

Chapter 15 Structural Design Requirements for Design-Build Contracts

Contents

15.1	Manual Description	15-1
15.1.1	Purpose.....	15-1
15.1.2	Specifications.....	15-1
15.2	Bridge Configuration Criteria.....	15-2
15.2.1	General	15-2
15.2.2	Railroad Crossings.....	15-3
15.2.3	Temporary Bridges	15-3
15.2.4	Inspection and Maintenance Access	15-3
15.2.5	Bridge Types.....	15-5
15.2.6	Aesthetic Design Elements	15-5
15.2.7	Architectural Design Standards.....	15-6
15.2.8	Methods	15-6
15.2.9	Design-Builder Urban Design Team	15-6
15.2.10	Analysis and Design Criteria for Structural Widening and Modifications	15-7
15.2.11	Bridge Security.....	15-7
15.3	Load Criteria	15-9
15.3.1	Scope.....	15-9
15.3.2	Load Factors and Load Combinations	15-9
15.3.3	Permanent Loads.....	15-9
15.3.4	Live Loads.....	15-10
15.3.5	Noise Barrier Walls	15-11
15.4	Seismic Design and Retrofit.....	15-12
15.4.1	General	15-12
15.4.2	WSDOT Additions and Modifications to AASHTO Guide Specifications for LRFD Seismic Bridge Design	15-12
15.4.3	Seismic Design Requirements for Bridge Modifications and Widening Projects ...	15-24
15.4.4	Seismic Retrofitting of Existing Bridges.....	15-26

15.5	Concrete Structures	15-28
15.5.1	General	15-28
15.5.2	Materials	15-28
15.5.3	Design Considerations	15-31
15.5.4	Superstructures	15-32
15.5.5	Concrete Bridge Decks	15-36
15.6	Steel Structures	15-38
15.6.1	Design Considerations	15-38
15.6.2	Girder Bridges	15-41
15.6.3	Design of I-Girders	15-42
15.6.4	Plan Details	15-47
15.6.5	Painting of Existing Steel Bridges	15-50
15.6.6	Corrosion of Steel Foundations and Buried Structures	15-50
15.7	Substructure Design	15-51
15.7.1	General Substructure Considerations	15-51
15.7.2	Foundation Modeling for Seismic Loads	15-52
15.7.3	Column Design	15-54
15.7.4	Crossbeam	15-56
15.7.5	Abutment Design and Details	15-56
15.7.6	Abutment Wing Walls and Curtain Walls	15-58
15.7.7	Footing Design	15-58
15.7.8	Shafts	15-59
15.7.9	Piles and Piling	15-61
15.7.10	Concrete-Filled Steel Tubes	15-62
15.8	Walls and Buried Structures	15-63
15.8.1	Retaining Walls	15-63
15.8.2	Noise Barrier Walls	15-66
15.8.3	Buried Structures	15-67
15.9	Bearings and Expansion Joints	15-70
15.9.1	Expansion Joints	15-70
15.9.2	Bearings	15-76

15.10 Signs, Barriers, Bridge Approach Slabs, and Utilities	15-82
15.10.1 Sign and Luminaire Supports	15-82
15.10.2 Bridge Traffic Barriers	15-90
15.10.3 At Grade Concrete Barriers	15-92
15.10.4 Bridge Traffic Barrier Rehabilitation	15-94
15.10.5 Bridge Railing	15-95
15.10.6 Bridge Approach Slabs	15-95
15.10.7 Traffic Barrier on Bridge Approach Slabs	15-97
15.10.8 Utilities Installation on New and Existing Structures	15-98
15.10.9 Review Procedure for Utility Installations on Existing Structures	15-101
15.10.10 Anchors for Permanent Attachments	15-101
15.10.11 Drainage Design	15-102
15.11 Detailing Practices	15-103
15.11.1 Standard Practices	15-103
15.11.2 Bridge Office Standard Drawings and Office Examples	15-107
15.11.3 Plan Sheets	15-107
15.11.5 Structural Steel	15-110
15.11.6 Aluminum Section Designations	15-112
15.11.7 Abbreviations	15-112
15.12 Bridge Load Rating	15-113
15.12.1 General	15-113
15.12.2 Load Rating Software	15-113
15.13 Appendices	15-114
Appendix 15.2-A1 Conceptual Plan Checklist	15-115
15.99 References	15-116

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) and AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (SEISMIC).

15.1.2 Specifications

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

- AASHTO LRFD
- AASHTO SEISMIC
- WSDOT [Standard Plans](#) M 21-01 – A-50 Series Standard Plans for Embankment Geometry at Bridge Ends

15.2 Bridge Configuration Criteria

15.2.1 General

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](#).

B. Bridge Redundancy

Bridge substructure shall have the following minimum number of columns to be considered to provide conventional levels of redundancy in accordance with AASHTO LRFD *Bridge Design Specification* Section 1.3.4:

- One column minimum for roadway widths 40' wide and under.
- Two columns minimum for roadway widths over 40' to 60'.
- Three columns minimum for roadway widths over 60'.

Bridge superstructure shall have the following minimum number of webs to be considered to provide conventional levels of redundancy in accordance with AASHTO LRFD *Bridge Design Specification* Section 1.3.4:

- Three webs minimum for roadway widths 32' and under.
- Four webs minimum for roadway widths over 32'. See [Bridge Standard Drawing 2.3-A2-1](#) for details.

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

A. Horizontal Clearances

For railroad overcrossings, minimum horizontal clearances are as noted below:

	Railroad Alone
Fill Section	14'
Cut Section	16'

Horizontal clearance shall be measured from the center of the outside track to the face of pier. When the track is on a curve, the minimum horizontal clearance shall be increased at the rate of $1\frac{1}{2}$ " for each degree of curvature. An additional 8' of clearance for off-track equipment shall only be provided when specifically requested by the railroad.

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, "Guidelines for Design of Highway Separation Structures over Railroad (Overhead Grade Separation)"
- [AREMA Manual](#)
- WSDOT Railroad Liaison Engineer
- The Railroad

C. Substructure

For highway over railway grade separations, the top of footings for bridge piers or retaining walls adjacent to railroad tracks shall be 2' or more below the elevation of the top of tie and shall not have less than 2' of cover from the finished ground. The footing face shall not be closer than 10' to the center of the track.

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

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.

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.

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.

D. Abutment Slopes

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

E. Access and Lighting**1. Concrete Box and Prestressed Concrete TubGirders**

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 shall conform to the following superstructure depth-to-span ratios based on past WSDOT experience, superseding AASHTO LRFD Section 2.5.2.6.3.

The optional live load deflection limits of AASHTO LRFD Sections 3.6.1.3.2 and 2.5.2.6.2 shall be satisfied.

For both simple and continuous spans, the span length is the horizontal distance between centerlines of bearings.

Refer to [Section 2.2.4](#) for superstructure depth requirements for inspection and maintenance access.

For the required minimum depth to span ratios see [Section 2.4.1](#).

WSDOT restricts the use of cast-in-place reinforced concrete Tee-Beam girder for bridge superstructure. This type of superstructure may only be used for bridges with tight curvatures or irregular geometry upon Bridge Design Engineer approval.

WSDOT restricts the use of timber girders for bridge superstructures to non-vehicle use bridges or temporary bridges.

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 aid 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.

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 as Context Sensitive Design (CSS) and Crime Prevention Through Environmental Design (CEPTED).

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

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.

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.

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 $\frac{1}{2}$ 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

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

15.3.2 Load Factors and Load Combinations

A value of 1.0 shall be used for h_i in Equation 3.4.1-1 of AASHTO LRFD except for the design of columns when a minimum value of g_i is required by Article 3.4.1 of AASHTO LRFD. In such a case, h_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 AASHTO LRFD.

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 AASHTO LRFD.

15.3.3 Permanent Loads

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

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

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 AASHTO LRFD
- For bridges modified in such a way that do not include new substructure elements – Live load criteria of the original design
- For bridges used for temporary detour or other temporary purposes – minimum 75 percent of HL-93 live load in accordance with AASHTO LRFD

The application of design vehicular live loads shall be as specified in AASHTO LRFD Section 3.6.1.3. The design tandem, or “low boy”, defined in LRFD 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 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.

B. Live Load Deflection Evaluation

Article 2.5.2.6.2 of the AASHTO LRFD is mandatory in its entirety.

C. Distribution to Superstructure

1. Cross sections a, b, c, e, k, and also i and j if sufficiently connected to act as a unit from AASHTO LRFD 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 AASHTO LRFD Section C4.6.2.2d 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.

2. Cross section Type d from AASHTO LRFD 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 AASHTO LRFD and the total number of webs in the cross section. The correction factor for live load distribution for skewed supports as specified in AASHTO LRFD Tables 4.6.2.2.2e-1 and 4.6.2.2.3c1 for shear shall apply.

