
15.3.3.A	Future Deck Overlay Requirement	15-7
15.3.4	Live Loads	15-8
15.3.4.A	Design Live Load	15-8
15.3.4.B	Live Load Deflection Evaluation	15-8
15.3.4.C	Distribution to Superstructure	15-8
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	15-8
15.3.4.C.2	Cross section Type d from LRFD-BDS Table 4.6.2.2.1-1	15-9
15.3.4.C.3	Distribution to Substructure	15-9
15.3.4.C.4	Distribution to Crossbeam	15-9
15.3.5	Noise Barrier Walls	15-9
15.4	Seismic Design and Retrofit	15-10
15.4.1	General	15-10
15.4.2	WSDOT Additions and Modifications to AASHTO Guide Specifications for LRFD Seismic Bridge Design	15-10
15.4.2.A	Definitions	15-10
15.4.2.B	Earthquake Resisting Systems (ERS) Requirements for Seismic Design Categories (SDCs) C and D	15-10
15.4.2.C	Seismic Ground Shaking Hazard	15-12
15.4.2.D	Temporary and Staged Construction	15-12
15.4.2.E	Balanced Stiffness Requirements and Balanced Frame Geometry Recommendation	15-12
15.4.2.F	Selection of Analysis Procedure to Determine Seismic Demand	15-12
15.4.2.G	Member Ductility Requirement for SDCs C and D	15-12
15.4.2.H	Longitudinal Restraints	15-12
15.4.2.I	Abutments	15-13
15.4.2.I.1	4.2.11.1 - General	15-13
15.4.2.I.2	4.2.11.2 - Longitudinal Direction	15-13
15.4.2.I.3	4.2.11.3 - Transverse Direction	15-14
15.4.2.I.4	4.2.11.4 - Curved and Skewed Bridges	15-14
15.4.2.J	Foundation – General	15-14
15.4.2.K	Foundation – Spread Footing	15-15
15.4.2.L	Procedure 3: Nonlinear Time History Method	15-15
15.4.2.M	Drilled Shafts	15-15
15.4.2.N	Liquefaction Design Requirements	15-15
15.4.2.O	Reinforcing Steel	15-15
15.4.2.P	Concrete Modeling	15-15

15.4.2.Q	Lateral Confinement for Non-Oversized Strengthened Pile Shaft for SDCs C and D	15-15
15.4.2.R	Requirements for Capacity Protected Members	15-16
15.4.2.S	Superstructure Capacity Design for Transverse Direction (Integral Bent Cap) for SDCs C and D	15-16
15.4.2.T	Integral Bent Cap Joint Shear Design	15-16
15.4.2.U	Cast-in-Place and Precast Concrete Piles	15-16
15.4.3	Seismic Design Requirements for Bridge Modifications and Widening Projects .	15-17
15.4.3.A	Seismic Analysis and Retrofit Policy	15-17
15.4.3.B	Design and Detailing Considerations	15-17
15.4.3.B.1	Differential Settlement	15-17
15.4.3.B.2	Isolation Bearings	15-17
15.4.4	Seismic Retrofitting of Existing Bridges	15-17
15.4.4.A	Seismic Analysis Requirements	15-17
15.4.4.B	Seismic Retrofit Design	15-18
15.4.4.C	Isolation Bearings	15-18
15.5	Concrete Structures	15-19
15.5.1	General	15-19
15.5.2	Materials	15-19
15.5.2.A	Concrete	15-19
15.5.2.A.1	Cast-in-place (CIP) Concrete	15-19
15.5.2.A.2	Modulus of Elasticity	15-19
15.5.2.A.3	Shrinkage and Creep	15-19
15.5.2.A.4	Grout	15-19
15.5.2.A.5	Mass Concrete	15-20
15.5.2.A.6	Shotcrete	15-20
15.5.2.A.7	Lightweight Aggregate Concrete	15-20
15.5.2.B	Reinforcing Steel	15-20
15.5.2.B.1	Grades	15-20
15.5.2.B.2	Compressive Development Length	15-20
15.5.2.B.3	Splices	15-20
15.5.2.B.4	Welded Wire Reinforcement in Prestressed Concrete Girders, Walls, Barriers and Deck Panels	15-20
15.5.2.B.5	Reinforcing Bar Dowels and Resin Bonded Anchors .	15-21
15.5.2.C	Prestressing Steel	15-21
15.5.2.D	Post-Tensioning Systems	15-21
15.5.3	Design Considerations	15-21
15.5.3.A	Service and Fatigue Limit States	15-21
15.5.3.B	Strength Limit State	15-21
15.5.3.C	Post-Tensioning	15-22