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.

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 AASHTO LRFD 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 and the AASHTO SEISMIC are the foundation seismic design criteria documents used to design highway bridges in Washington State.

This chapter supplements and supersedes the AASHTO LRFD by providing WSDOT seismic design criteria, policy and practice.

The importance classifications for all highway bridges in Washington State are classified as “**Ordinary**” except for special major bridges. Special major bridges fitting the classifications of either “Critical” or “**Recovery**” will be so designated in the RFP.

Bridges are considered as Critical, **Recovery**, or **Ordinary** for their operational classification as described below. Two-level performance criteria are required for design of **Recovery** and Critical bridges.

- **Critical Bridges**

Critical bridges are expected to provide immediate access to emergency and similar life-safety facilities after an earthquake. The Critical designation is typically reserved for high-cost projects where WSDOT intends to protect the investment or for projects that would be especially costly to repair if they were damaged during an earthquake.

- **Recovery Bridges**

Recovery bridges serve as vital links for rebuilding damaged areas and provide access to the public shortly after an earthquake.

- **Ordinary Bridges**

All bridges not designated as either Critical or **Recovery** shall be designated as **Ordinary**.

The expected seismic performance, post-earthquake service levels, and post-earthquake damage states for Critical, **Recovery**, or **Ordinary** bridges shall be in accordance with Section 4.1.

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

WSDOT modifications to the AASHTO SEISMIC are as follows:

A. Definitions

Guide Specifications Article 2

Revise existing definitions and add new definitions as follows:

- **Oversized Pile Shaft**

A drilled shaft foundation that is larger in diameter than the supported column and has a reinforcing cage larger than and independent of the column's reinforcement cage. The size of the shaft shall be in accordance with [Section 7.8.2](#).

- **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.

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

Guide Specifications 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 [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 Bridge Seismic Design* 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.

C. Seismic Ground Shaking Hazard**Guide Specifications 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 AASHTO SEISMIC.

In cases where the site coefficients used to adjust mapped values of design ground motion for local conditions are inappropriate to determine the design spectra in accordance with general procedure of Article 3.4.1 (such as the period at the end of constant design spectral acceleration plateau (T_s) is greater than 1.0 second or the period at the beginning of constant design spectral acceleration plateau (T_o) is less than 0.2 second), a site-specific ground motion response analysis shall be performed.

The spectral response parameters shall be determined using USGS 2014 *Seismic Hazard Maps and Site Coefficients* defined in Section 4.2.3.

D. Selection of Seismic Design Category (SDC)**Guide Specifications Article 3.5**

A pushover analysis shall be used to determine displacement capacity for both SDCs C and D.

E. Temporary and Staged Construction**Guide Specifications 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.

F. Load and Resistance Factors**Guide Specifications Article 3.7**

Use load factors of 1.0 for all permanent loads. The load factor for live load shall be 0.0 when pushover analysis is used to determine the displacement capacity. Use a live load factor of 0.5 for all other extreme event cases. Unless otherwise noted, all ϕ factors shall be taken as 1.0.

G. Balanced Stiffness Requirements and Balanced Frame Geometry Recommendation**Guide Specifications 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.

H. Selection of Analysis Procedure to Determine Seismic Demand

Guide Specifications 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.

I. Member Ductility Requirement for SDCs C and D

Guide Specifications 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.

J. Longitudinal Restrainers

Guide Specifications 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 Structures](#) (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 Equation C4.13.1-1.

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

K. Abutments

Guide Specifications Article 5.2

Abutments are revised as follows:

1. 4.2.11.1 - General

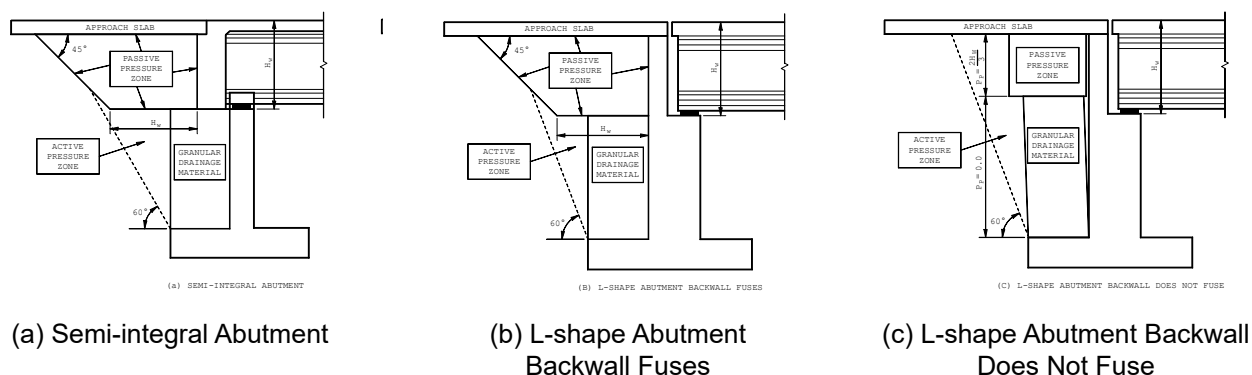
The participation of abutment walls in providing resistance to seismically induced inertial loads may be considered in the seismic design of bridges either to reduce column sizes or reduce the ductility demand on the columns. Damage to backwalls and wingwalls during earthquakes may be considered acceptable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

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.

3. 4.2.11.2.1 - Abutment Stiffness and Passive Pressure Estimate

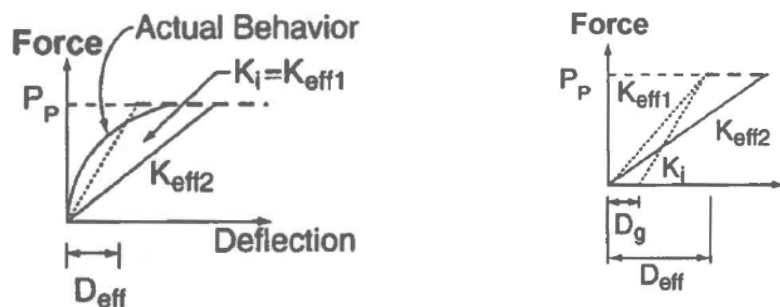
Abutment stiffness, K_{eff} in kip/feet, and passive capacity, P_p in kips, shall be characterized by a bilinear or other higher order nonlinear relationship as shown in Figure 4.2.11-2. When the motion of the back wall is primarily translation, passive pressures may be assumed uniformly distributed over the height (H_w) of the back wall or end diaphragm. The total passive force shall be determined as:

$$P_p = p_p H_w W_w \quad (4.2.11.2.1-1)$$

Where:

- p_p = passive lateral earth pressure behind backwall or diaphragm (ksf)
- H_w = height of back wall or end diaphragm exposed to passive earth pressure (feet)
- W_w = width of back wall or diaphragm (feet)

Figure 4.2.11-2 Characterization of Abutment Capacity and Stiffness



(a) Semi-integral Abutment

(b) L-shape Abutment

4. 4.2.11.2.2 - Calculation of Best Estimate Passive Pressure P_p

If the strength characteristics of compacted or natural soils in the “passive pressure zone” are known, then the passive force for a given height, H_w , shall be calculated using accepted analysis procedures. These procedures should account for the interface friction between the wall and the soil. The properties used shall be those indicative of the entire “passive pressure zone” as indicated in Figure 1. Therefore, the properties of backfill present immediately adjacent to the wall in the active pressure zone may not be appropriate as a weaker failure surface can develop elsewhere in the embankment.

For L-shape abutments where the backwall is not designed to fuse, H_w shall conservatively be taken as the depth of the superstructure, unless a more rational soil-structure interaction analysis is performed.

If presumptive passive pressures are to be used for design, then the following criteria shall apply:

- Soil in the “passive pressure zone” shall be compacted in accordance with *Standard Specifications* Section 2-03.3(14)I.
- For cohesionless, nonplastic backfill (fines content less than 30 percent), the passive pressure p_p shall be assumed equal to $2H_w/3$ ksf per foot of wall length.
- For other cases, including abutments constructed in cuts, the passive pressures shall be developed by a Geotechnical Engineer.

5. 4.2.11.2.3 - Calculation of Passive Soil Stiffness

Equivalent linear secant stiffness, K_{eff} in kip/feet, is required for analyses. For semi-integral or L-shape abutments initial secant stiffness may be determined as follows:

$$K_{eff1} = \frac{P_p}{(F_w H_w)} \quad (4.2.11.2.3-1)$$

Where:

- P_p = passive lateral earth pressure capacity (kip)
- H_w = height of back wall (feet)
- F_w = the value of F_w to use for a particular bridge is found in Table C3.11.1-1 of the AASHTO LRFD.

For L-shape abutments, the expansion gap shall be included in the initial estimate of the secant stiffness as specified in:

$$K_{eff1} = \frac{P_p}{(F_w H_w + D_g)} \quad (4.2.11.2.3-2)$$

Where:

- D_g = width of gap between backwall and superstructure (feet)

For SDCs C and D, where pushover analyses are conducted, values of P_p and the initial estimate of K_{eff1} should be used to define a bilinear load-displacement behavior of the abutment for the capacity assessment.

6. 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 AASHTO SEISMIC shall be provided. 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.

7. 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.

L. Foundation – General

Guide Specifications 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.

M. Foundation – Spread Footing

Guide Specifications 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.

N. Procedure 3: Nonlinear Time History Method

Guide Specifications 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.

O. I_{eff} for Box Girder Superstructure

Guide Specifications Article 5.6.3

The gross moment of inertia shall be used for box girder superstructure modeling.

P. Foundation Rocking

Guide Specifications Article 6.3.9

Foundation rocking shall not be used for the design of WSDOT bridges.

Q. Drilled Shafts**Guide Specifications Article C6.5**

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

R. Longitudinal Direction Requirements**Guide Specifications Article 6.7.1**

Case 2: Earthquake Resisting System (ERS) with abutment contribution may be used provided that the mobilized longitudinal passive pressure is not greater than 70 percent of the value obtained using the procedure given in Article 5.2.2.1.

S. Liquefaction Design Requirements**Guide Specifications Article 6.8**

Soil liquefaction assessment shall be based on [Geotechnical Design Manual](#) Section 6.4.2.8.

T. 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.

U. Concrete Modeling**Guide Specifications 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 of 0.008 and the member ductility demand shall be limited to 4 maximum.

V. Expected Nominal Moment Capacity**Guide Specifications Article 8.5**

Replace the definition of λ_{mo} with the following:

$$\begin{aligned}\lambda_{mo} &= \text{overstrength factor} \\ &= 1.2 \text{ for ASTM A 706 Grade 60 reinforcement} \\ &= 1.4 \text{ for ASTM A 615 Grade 60 reinforcement}\end{aligned}$$

W. Interlocking Bar Size**Guide Specifications Article 8.6.7**

The longitudinal reinforcing bar inside the interlocking portion of a column (interlocking bars) shall be the same size of bars used outside the interlocking portion.

X. Splicing of Longitudinal Reinforcement in Columns Subject to Ductility Demands for SDCs C and D**Guide Specifications Article 8.8.3**

The splicing of longitudinal column reinforcement outside the plastic hinging region 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. The design engineer shall clearly identify the locations where splices in longitudinal column reinforcement are permitted on the plans. In general where the length of the rebar cage is less than 60 feet (72 feet for No. 14 and No. 18 bars), no splice in the longitudinal reinforcement shall be allowed.

Y. Development Length for Column Bars Extended into Oversized Pile Shafts for SDCs C and D**Guide Specifications Article 8.8.10**

Extending column bars into oversized shaft shall be in accordance with Section 7.4.4.C, based on [TRAC Report WARD 417.1](#) "Non-Contact Lap Splice in Bridge Column Shaft Connections".

Z. Lateral Confinement for Oversized Pile Shaft for SDCs C and D**Guide Specifications Article 8.8.12**

The requirement of this article for shaft lateral reinforcement in the column-shaft splice zone may be replaced with the requirements of [Section 7.8.2.K](#).

AA. Lateral Confinement for Non-Oversized Strengthened Pile Shaft for SDCs C and D**Guide Specifications 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.

AB. Requirements for Capacity Protected Members**Guide Specifications 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.

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

Guide Specifications 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.

AD. Superstructure Design for Non Integral Bent Caps for SDCs B, C, and D

Guide Specifications Article 8.12

Non integral bent caps shall not be used for continuous concrete bridges in SDC B, C, and D except at the expansion joints between superstructure segments.

AE. Integral Bent Cap Joint Shear Design

Guide Specifications 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.

AF. Cast-in-Place and Precast Concrete Piles

Guide Specifications 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.

AG. Seismic Resiliency using Innovative Materials and Construction

Innovative materials and bridge construction are ideas that encourage engineers to consider principles that will enhance bridge performance, speed up construction, or add any other benefit to the industry. BDM [Section 14.4](#) describes the self-centering columns that are designed restore much of their original shape after a seismic event. They're intended to improve the serviceability of a bridge after an earthquake. Self-centering columns are constructed with a precast concrete column segment with a duct running through it longitudinally. They rest on footings with post-tensioning (PT) strand developed into them. Once the precast column piece is set on the footing, the PT strand threads through the duct and gets anchored into the crossbeam above the column. The PT strand is unbonded to the column segment. As a column experiences a lateral load, the PT strand elastically stretches to absorb the seismic energy and returns to its original tension load after the seismic event. The expectation is the column would rotate as a rigid body and the PT strand would almost spring the column back to its original orientation. Like self-centering columns, Shape Memory Alloy (SMA) and Engineered Cementitious Composite (ECC) products are introduced into bridge design as a means to improve ductility, seismic resilience, and serviceability of a bridge after an earthquake. SMA is a class of alloys that are manufactured from either a combination of nickel and titanium or copper, magnesium and aluminum. The alloy is shaped into round bars in sizes similar to conventional steel reinforcement. When stressed, the SMA can undergo large deformations and return to original shape.

15.4.3 Seismic Design Requirements for Bridge Modifications and Widening Projects

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 FRP Section 2.13 and Section 4.3.3.

The spectral response parameters shall be determined using USGS 2014 Seismic Hazard Maps and Site Coefficients defined in Section 4.2.3.

B. Design and Detailing Considerations

1. Support Length

The support length at existing abutments, piers, inspan hinges, and pavement seats shall be checked. If there is a need for longitudinal restrainers, transverse restrainers, or additional support length on the existing structure, they shall be included in the widening design.

2. Connections Between Existing and New Elements

Connections between the existing elements and new elements shall be designed for maximum overstrength forces. Where yielding is expected in the crossbeam connection at the extreme event limit state, the new structure shall be designed to carry live loads independently at the Strength I limit state. In cases where large differential settlement and/or a liquefaction induced loss of bearing strength are expected, the connections may be designed to deflect or hinge in order to isolate the two parts of the structure. Elements subject to inelastic behavior shall be designed and detailed to sustain the expected deformations.

Longitudinal joints that isolate the decks between the existing and new structures are not permitted.

3. 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.

4. Foundation Types

The foundation type of the new structure should match that of the existing structure. However, a different type of foundation may be used for the new structure due to geotechnical recommendations or the limited space available between existing and new structures. For example, a shaft foundation may be used in lieu of spread footing.

5. Existing Strutted Columns

The horizontal strut between existing columns may be removed. The existing columns shall then be analyzed with the new unbraced lengths and retrofitted if necessary.

6. Non Structural Element Stiffness

Median barriers and other potentially stiffening elements shall be isolated from the columns to allow column deformation.

Deformation capacities of existing bridge members that do not meet current detailing standards shall be determined using the provisions of Section 7.8 of the [Retrofitting Manual for Highway Structures: Part 1 – Bridges](#), FHWAHRT06032.

Deformation capacities of existing bridge members that meet current detailing standards shall be determined using the latest edition of the AASHTO SEISMIC.

In lieu of specific data, the reinforcement properties provided in [Table 4.3.2-1](#) shall be used.

7. 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

Seismic retrofitting of existing bridges shall be performed in accordance with the FHWA publication FHWA HRT 06032, [Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges](#) as follows:

- Article 1.5.3 The spectral response parameters shall be determined using USGS 2014 Seismic Hazard Maps and Site Coefficients defined in Section 4.2.3.
- Article 7.4.2 Seismic Loading in Two or Three Orthogonal

Revise the first paragraph as follows:

When combining the response of two or three orthogonal directions the design value of any quantity of interest (displacement, bending moment, shear or axial force) shall be obtained by the 100-30 percent combination rule as described in AASHTO SEISMIC Article 4.4.

- Delete Eq. 7.44 and replace with the following:

$$L_p = \text{the maximum of } [(8800\epsilon y_{db}) \text{ or } (0.08L + 4400\epsilon y_{db})] \quad (7-44)$$

- Delete Eq. 7.49 and replace with the following:

$$\phi_p = \left(5 \left(\frac{V_i - V_m}{V_i - V_f} \right) + 2 \right) \phi_y$$

- Delete Eq. 7.51 and replace with the following:

$$\phi_p = \left(4 \left(\frac{V_{ji} - V_{jh}}{V_{ji} - V_{jf}} \right) + 2 \right) \phi_y$$

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.

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.