15.5.4	Superstructures	15-22
15.5.4.A	Reinforced Concrete Superstructures	15-22
15.5.4.B	Box Girder Superstructures	15-22
15.5.4.B.1	Intermediate Diaphragms for Curved Concrete Box Girder Bridges	15-22
15.5.4.B.2	Drains	15-22
15.5.4.C	Prestressed Concrete Girder Superstructures	15-22
15.5.4.C.1	WSDOT Standard Girder Types and Construction Sequences	15-22
15.5.4.C.2	Superstructure Continuity	15-23
15.5.4.C.3	Continuous Structure Configuration	15-23
15.5.4.C.4	Girder Ends	15-23
15.5.4.C.5	Diaphragms	15-23
15.5.4.C.6	Barrier and Sidewalk Load Distribution	15-24
15.5.4.C.7	Composite Action	15-24
15.5.4.C.8	Dead Loads	15-24
15.5.4.C.9	Girder Stirrups	15-24
15.5.4.C.10	Transformed Section Properties	15-24
15.5.4.C.11	Deck Shrinkage	15-24
15.5.4.C.12	Deck Girder Superstructures	15-24
15.5.4.C.13	Slab Girders	15-25
15.5.4.C.14	Deck Bulb-Tee Girders	15-25
15.5.4.C.15	Wide Flange Deck Girders	15-25
15.5.4.C.16	Wide Flange Thin Deck Girders	15-25
15.5.4.C.17	Tub Girders	15-25
15.5.4.C.18	Spliced Prestressed Concrete Girders	15-25
15.5.5	Concrete Bridge Decks	15-25
15.5.5.A	Bridge Deck Requirements	15-26
15.5.5.B	Bridge Deck Reinforcement	15-26
15.5.5.C	Stay-in-Place (SIP) Deck Panels	15-26
15.5.5.D	Bridge Deck Protection	15-26
15.5.5.E	Bridge Deck HMA Paving	15-27
15.6	Steel Structures	15-28
15.6.1	Bridge Steels	15-28
15.6.2	Nonredundant Steel Tension Members, Redundancy, and Fracture Control	15-28
15.7	Substructure Design	15-29
15.7.1	General Substructure Considerations	15-29
15.7.1.A	Foundation Seals	15-29
15.7.1.B	Scour	15-29
15.7.1.C	Combination of Extreme Event Effects	15-29
15.7.1.C.1	Downdrag	15-29
15.7.1.C.2	Lateral Ground Displacement	15-29

15.7.2	Foundation Modeling for Seismic Loads	15-29
15.7.2.A	General	15-29
15.7.2.B	Bridge Model Section Properties	15-29
15.7.2.C	Spread Footing Modeling Methods	15-29
15.7.2.D	Deep Foundation Modeling Methods	15-29
15.7.2.D.1	Group Effects	15-29
15.7.2.D.2	Shaft Caps and Pile Footings	15-29
15.7.2.E	Design of Deep Foundations for Lateral Forces	15-30
15.7.2.E.1	Determination of Tip Elevations	15-30
15.7.2.E.2	Design for Lateral Loads	15-30
15.7.3	Column Design	15-30
15.7.3.A	Shear Design	15-30
15.7.3.B	Column Silos	15-30
15.7.3.C	Longitudinal Reinforcement	15-30
15.7.3.D	Longitudinal Reinforcement Splices	15-30
15.7.3.E	Longitudinal Reinforcement Development	15-31
15.7.3.E.1	Crossbeams	15-31
15.7.3.E.2	Footings	15-31
15.7.3.E.3	Drilled Shafts	15-31
15.7.3.F	Transverse Reinforcement	15-31
15.7.3.F.1	General	15-31
15.7.3.F.2	Spiral Splices and Hoops	15-31
15.7.3.G	Reduced Column Section	15-31
15.7.4	Crossbeam	15-31
15.7.5	Abutment Design and Details	15-31
15.7.5.A	General	15-31
15.7.5.A.1	Bent-Type and Isolated Abutments	15-31
15.7.5.A.2	Abutments on Structural Earth (SE) Walls and Geosynthetic Walls	15-32
15.7.5.B	Embankment and Backfill	15-32
15.7.5.B.1	General Clearances	15-32
15.7.5.B.2	Abutments on SE Walls and Geosynthetic Walls	15-32
15.7.5.B.3	Drainage and Backfill	15-32
15.7.5.C	Abutment Loading	15-32
15.7.5.D	Abutment Details	15-32
15.7.5.D.1	Bearing Seats	15-32
15.7.5.D.2	Transverse Girder Stops	15-32
15.7.5.D.3	Abutment Walls	15-32
15.7.6	Abutment Wing Walls and Curtain Walls	15-33
15.7.7	Footing Design	15-33
15.7.7.A	General Footing Criteria	15-33
15.7.7.B	Spread Footing Design	15-33
15.7.7.B.1	Foundation Design	15-33
15.7.7.B.2	Structural Design	15-33
15.7.7.C	Pile-Supported Footing Design	15-33