C. Earthquake Restrainers

Longitudinal restrainers shall be high strength steel rods conform to ASTM F 1554 Grade 105, including Supplement Requirements S2, S3 and S5. Nuts, and couplers if required, shall conform to ASTM A 563 Grade DH. Washers shall conform to AASHTO M 293. High strength steel rods and associated couplers, nuts and washers shall be galvanized after fabrication in accordance with AASHTO M 232. The length of longitudinal restrainers shall be less than 24 feet.

D. 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 AASHTO LRFD, AASHTO SEISMIC, WSDOT Special Provisions and the WSDOT [Standard Specifications](#).

15.5.2 Materials

A. Concrete

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

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_f) shall be taken as 1.0 unless project specific data is available.

3. Shrinkage and Creep

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

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.

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.

6. Shotcrete

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

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.](#)

B. Reinforcing Steel

1. Grades

Steel reinforcing bars shall conform to [Section 5.1.2](#) and Section 15.4.2.T.

2. Compressive Development Length

The minimum compressive development length shall be 1'-0".

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.

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.

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.

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.

15.5.3 Design Considerations

A. Service and Fatigue Limit States

The exposure factor for AASHTO LRFD 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](#).

B. Strength Limit State

The shear design of prestressed members shall be based on the general procedure of AASHTO LRFD 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 AASHTO LRFD Section 5.7.3.4.1. AASHTO LRFD 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.

C. Post-Tensioning

A 2" minimum clearance shall be provided between post-tensioning ducts with outer diameter greater than 3". A 1" minimum clearance shall be provided between post-tensioning ducts with outer diameter less than or equal to 3".

Confinement reinforcement shall be provided to confine curved post-tensioning tendons in accordance with [Section 5.8.1.F](#).

The size of the anchorage zone in the plane of anchor plates shall be large enough to provide a minimum of 1" clearance from the plates to any free edge.

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 shall 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

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.

B. Box Girder Superstructures

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 $600 \text{ feet} \leq R < 800 \text{ feet}$ - at midspan.

For $400 \text{ feet} \leq R < 600 \text{ feet}$ - at $\frac{1}{3}$ points of span.

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

2. Temperature Effects

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

- a. A mean temperature 50°F with rise 45°F and fall 45°F for longitudinal analysis using one-half of the modulus of elasticity (Maximum Seasonal Variation.)
- b. The superstructure box girder shall be designed transversely for a temperature differential between inside and outside surfaces of $\pm 15^{\circ}\text{F}$ with no reduction in modulus of elasticity (Maximum Daily Variation).
- c. The superstructure box girder shall be designed longitudinally for a top slab temperature increase of 20°F with no reduction in modulus of elasticity.

3. Drains

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

C. Prestressed Concrete Girder Superstructures

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](#).

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.

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.

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 AASHTO LRFD Section 5.9.4.4.1. The end vertical reinforcement shall not be larger than #5 bars and spacing shall not be less than $2\frac{1}{2}$ ". The remaining splitting reinforcement not fitting within the $h/4$ zone may be placed beyond the $h/4$ zone at a spacing of $2\frac{1}{2}$ ".

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](#).

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.

7. Composite Action

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

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.

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 5.6.2H.

10. Transformed Section Properties

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

11. Deck Shrinkage

The elastic gain in prestressing strands due to slab shrinkage shall be computed in accordance with AASHTO LRFD 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 AASHTO LRFD Equation 5.4.2.3.3-1.

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.

13. Slab Girders

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

14. Deck Bulb-Tee Girders

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

15. Wide Flange Deck Girders

Wide flange deck girders shall be limited to pedestrian bridges, temporary bridges and to widening existing similar structures.

Wide flange deck girders with mechanical connections shall have an HMA or concrete overlay. A waterproofing membrane shall be provided with an HMA overlay. Wide flange deck girders with UHPC connections shall have a 1½" concrete overlay.

16. Wide Flange Thin Deck Girders

Wide flange deck girders shall be limited to pedestrian bridges, temporary bridges and to widening existing similar structures.

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.

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.

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 AASHTO LRFD 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.

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.

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 AASHTO LRFD Section 9.7.2.3, shall not exceed the deck thickness.

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.

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](#).

E. Bridge Deck HMA Paving

Asphalt resurfacing including bituminous surface treatment (BST) on bridge decks and slab bridges shall be in accordance with the Bridge deck Condition Report (BCR) provided for each bridge. Construction shall be in accordance with the bridge paving specifications included in the RFP.

15.6 Steel Structures

15.6.1 Design Considerations

A. Codes, Specification, and Standards

Steel highway bridges shall be designed to the following codes and specifications:

- AASHTO LRFD, latest edition
- AASHTO SEISMIC

The following codes and specifications shall govern steel bridge construction:

- WSDOT [Standard Specifications](#), latest edition
- AASHTO/AWS D1.5M/D1.5: *Bridge Welding Code*, latest edition

B. WSDOT Steel Bridge Practice

Unshored composite construction is used for plate girder and box girder bridges. Shear connectors shall be placed throughout positive and negative moment regions, for full composite behavior. A minimum of one percent longitudinal deck steel, in accordance with AASHTO LRFD Article 6.10.1.7, shall be placed in negative moment regions. For service level stiffness analysis, such as calculating live load moment envelopes, the bridge deck shall be considered composite and uncracked for the entire bridge length. For negative moment at strength limit states, the bridge deck shall be ignored while reinforcing steel is included for stress and section property calculations.

Stiffeners used to connect cross frames shall be welded to top and bottom flanges. Jacking stiffeners shall be used adjacent to bearing stiffeners, on girder or diaphragm webs, in order to accommodate future bearing replacement. Coordinate jack placement in substructure and girder details.

Steel framing shall consist of main girders and cross frames. Bottom lateral systems shall only be used when required for torsional stability in curved bridges and when temporary bracing is not practical. When lateral systems are used, they shall be detailed carefully for adequate fatigue life.

Standard corrosion protection for steel bridges is the *Standard Specifications Section 6-07* four-coat paint system west of the Cascades and where paint is required for appearance. Unpainted weathering steel shall only be used east of the Cascades.

WSDOT does not allow the use of steel stay-in-place deck forms.

C. Preliminary Girder Proportioning

Live load deflections shall be limited in accordance with the optional criteria of AASHTO LRFD Articles 2.5.2.6.2 and 3.6.1.3.2.

The superstructure depth shall be shown as the distance from the top of the bridge deck to the bottom of the web. On straight bridges, interior and exterior girders shall be designed and detailed as identical. Spacing should be such that the distribution of wheel loads on the exterior girder is close to that of the interior girder. The number of girder lines should be minimized, with a maximum spacing of 14-16 feet. Steel bridges shall be redundant, with three or more girders lines for I girders and two or more boxes for box girders, except as otherwise allowed by the RFP Criteria.

D. Bridge Steels

Use AASHTO M 270/ASTM A 709 grades 50 or 50W for plate girders and box girders. Use of AASHTO M 270/ASTM A 709 Grade HPS 70W is permissible but availability is limited and shall be confirmed prior to actual use in design. AASHTO M 270 grades HPS 50W and HPS100W may only be used if allowed by the RFP Criteria. For wide flange beams, use AASHTO M 270/ASTM A 709 Grade 50S or ASTM A 992. For ancillary members such as expansion joint headers, utility brackets, bearing components or small quantities of tees, channels, and angles, ASTM M 270/ASTM A 709 bridge steels are acceptable but are not required. In these cases, equivalent ASTM designated steels may be used.

All main load-carrying members or components subject to tensile stress shall be identified in the plans and shall meet the minimum Charpy V-notch (CVN) fracture toughness values as specified in AASHTO LRFD Table 6.6.2-2, temperature zone 2. Fracture critical members or components shall also be designated in the plans.

Structural tubes and pipes are not considered prequalified under the *Bridge Welding Code*. They are covered under the Structural Welding Code AWS D1.1. Structural tubing ASTM A 500 shall not be used for dynamic loading applications. ASTM A 1085 is a newer cold formed and welded HSS section specification that is a Gr 50 steel. Supplements for heat treating and CVNs are included and may also be specified. CVN tests are typically performed in the flats of the HSS square or rectangular tube sections. CVN values in the bend radius of the tubes may be lower than values obtained in the flats, **however recent informational testing has shown the CVN values in the corners meet or exceeds the requirements for Fractured Critical Member steels in temperature zone 2.** Heat treating of the sections can improve the values, but no data is currently available. ASTM A 1085 shall not be specified for dynamic loading applications **until further data is available.** Availability and minimum tonnage or bundle quantity orders shall be investigated prior to specifying ASTM A 1085.

E. Plate Sizes

Plate thicknesses of less than $\frac{5}{16}$ inches shall not be used for bridge applications.