15.7.8	Shafts	15-33
15.7.8.A	Axial Resistance	15-33
15.7.8.B	Structural Design and Detailing	15-34
15.7.9	Piles and Piling	15-34
15.7.9.A	Pile Types	15-34
15.7.9.B	Pile Groups	15-34
15.7.9.C	Battered Piles	15-34
15.7.9.D	Structural Design and Detailing of CIP Concrete Piles	15-34
15.7.9.E	Structural Steel Pipe Piles	15-34
15.7.9.F	Pile Resistance	15-34
15.7.10	Concrete-Filled Steel Tubes	15-34
15.7.10.A	Design Requirements	15-34
15.8	Walls and Buried Structures	15-35
15.8.1	Retaining Walls	15-35
15.8.2	Noise Barrier Walls	15-35
15.8.3	Buried Structures	15-35
15.9	Bearings and Expansion Joints	15-36
15.9.1	Expansion Joints	15-36
15.9.1.A	General Considerations	15-36
15.9.1.A.1	Concrete Bridges	15-36
15.9.1.A.2	Steel Bridges	15-36
15.9.1.B	General Design Criteria	15-36
15.9.1.B.1	Shrinkage Effects	15-37
15.9.1.B.2	Thermal Effects	15-37
15.9.1.C	Small Movement Range Joints	15-38
15.9.1.C.1	Compression Seals	15-38
15.9.1.C.2	Rapid-Cure Silicone Sealants	15-38
15.9.1.C.3	Asphaltic Plug Joints	15-38
15.9.1.C.4	Headers	15-38
15.9.1.D	Medium Movement Range Joints	15-39
15.9.1.D.1	Steel Sliding Plate Joints	15-39
15.9.1.D.2	Strip Seal Joints	15-39
15.9.1.D.3	Bolt-down Panel Joints	15-39
15.9.1.E	Large Movement Range Joints	15-39
15.9.1.E.1	Steel Finger Joints	15-39
15.9.1.E.2	Modular Expansion Joints	15-40
15.9.2	Bearings	15-41
15.9.2.A	General Considerations	15-41
15.9.2.B	Force Considerations	15-41
15.9.2.C	Movement Considerations	15-41
15.9.2.C.1	Elastomeric and Fabric Pad Bearings	15-41
15.9.2.C.2	HLMR Bearings	15-41
15.9.2.D	Detailing Considerations	15-42

15.9.2.E	Bearing Types	15-42
15.9.2.E.1	Elastomeric Bearings	15-42
15.9.2.E.2	Fabric Pad Sliding Bearings	15-43
15.9.2.E.3	Pin Bearings	15-44
15.9.2.E.4	Rocker and Roller Type Bearings	15-44
15.9.2.E.5	Spherical Bearings	15-44
15.9.2.E.6	Disc Bearings	15-44
15.9.2.E.7	Seismic Isolation Bearings	15-44
15.9.2.F	Miscellaneous Details	15-44
15.9.2.F.1	Temporary Support before Grouting Masonry Plate	15-44
15.9.2.F.2	Anchor Bolts	15-45
15.9.2.G	Contract Drawing Representation	15-45
15.9.2.H	Shop Drawing Review	15-45
15.9.2.I	Bearing Replacement Considerations	15-45
15.10	Signs, Barriers, Bridge Approach Slabs, and Utilities	15-46
15.10.1	Sign and Luminaire Supports	15-46
15.10.2	Bridge Traffic Barriers	15-46
15.10.2.A	General Guidelines and Policy	15-46
15.10.3	At Grade Concrete Barriers	15-46
15.10.3.A	Moment Slab Traffic Barrier Design	15-46
15.10.3.A.1	Structural Capacity	15-46
15.10.3.A.2	Global Stability	15-46
15.10.3.A.3	Geometry	15-46
15.10.3.A.4	Soil Reinforcement	15-46
15.10.3.A.5	Wall Panel	15-47
15.10.3.B	Precast Concrete Barrier	15-47
15.10.4	Bridge Traffic Barrier Rehabilitation	15-47
15.10.4.A	General Guidelines and Policy	15-47
15.10.4.A.1	Design Criteria	15-47
15.10.5	Bridge Railing	15-48
15.10.5.A	General Guidelines and Policy	15-48
15.10.6	Bridge Approach Slabs	15-48
15.10.6.A	Bridge Approach Slab Design Criteria	15-48
15.10.6.B	Bridge Approach Slab Detailing	15-48
15.10.6.C	Skewed Bridge Approach Slabs	15-49
15.10.6.D	Approach Anchors and Expansion Joints	15-49
15.10.6.E	Bridge Approach Slab Addition or Retrofit to Existing Bridges	15-49
15.10.6.F	Bridge Approach Slab Staging	15-49
15.10.7	Traffic Barrier on Bridge Approach Slabs	15-50
15.10.7.A	Bridge Approach Slab over Wing Walls, Cantilever Walls or Geosynthetic Walls	15-50
15.10.7.B	Bridge Approach Slab over SE Walls	15-50

15.10.8	Utilities Installation on New and Existing Structures	15-50
15.10.8.A	General Concepts	15-50
15.10.8.A.1	Coating and Corrosion Protection	15-50
15.10.8.B	Utility Design Criteria	15-50
15.10.8.B.1	Utility Location	15-51
15.10.8.B.2	Termination at the Bridge Ends	15-51
15.10.8.B.3	Utility Expansion	15-51
15.10.8.B.4	Utility Blockouts	15-51
15.10.8.B.5	Gas Lines or Volatile Fluids	15-51
15.10.8.B.6	Water Lines	15-51
15.10.8.B.7	Sewer Lines	15-51
15.10.8.B.8	Electrical (Power and Communications)	15-51
15.10.8.C	Box/Tub Girder Bridges	15-51
15.10.8.D	Traffic Barrier Conduit	15-51
15.10.8.E	Conduit Types	15-51
15.10.8.F	Utility Supports	15-51
15.10.8.F.1	Pipe Hangers	15-52
15.10.8.F.2	Surface Mounting	15-52
15.10.9	Review Procedure for Utility Installations on Existing Structures	15-52
15.10.10	Anchors for Permanent Attachments	15-52
15.10.11	Drainage Design	15-52
15.11	Detailing Practices	15-53
15.12	Bridge Load Rating.	15-54
15.12.1	General	15-54
15.13	Appendices	15-55
Appendix 15.2-A1	Conceptual Plan Checklist	15-56
15.99	References	15-57

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).

Where references are made in this manual to the Contractor, it shall refer to the Design Builder.

Any element in this manual that requires State Bridge Design Engineer approval shall not be allowed unless otherwise approved through the RFP Technical Requirements.

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

See [Sections 2.3.11.D](#) and [7.5.3](#).

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 Widening 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:

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

$\gamma_{LL} = 1.0$ 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 –
[See Section 10.13.2.C.](#)
- 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).