F. Fasteners

All bolted connections shall be friction type (slip-critical). Assume Class B faying surfaces where inorganic zinc primer is used.

General Guidelines for Steel Bolts**A. ASTM F 3125 GR A325 & GR F1852**

High strength steel, headed bolts for use in structural joints. These bolts may be hot-dip galvanized in accordance with ASTM F2329 or mechanically galvanized in accordance with ASTM B695 – Class 55. Do not specify for anchor bolts. Galvanized GR F1852 “Twist-Off” style bolts are not permitted on WSDOT structures.

B. A449

High strength steel bolts and studs for general applications including anchor bolts. Recommended for use where strengths equivalent to ASTM F3125 GR A325 bolts (up to 1” diameter) are desired but custom geometry or lengths are required. These bolts may be hot-dip galvanized. Do not use these as anchor bolts for seismic applications due to low CVN impact toughness.

C. F1554 - Grade 105

Higher strength anchor bolts to be used for larger sizes (1½” to 4”). When used in seismic applications, specify supplemental CVN requirement S4 with a test temperature of -20°F. Lower grades may also be suitable for sign structure foundations. This specification shall be used for seismic restrainer rods, and may be galvanized. The equivalent AASHTO M 314 shall not be specified as it does not include the CVN supplemental requirements.

D. ASTM F3125 GR A490 & GR F2280

High strength alloy steel, headed bolts for use in structural joints. These bolts shall not be hot dip galvanized, because of the high susceptibility to hydrogen embrittlement. In lieu of galvanizing, the application of an approved zinc rich paint may be specified. Other coating applications are available and are specified in ASTM F3125. Alternate coatings shall be approved by the WSDOT Bridge Design Engineer prior to their use. Do not specify for anchor bolts. Only uncoated GR F2280 “Twist-Off” bolts are permitted on WSDOT structures.

E. A354 - Grade BD

High strength alloy steel bolts and studs. These are suitable for anchor bolts where strengths equal to ASTM F 3125 GR A490 bolts are desired. These bolts shall not be hot dip galvanized. If used in seismic applications, specify minimum CVN toughness of 25 feet-lb at 40°F.

15.6.2 Girder Bridges**A. Tub or Box Girders**

Steel box girders shall be trapezoidal tub sections. Tub girders will be referred to herein as box girders, as in AASHTO LRFD Article 6.11.

A top lateral system shall be placed inside the girder and shall be treated as an equivalent plate for design, closing the open section and providing torsional stiffness until the bridge deck is fully cured. Stability of the shape shall be ensured for all stages of construction in accordance with AASHTO LRFD Article 6.11.3. Box girder bridges with a multiple box girder cross section shall have a single bearing per box. For bridges of a single box girder cross section, two bearings per box shall be used. Plate diaphragms with access holes shall be used in place of pier cross frames.

With the exception of effects from inclined webs, top flanges and webs shall be designed as if they were part of individual I-girders.

In order to maximize web spacing while minimizing bottom flange width, place webs out of plumb on a slope of 1 in 4. When required to stiffen bottom flanges in compression tee sections shall be used for longitudinal stiffeners. Channel bracing may be used at cross frame locations for transverse stiffeners. Bottom flange stiffeners shall be terminated at field splices. Otherwise, carefully ground weld terminations are required in tension regions with high stress range. Stiffener plates shall be welded across the bottom flange at cross frame locations, and shall not be combined with web vertical stiffeners.

B. Fracture Critical Superstructures

Non-redundant, fracture critical single tub superstructures, and twin I-girder systems, may only be considered if allowed by RFP Criteria. UBIT access or some form of permanent false decking or other inspection access is required for fracture critical inspections. The maximum roadway width for either a single box or twin I-girder superstructure is 27 feet. Where roadway width exceeds 27 feet, additional webs/girders shall be used. Mainline structures exceeding 38 feet in width shall use four webs/girders minimum.

Increased vertical clearance from mainline traffic shall be provided for either of these bridge types. The minimum is 20 feet. The web depths may be reduced below AASHTO LRFD Table 2.5.2.6.3-1 minimums provided live load deflection criteria are met. However, web depth less than 5'-0" is not permitted. The limit state load modifier relating to redundancy, $\eta_r = 1.05$, as specified in AASHTO LRFD Section 1.3.4, shall be used in the design of non-redundant steel structures. For load rating

non-redundant bridges, a system factor of 0.85 is required in the AASHTO *Manual for Bridge Evaluation* (MBE) on the axial and flexural capacity of the girders. The girder design shall satisfy both the AASHTO LRFD code and the MBE.

The AASHTO LRFD approximate live load distribution factors are not applicable to these girder types. The level rule or the preferred refined analysis shall be used. Where highly curved, only a refined analysis shall be used.

Fracture-critical members and system redundant members shall be designed for infinite fatigue life, which requires design using the Fatigue I load combination specified in AASHTO LRFD Table 3.4.1-1 and the nominal fatigue resistance specified in Article 6.6.1.2.5.

15.6.3 Design of I-Girders

A. Limit States for AASHTO LRFD

The fatigue live load specified in AASHTO LRFD Article 3.6.1.4 shall be used for checking girder details in accordance with article 6.6. It is generally possible to meet the constant amplitude fatigue limit (CAFL) requirement for details with good fatigue performance. Limiting the calculated fatigue range to the CAFL ensures infinite fatigue life. Webs shall be checked for fatigue loading in accordance with AASHTO LRFD Article 6.10.5.3, using the calculated fatigue stress range for flexure or shear. Flanges and webs shall meet strength limit state requirements for both construction and final phases.

Pier cross frames shall be designed for seismic loading, extreme event load combination. Bolts shall be treated as bearing type connections with AASHTO LRFD Article 6.5.4.2 resistance factors. The resistance factor for all other members is 1.0 at extreme limit state.

B. Composite Section

Live load plus impact shall be applied to the transformed composite section using E_s/E_c , commonly denoted n . Long-term loading (dead load of barriers, signs, luminaries, overlays, etc.) shall be applied to the transformed composite section using $3n$. Positive moments are applied to these composite sections accordingly; both for service and strength limit states. The bridge deck may be considered effective in negative moment regions provided tensile stresses in the deck are below the modulus of rupture. This is generally possible for Service I load combination and fatigue analysis. For strength limit state loadings, the composite section includes longitudinal reinforcing while the bridge deck is ignored.

C. Flanges

The maximum flange thickness is 3-inches. All plates for flange material 2" or less shall be purchased such that the ratio of reduction of thickness from a slab to plate shall be at least 3.0:1.

Plates for flange material greater than 2" thick shall be supplied based on acceptable ultrasonic testing (UT) inspection in accordance with ASTM A 578. UT scanning and acceptance shall be as follows:

- The entire plate shall be scanned in accordance with ASTM A 578 and shall meet Acceptance Standard C, and
- Plate material within 12-inches of flange complete joint penetration splice welds shall be scanned in accordance with ASTM A 578 Supplementary S1 and shall meet Acceptance Standard C

D. Webs

If different web thickness is needed, the transition shall be at a welded splice.

Horizontal web splices shall not be used unless web height exceeds 12'-6".

All welded web splices on exterior faces of exterior girders and in tension zones elsewhere shall be ground smooth. Web splices of interior girders need not be ground in compression zones.

E. Transverse Stiffeners

Transverse stiffeners shall be used in pairs at cross frame locations on interior girders and on the inside of webs of exterior girders. They shall be welded to the top flange, bottom flange and web at these locations. This detail is considered fatigue category C' for longitudinal flange stress. Stiffeners used between cross frames shall be located on one side of the web, welded to the compression flange, and cut short of the tension flange. Stiffeners located between cross frames in regions of stress reversal shall be welded to one side of the web and cut short of both flanges. Alternatively, they may be welded to both flanges if fatigue Category C' is checked.

Stiffened webs require end panels to anchor the first tension field. The jacking stiffener to bearing stiffeners space shall not be used as the anchor panel. The first transverse stiffener shall be placed at no greater spacing than 1.5 times the web depth from the bearing or jacking stiffener.

F. Longitudinal Stiffeners

Longitudinal stiffeners may be used in long spans where web depths exceed 10 feet in accordance with AASHTO LRFD Article 6.10.11.3. Weld terminations for longitudinal stiffeners are fatigue prone details and shall be detailed accordingly. Longitudinal stiffener plates shall be continuous, splices being made with full penetration welds before being attached to webs. Transverse stiffeners shall be pieced to allow passage of longitudinal stiffeners.

G. Bearing Stiffeners

Bearing stiffeners shall be vertical under total dead load.

Pier cross frames may transfer large seismic lateral loads through top and bottom connections. Weld size shall be designed to ensure adequate load path from deck and cross frames into bearings.