15.4.2.1 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:

15.4.2.1.1 4.2.11.1 - General

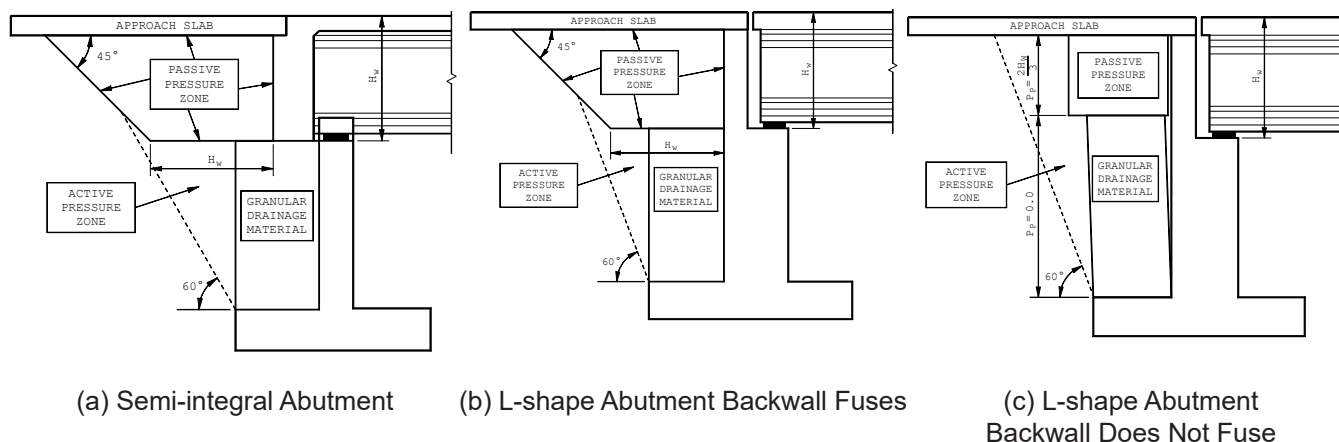
The participation of abutment walls in providing resistance to seismically induced inertial loads may be considered in the seismic design of bridges either to reduce column sizes or reduce the ductility demand on the columns. Damage to backwalls and wingwalls during earthquakes may be considered acceptable 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.

15.4.2.1.2 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



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_l) 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 conform to the requirements of Section 5.1.1.J.

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 $\leq R < 800$ feet - at midspan.

For 400 feet $\leq R < 600$ feet - at $\frac{1}{2}$ points of span.

For $R < 400$ feet - at $\frac{1}{4}$ points of span.

15.5.4.B.2 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" or 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 girders shall meet the requirements of Section 5.6.7.B. CIP toppings slabs shall be provided, except where explicitly allowed otherwise when used as part of a split box buried structure.

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 in²/feet for the top mat and 0.27 in²/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 Bridge Steels

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

15.6.2 **Nonredundant Steel Tension Members, Redundancy, and Fracture Control**

Nonredundant **Steel Tension Member** 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.

15.7.2 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

See [Section 7.2.3](#).

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

See [Section 7.3.3](#).

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

See [Section 7.3.5.B](#).

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

See [Section 7.8.2 Item 11](#).

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 8.1.2.A.1](#).

15.7.5.B Embankment and Backfill**15.7.5.B.1 General Clearances**

[See Section 7.5.3.](#)

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

[See Section 7.5.11.](#)

15.7.5.C Abutment Loading

[See Section 7.5.4.](#)

15.7.5.D Abutment Details**15.7.5.D.1 Bearing Seats**

[See Section 7.5.6.B.](#)

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

[See Section 7.5.10.](#)

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 $\frac{3}{4} L_d$.

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". Requirements from [Sections 7.7.5.A and 7.7.5.B](#) shall be followed.

15.7.8 Shafts

15.7.8.A Axial Resistance

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.

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.

15.7.8.B Structural Design and Detailing

See [Section 7.8.2](#).

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

See [Section 7.9.6](#).

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

Retaining Walls shall meet the requirements in [Section 8.1](#), except as otherwise allowed in the Technical Requirements.

15.8.2 Noise Barrier Walls

Noise Barrier Walls shall meet the requirements in [Section 8.2](#), except as otherwise allowed in the Technical Requirements.

15.8.3 Buried Structures

Buried Structures shall meet the requirements in [Section 8.3](#), except as otherwise allowed in the Technical Requirements. Any structure not defined therein and supporting earth or Roadway fill shall not be classified as a Buried Structure but shall be considered a bridge, meeting all requirements for a bridge. Alternate material(s) than specified in [Section 8.3.1](#) are not permitted unless otherwise allowed in the Technical Requirements.