H. Cross Frames

Cross frames and connections shall be detailed for repetitive fabrication, adjustment in the field, and openness for inspection and painting. Cross frames consisting of back-to-back angles separated by gusset plates are not permitted. Cross frames are generally patterned as K-frames or as X-frames. Oversize holes will not be allowed in cross frame connections if girders are curved.

Intermediate cross frames for straight girders with little or no skew shall be designed as secondary members. Member sizes shall be selected to meet minimum slenderness requirements and design connections only for anticipated loads, not for 75 percent strength of member.

Cross frames shall be installed parallel to piers for skew angles of 0 degrees to 20 degrees. For greater skew angles, other arrangements may be used. Cross frames for curved girder bridges are main load carrying members and tension components shall be so designated in the plans. Web stiffeners at cross frames shall be welded to top and bottom flanges.

I. Bottom Laterals

In accordance with 6.1.2, bottom lateral systems shall be avoided except when required for stability in curved bridges and shall be used only when temporary bracing is not practical.

Where lateral gusset plates are fillet welded to girder webs, the fatigue stress range in the girder is limited to Category E without transition radius or Category D with carefully made transition radius. The gusset plates shall be bolted to the girder web in regions of high tension stress range.

J. Bolted Field Splice for Girders

Field splices shall be bolted. Bolted web splices shall not involve thin fill material. Thickness transitions for webs, if needed, shall be done with welded shop splices.

Fillers used in bolted splices shall be developed as specified in AASHTO LRFD Article 6.13.6.1.4. Splice bolts shall be checked for Strength load combinations and slip at Service II load combination. When faying surfaces are blasted and primed with inorganic zinc paint, a Class B surface condition shall be assumed.

K. Camber

Camber shall include effects of profile grade, superelevation, anticipated dead load deflections, and bridge deck shrinkage (if measurable). Permanent girder deflections shall be shown in the plans in the form of camber diagrams and tables. Dead load deflections are due to steel self-weight, bridge deck dead load, and superimposed dead loads such as overlay, sidewalks, and barriers. A constant distance from top of web to top of bridge deck shall be assumed for design, however forms and haunch height shall be adjusted to meet bridge deck thickness and profile as specified in [Standard Specifications](#) Section 6-03.3(39).

Two camber curves are required, one for total dead load plus bridge deck formwork and one for steel framing self-weight. The difference between these curves is used to set bridge deck forms.

Girder self-weight shall include the basic section plus stiffeners, cross frames, welds, shear studs, etc. These items may be accounted for by adding an appropriate percentage of basic section weight. Total dead load camber shall consist of deflection due to:

1. Steel weight, applied to steel section. Include 10 psf bridge deck formwork allowance in the total dead load camber, but not in the steel weight camber. The effect of removing formwork is small in relation to first placement, due to composite action between girders and bridge deck. It isn't necessary to account for the removal.
2. Bridge deck weight, applied to steel section.
3. Traffic barriers, sidewalks, and overlays, applied to long-term composite section using 3n. Do not include weight of future overlays in the camber calculations.
4. Bridge deck shrinkage (if $\geq \frac{3}{4}$ ").

Traffic barriers, sidewalks, overlays, and other items constructed after the bridge deck placement shall be analyzed as if applied to the long-term composite section full length of the bridge. The modulus of elasticity of the bridge deck concrete shall be reduced to one third of its short term value.

For bridge deck shrinkage calculation, apply a shrinkage strain of 0.0002 to the long-term composite section using 3n.

In addition to girder deflections, girder rotations at bearing stiffeners shall be shown. Camber tolerance is governed by the *Bridge Welding Code* AWS D1.5, Chapter 3.5. A note of clarification shall be added to the plan camber diagram: "For the purpose of measuring camber tolerance during shop assembly, assume top flanges are embedded in concrete without a designed haunch." This allows a high or low deviation from the theoretical curve, otherwise no negative camber tolerance is allowed.

A screed adjustment diagram shall be included with the camber diagram. This diagram, with dimension table, shall be the remaining calculated deflection just prior to bridge deck placement, taking into account the estimated weight of deck formwork and deck reinforcing. The weight of bridge deck formwork may be taken equal to 10 psf, or the assumed formwork weight used to calculate total camber. The weight of reinforcing may be taken as the span average distributed uniformly. The screed adjustment should equal: (Total Camber – Steel Camber) - (deflection due to forms + rebar). The screed adjustment shall be shown at each girder line. This will indicate how much deflection and twisting is anticipated during bridge deck placement, primarily due to span curvature and/or skew. These adjustments shall be applied to theoretical profile grades, regardless of actual steel framing elevations. The adjustments shall be designated "C". The diagram shall be designated as "Screed Setting Adjustment Diagram." The table of dimensions shall be kept separate from

the girder camber, but at consistent locations along girders. That is, at 1/10th points or panel points. A cross section view shall be included with curved span bridges, showing effects of twisting.

For the purpose of setting bridge deck soffit elevations, a correction shall be made to the plan haunch dimension based on the difference between theoretical flange locations and actual profiled elevations. The presence of bridge deck formwork shall be noted at the time of the survey. The presence of false decking need not be accounted for in design or the survey.

L. Bridge Deck Placement Sequence

The bridge deck shall be placed in a prescribed sequence allowing the concrete in each segment to shrink with minor influence on other segments. Negative moment regions (segments over interior piers) shall be placed after positive moment regions have had time to cure. Successive segments shall not be placed until previous segments attain sufficient strength. The designer shall check bridge deck tensile stresses imposed on adjoining span segments.

M. Bridge Bearings for Steel Girders

Make bearing selection consistent with required motions and capacities.

N. Surface Roughness and Hardness

The standard measure of surface roughness is the microinch value. Surface roughness shall be shown on the plans for all surfaces for which machining is required unless covered by the [Standard Specifications](#) or Special Provisions. Surface hardness of thermal cut girder flanges is also controlled.

O. Welding

The minimum fillet weld size shall be as shown in the following table. Weld size is determined by the thicker of the two parts joined unless a larger size is required by calculated stress. The weld size need not exceed the thickness of the thinner part joined.

Base Metal Thickness of Thicker Part Joined	Minimum Size of Fillet Weld
To $\frac{3}{4}$ " inclusive	$\frac{1}{4}$ "
Over $\frac{3}{4}$ "	$\frac{5}{16}$ "

P. Shop Assembly

For straight girders, a progressive longitudinal shop assembly shall be performed to ensure proper fit of subsections, field splices, and cross frame connections, etc., in the field. Progressive transverse assembly, in combination with progressive longitudinal assembly shall be performed for bridges with horizontal curvature or skews greater than 20-degrees. For transverse assembly, specify all cross frame and pier diaphragm connections to be completed while assembled.

During shop assembly, girder segments shall be blocked or supported in the no-load condition (no gravity effects). For curved I-girders, cross frames shall be fabricated to fit the no-load condition. Design of cross frames and pier diaphragms shall take into account twist and rotations of webs during construction. This situation should be carefully studied by finite element analysis to determine amount and type of movement anticipated during construction. Unlike curved girders rotating away from plumb at midspan, girder webs for skewed construction shall be kept plumb at piers.

For bridges with skews greater than 20-degrees, the fit of girder segments, cross frames, and pier diaphragms shall be carefully analyzed during design to select the proper fit condition, which shall be either the no-load condition of the steel dead load condition. The detailing, fabrication, and shop assembly shall be specified to match the condition used in the analysis and design.

15.6.4 Plan Details

A. General

Detailing practice shall follow industry standards. Designations for structural steel can be found in *AISC Detailing for Steel Construction*. Detailing shall also conform to national unified guidelines published by *AASHTO/NSBA Steel Bridge Collaboration*.

Connections in the field shall be bolted. Cross frame members may be shop bolted or welded assemblies and shall be shipped to the field in one unit. Connections of bolted cross frame assemblies shall be fully tensioned prior to shipping. Cross frame assemblies shall be field bolted to girders during erection.

B. Framing Plan

The Framing Plan shall show ties between the survey line, girder lines, backs of pavement seats, and centerlines of piers. Locate panel points (cross frame locations). Provide geometry, bearing lines, and transverse intermediate stiffener locations. Show field splice locations. Map out different lateral connection details.

C. Girder Elevation

The Girder Elevation is used to define flanges, webs, and their splice locations. Show shear connector spacing, location, and number across the flange. Show shear connector locations on flange splice plates or specifically call out when no connectors are required on splice plates. Locate transverse stiffeners and show where they are cut short of tension flanges. Show the tension regions of the girders with a V for the purpose of ordering plate material, inspection methods (NDE), and *Bridge Welding Code* acceptance criteria. Identify tension welded butt splices for which radiographic examination (RT) is required. Permissible welded web splices shall be shown. If there are fracture critical components, they shall be clearly identified as FCM. **If a member is identified as fracture critical with an "FCM" symbol, it is not necessary to also call the plate or member with a "V" for Charpy-V-Notch as this is covered with the "FCM" designation.**

D. Typical Girder Details

Specific sheets shall be devoted to showing typical details to be used throughout the girders. Such details include the weld details, various stiffener plates and weld connections, locations of optional web splices, and drip plate details. Field splices for flanges shall accommodate web location tolerance of $\pm \frac{1}{4}$ " in accordance with the AWS Bridge Welding Code 3.5.1.5. Allow a minimum of $\frac{1}{4}$ " for out of position web plus $\frac{3}{8}$ " for fillet weld, or a total of $\frac{5}{8}$ " minimum clear between theoretical face of web and edge of splice plate. The bottom flange splice plate shall be split to allow moisture to drain (use 4 equal bottom flange splice plates). The fill plate does not need to be split.

Vertical stiffeners used to connect cross frames shall be welded to top and bottom flanges to reduce out-of-plane bending of the web. All stiffeners shall be coped, clipped (or cut short in the case of transverse stiffeners without cross frames) a distance between $4t_w$ and $6t_w$ to provide web flexibility, in accordance with AASHTO LRFD Article 6.10.11.1.1.

F. Cross Frame Details

Show member sizes, geometrics (work lines and work points), and connection details. Double angles shall not be used for cross frames. Cross frames shall be complete subassemblies for field installation.

Internal cross frames and top lateral systems for box girders are shop welded, primarily. All connection types shall be closely examined for detail conflict and weld access. Clearance between bridge deck forming and top lateral members shall be considered.

G. Camber Diagram and Bearing Stiffener Rotation

Camber curves shall be detailed to provide dimensions at tenth points. Dimensions may also be given at cross frame locations. In order to place bearing stiffeners in the vertical position after bridge deck placement, show expected girder rotations at piers.

Show deflection camber only. Geometric camber for profile grade and superelevation will be calculated by the shop detailer from highway alignment shown on the Layout sheets.

A separate diagram and table, with bridge cross section, shall be included to show how elevations at edges of deck can be determined just before concrete placement. This will give adjustments to add to profile grades, based on remaining dead load deflections, with deck formwork and reinforcing being present.

H. Bridge Deck

New bridge decks for steel I-girders or box girders shall use Deck Protection System 1.

The bridge deck shall be detailed in section and plan views. For continuous spans, show a section showing negative moment longitudinal reinforcing. The plan views shall detail typical reinforcing and cutoff locations for negative moment steel. Negative moment steel shall not be terminated at one location.

The pad dimension is assumed to be constant throughout the span length. Ideally, the girder is cambered to compensate for dead loads and vertical curves. However, fabrication and erection tolerances result in considerable deviation from theoretical elevations. The pad dimension is therefore considered only a nominal value and is adjusted as needed along the span once the steel has been erected and profiled. The screed for the slab is to be set to produce correct roadway profile. The plans shall reference this procedure contained in *Standard Specifications* Section 6-03.3(39). The pad dimension is to be noted as nominal.

I. Handrail Details, Inspection Lighting, and Access

When required by the RFP Criteria, include handrails with typical girder details. Locations may be adjusted to avoid conflicts with other details such as large gusset plates. Box girders require special consideration for inspection access. Access holes or hatches shall be detailed to exclude birds and the public. They shall be positioned where ladders can be used to gain access. Locate hatches in girder webs at abutments. Provide for round trip access and penetrations at all intermediate diaphragms. Access for removing bridge deck formwork shall be provided. Box girders shall have electrical, inspection lighting, and ventilation details for the aid of inspection and maintenance. Refer to the [Design Manual Chapter 1040](#) for bridge inspection lighting requirements.

To facilitate inspection, interior paint shall be **SAE AMS** Standard 595 color number 17925 (white). One-way inspection of all interior spaces shall be made possible by round trip in adjoining girders. This requires some form of walkway between boxes and hatch operation from both sides. If locks are needed, they must be keyed to one master. Air vents shall be placed along girder webs to allow fresh air to circulate.

J. Box Girder Details

Provide a top lateral system in each box, full length of a girder.

The top laterals shall be bolted directly to the top flange or intermediate bolted gusset plate (in which case, the lateral members may be welded to the gusset plate). In order to maximize the clearance for deck forms, all lateral connections shall progress down from the bottom surface of the top flange. The haunch distance between top of web and deck soffit shall be 6" or greater to allow deck forming to clear top lateral members.

To facilitate continuous welding of the bottom flange to webs, the stiffeners shall be held back and attached to the bottom flange by a member brought in after the bottom longitudinal welds are complete.

The offset between center of web and edge of bottom flange shall be 2"

Use tee shapes, either singly or in pairs, for stiffening wide bottom flanges.

Box girder inside clear height shall be 5 feet or more to provide reasonable inspection access. Less than 5 feet inside clear height is not be permitted.

Drain holes shall be installed at all low points.

Geometrics for boxes shall be referenced to a single workline, unless box width tapers. The box cross section remains tied to a centerline intersecting this workline and normal to the bridge deck. The section rotates with superelevation transition rather than warping.

15.6.5 *Painting of Existing Steel Bridges*

Refer to [Section 6.6](#) for requirements and procedures associated with painting of existing steel bridges.

15.6.6 *Corrosion of Steel Foundations and Buried Structures*

Refer to [Section 6.7](#) for corrosion and abrasion protection requirements to ensure a minimum 75-year design life for steel foundation elements and metal plate buried structures.

15.7 Substructure Design

15.7.1 General Substructure Considerations

A. Foundation Seals

The bottom of seal, if used, shall be no higher than the scour depth elevation.

B. Scour

The design flood local scour depth (100 year event) shall be included in the Strength I limit state analysis, and the check flood local scour depth (500 year event) shall be included in the Extreme Event II limit state analysis. When including the effects of scour, the design shall consider the loss of overburden and foundation side resistance.

Where lateral stream migration is a possibility, the design shall include a reliability-based estimate of the effects on the structure.

If stream migration risk is classified as anything other than “Low” in the Preliminary Hydraulic Design Report, then the bottom of any footing, pile cap, or shaft cap shall be 2 feet minimum below the 500-year elevation for any new permanent structure design.

Additional requirements from 7.1.7 shall also be followed.

C. Combination of Extreme Event Effects

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.

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.

3. Scour

The effects of local scour shall be combined with earthquake loading. At the Extreme Event I limit state, the design shall consider a scour depth equal to 25 percent of the design flood scour depth.

15.7.2 Foundation Modeling for Seismic Loads

A. General

Bridge modeling for seismic events shall be in accordance with requirements of the AASHTO SEISMIC Section 5.

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

B. Bridge Model Section Properties

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

1. Cracked Properties for Columns

Effective section properties shall be in accordance with the AASHTO SEISMIC Section 5.6.

2. Shaft Properties

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

For a stiff substructure response:

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

For a soft substructure response:

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

3. Cast-in-Place Pile Properties

For a stiff substructure response:

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

For a soft substructure response:

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

C. Spread Footing Modeling Methods

The method for calculating footing springs is given in [Section 7.2.7](#).

D. Deep Foundation Modeling Methods

The method used to model deep foundations shall conform to AASHTO SEISMIC Section 5.3.

1. Group Effects

The reduction factors for lateral resistance due to the interaction of deep foundation members is provided in AASHTO SEISMIC Section 8.12.2.5.

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.

E. Design of Deep Foundations for Lateral Forces

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.

2. Design for Lateral Loads

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

- a. Static soil properties with both stiff and soft shaft or pile properties.
- b. Dynamic or degraded soil properties with both stiff and soft shaft or pile properties.
- c. 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.
- d. Scour condition with stiff and soft shaft or pile properties.

15.7.3 Column Design

A. Shear Design

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

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 AASHTO SEISMIC prior to use of column silos. Column silos shall meet the requirements of [Section 7.3.4](#).

C. Longitudinal Reinforcement

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

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

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

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.

E. Longitudinal Reinforcement Development

1. Crossbeams

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

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 AASHTO SEISMIC Section 8.8.4 and AASHTO LRFD Section 5.10.8.2.1.

3. Drilled Shafts

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

The modification factor in AASHTO LRFD Section 5.10.8.2.1 that allows l_d to be decreased by the ratio of $(A_s \text{ required})/(A_s \text{ provided})$, shall not be used.

F. Transverse Reinforcement

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.

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.

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

A. General

1. Bent-Type and Isolated Abutments

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

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):

- WSDOT [Geotechnical Design Manual](#) Section 15.5.3.5
- AASHTO LRFD
- *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.

B. Embankment and Backfill

1. General Clearances

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

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

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.