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

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 $\leq 1\frac{3}{4}$ inch
Medium Movement Range Joints	$1\frac{3}{4}$ 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:

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

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:

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

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.C.4](#) 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 rail. The special anchorage shall be used for very high traffic volumes and applications subject to snowplow hits.

In applications highly susceptible to snowplow hits and concomitant damage, the intermittent steel plates shall be detailed to protrude $\frac{1}{4}$ " 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 [Standard Specifications Section 6-02.3\(13\)C Modular Expansion Joint Systems](#) at time of contract award.

Center beam field splices shall be carefully designed and constructed to mitigate fatigue susceptibility in accordance with the [modular expansion joint provisions contained in the Standard Specifications](#).

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 in [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 ⅛-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.

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 AASHTO 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 $\frac{3}{16}$ -inch thick and recessed $\frac{3}{32}$ -inch into a minimum $\frac{1}{2}$ -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 $\frac{1}{4}$ inch thick and shall be recessed $\frac{1}{8}$ -inch into a $\frac{1}{2}$ -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 $\frac{1}{16}$ -inch and a maximum thickness of $\frac{1}{8}$ -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 joint 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 [10.3.3.B.2](#) through [Section 10.3.3.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](#).

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*, 17th 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 redirection 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:

$$\text{Flexure} = (1.3) * (1.67) * (10 \text{ kip}) / (0.9) = 24.10 \text{ kip}$$

$$\text{Shear} = (1.3) * (1.67) * (10 \text{ kip}) / (0.85) = 25.54 \text{ kip}$$

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

See [Section 10.6.2](#).

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

See [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

See [Section 10.8.2.D](#).

15.10.8.B.5 Gas Lines or Volatile Fluids

See [Section 10.8.2.E](#).

15.10.8.B.6 Water Lines

See [Section 10.8.2.F](#).

15.10.8.B.7 Sewer Lines

See [Section 10.8.2.G](#).

15.10.8.B.8 Electrical (Power and Communications)

See [Section 10.8.2.H](#).

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

See [Section 10.8.4](#).

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

See [Section 10.8.6](#).

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.

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

See [Section 10.8.6.C](#).

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](#).

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 - Symbolology, Section 5 PS&E Standards as well as the *Plans Preparation Manual*.

15.12 Bridge Load Rating

15.12.1 General

Load Ratings shall meet the requirement in Chapter 13, except as otherwise allowed in the Technical Requirements. 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

Appendix 15.2-A1 Conceptual Plan Checklist

Project _____ SR _____ Concept Plan by _____ Check by _____ Date _____

<div>Plan</div> <ul style="list-style-type: none"> _____ 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 	<div>Miscellaneous</div> <ul style="list-style-type: none"> _____ Assumed Structure Type _____ Live Loading _____ Undercrossing Alignment Profiles/Elevs . _____ Bridge Deck Superelevation Diagrams _____ Bridge Deck Alignment Curve Data _____ Names <div>Elevation</div> <ul style="list-style-type: none"> _____ 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 ...
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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

15.99 References

1. See [Section 1.99](#).
2. AASHTO Guidelines for Performance-Based Seismic Design of Highway Bridges, Latest Edition.
3. AASHTO Guide Specifications for Seismic Isolation Design, Latest Edition.
4. AASHTO [Standard Specifications for Highway Bridges](#), 17th edition
5. AASHTO *Manual for Bridge Evaluation*
6. ACI 355.4-11 (2014) “Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary,” American Concrete Institute, Farmington Hills, MI.
7. WSDOT [Bridge Inspection Manual](#) M 36-64
8. WSDOT [Local Agency Guidelines](#) M 36-63
9. WSDOT [Standard Specifications for Road, Bridge, and Municipal Construction](#) ([Standard Specifications](#)) M 41-10
10. WSDOT [Standard Plans](#) M 21.01
11. Washington Utilities and Transportation Commission [Clearance Rules and Regulations Governing Common Carrier Railroads](#)
12. 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.
13. The Union Pacific Railroad “[Guidelines for Design of Highway Separation Structures over Railroad \(Overhead Grade Separation\)](#)”

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