3. Drainage and Backfill

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

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

C. Abutment Loading

1. Earthquake Load

EQ

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

2. Bearing Forces

TU

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

D. Abutment Details

1. Bearing Seats

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

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 $\phi_s = 0.9$.

3. Abutment Walls

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

15.7.6 Abutment Wing Walls and Curtain Walls

Wall footing thickness shall not be less than 1'-6"

15.7.7 Footing Design

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.

B. Spread Footing Design

1. Foundation Design

a. Bearing Stress

The maximum effective width for calculating uniform bearing stress is limited to $C+2D$ as shown in [Figure 7.7.4-3](#).

2. Structural Design

a. Footing Thickness and Shear

The minimum footing thickness shall be 1'-0". The minimum plan dimension shall be 4'-0".

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

c. Bottom Reinforcement

Reinforcement shall not be less than #6 bars at 12" centers to account for uneven soil conditions and shrinkage stresses.

d. 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.

C. Pile-Supported Footing Design

The minimum footing thickness shall be 2'-0". The minimum plan dimension shall be 4'-0".

1. Pile Embedment, Clearance, and Rebar Mat Location

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

2. Concrete Design

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

15.7.8 Shafts**A. Axial Resistance****1. Axial Resistance Group Reduction Factors**

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

2. 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.

B. Structural Design and Detailing

1. For shaft foundation supporting columns in SDC C or D, the shaft nominal moment capacity shall be designed to resist 1.25 times the moment demand generated in the shaft by the overstrength column plastic hinge moment at the base of the column.
2. Concrete Class 5000P shall be specified for the entire length of the shaft for wet or dry conditions of placement.
3. When shafts are constructed in water, the concrete specified for the casing shoring seal shall be Class 4000W.
4. The assumed concrete compressive strength may be taken as $0.85f'_c$ for structural design of shafts. For seismic design, the expected compressive strength may be increased by 1.3 in accordance with AASHTO Seismic Section 8.4.4.
5. The presence of permanent steel casing shall be taken into account in the shaft design (i.e. for stiffness, and etc.), but the structural capacity of permanent steel casing shall not be considered for structural design of drilled shafts unless the design conforms to Section 15.7.10.
6. Minimum cover requirements shall be as specified below:
 - Diameter less than or equal to 3'-0" = 3"
 - Diameter greater than 3'-0" and less than 5'-0" = 4"
 - Diameter greater than or equal to 5'-0" = 6"
7. The clear spacing between spirals and hoops shall not be less than 6" or more than 9". The clear spacing between spirals or hoops may be reduced in the splice zone in single column/single shaft connections if the concrete is vibrated.
8. The volumetric ratio and spacing requirements of the AASHTO SEISMIC for confinement need not be met.
9. #7 through #9 welded lap spliced hoops are acceptable to use provided they are not located in possible plastic hinge regions. Welded splices in hoops for shafts shall be completed prior to assembly of the shaft steel reinforcing cage. When hoops are used, the plans shall show a staggered splice pattern around the perimeter of the shaft so that no two adjacent splices are located at the same location.
10. In single column/single shaft configurations, the spacing of the shaft transverse reinforcement in the splice zone shall meet the requirements of the TRAC Report titled, "Noncontact Lap Splices in Bridge Column-Shaft Connections". The factor k represents the ratio of column tensile reinforcement to total column reinforcement at the nominal resistance. In the upper half of the splice zone, k shall be taken as 1.0. In the lower half of the splice zone, this ration could be determined from a column moment-curvature analysis.

11. Longitudinal reinforcement shall be provided for the full length of drilled shafts. The minimum longitudinal reinforcement in the splice zone of single column/single shaft connections shall be the larger of 0.75 percent A_g of the shaft or 1.0 percent A_g of the attached column. The minimum longitudinal reinforcement beyond the splice zone shall be 0.75 percent A_g of the shaft. The minimum longitudinal reinforcement in shafts without single column/single shaft connections shall be 0.75 percent A_g of the shaft.
12. The clear spacing between longitudinal reinforcement shall not be less than 6" or more than 9". If a shaft design is unable to meet this minimum requirement, a larger diameter shaft shall be considered.
13. Mechanical splices in longitudinal bars shall be placed in low stress regions and staggered 2'-0" minimum.
14. Where undersized permanent slip casing is used, provide a minimum of concrete cover of 3" for shafts with a diameter of 4'-0" and larger and 1½" for shafts with a diameter less than 4'-0".
15. Reinforcing bar centralizers shall be detailed in the plans as shown in Section 7.8.2-4.

15.7.9 Piles and Piling

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.

B. Pile Groups

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

C. Battered Piles

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

D. Structural Design and Detailing of CIP Concrete Piles

1. Concrete Class 5000P shall be specified for CIP concrete piles. The top 10' of concrete in the pile shall be vibrated.
2. For structural design, the reinforcement alone shall be designed to resist the total moment throughout the length of pile without considering strength of the steel casing. The minimum reinforcement shall be 0.75 percent A_g for SDC B, C and D and shall be provided for the full length of the pile. Minimum clearance between longitudinal bars shall meet the requirements in [Appendix 5.1-A2](#).

3. If the pile to footing/cap connection is not a plastic hinge zone longitudinal reinforcement need only extend above the pile into the footing/cap a distance equal to $1.0 l_d$ (tension). If the pile to footing/cap connection is a plastic hinge zone longitudinal reinforcement shall extend above the pile into the footing/cap a distance equal to $1.25 l_d$.
4. Transverse spiral reinforcement shall be designed to resist the maximum shear in the pile. The minimum spiral shall be a #4 bar at 9" pitch. If the pile to footing/cap connection is not a plastic hinge zone the volumetric requirements of AASHTO LRFD Section 5.11.4.5 need not be met.

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.

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

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

A. General

Design of retaining walls shall be based on the requirements and guidance cited herein and in the current AASHTO LRFD, AASHTO SEISMIC, AASHTO LRFD *Bridge Construction Specifications*, WSDOT General & Bridge Special Provisions and the [Standard Specifications](#) M 41-10 unless otherwise cited herein.

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

B. Loads

Retaining walls and their components shall be designed for all applicable loads defined in the current AASHTO LRFD Chapter 3.

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

C. Design of Reinforced Concrete Cantilever Retaining Walls

1. Standard Plan Reinforced Concrete Cantilever Retaining Walls

The standard plan reinforced concrete retaining walls have been designed in accordance with the requirements of the AASHTO LRFD, 4th Edition, 2007, and interims through 2008. See [Section 8.1](#) for a complete list of the design criteria used for these walls. Details for construction and the maximum bearing pressure in the soil are given in the [Standard Plans](#) Section D.

2. Non-Standard Reinforced Concrete Retaining Walls

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

For additional design criteria, refer to Section 8.1.4B

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

Typical soldier pile wall details are provided in the [Appendix 8.1-A1](#).

1. Ground Anchors (Tiebacks)

Either the “tributary area method” or the “hinge method” as outlined in AASHTO LRFD Section C11.9.5.1 shall be considered acceptable design procedures to determine the horizontal anchor design force.

The recommended factored design load of the anchor, recommended anchor installation angles (typically 10° – 45°), no load zone dimensions, and any other special requirements for wall stability shall be as provided by the geotechnical investigation performed for the project by the Geotechnical Engineer of record, and the associated geotechnical report based on that investigation.

The minimum vertical anchor spacing shall be the same as defined in AASHTO LRFD Section 11.9.4.2 for the minimum horizontal anchor spacing.

The anchor lock-off load is 60 percent of the controlling factored design load for temporary and permanent walls (see [Geotechnical Design Manual Chapter 15](#)).

Permanent ground anchors shall have double corrosion protection consisting of an encapsulation-protected tendon bond length as specified in the WSDOT General Special Provisions. Typical permanent ground anchor details are provided in the [Appendix 8.1-A1](#).

Temporary ground anchors may have either double corrosion protection consisting of an encapsulation-protected tendon bond length or simple corrosion protection consisting of grout-protected tendon bond length.

2. Design of Soldier Pile

Refer to Section 8.1.5B for design criteria

3. Design of Lagging

If construction operations are likely to occur above and behind the soldier pile wall alignment, the lagging shall be designed for an additional 250 psf surcharge due to temporary construction load.

a. Temporary Timber Lagging

Temporary lagging is as defined in the [Standard Specifications](#) Section 616.3(6). Temporary timber lagging shall be designed in accordance with [Standard Specifications](#) Section 616.3(6)B.

b. Permanent Lagging

Permanent lagging is as defined in the [Standard Specifications](#) Section 6.16.3(6). Permanent lagging shall be designed for 100 percent of the lateral load that could occur during the life of the wall in accordance with AASHTO LRFD Sections 11.8.5.2 and 11.8.6 for simple spans without soil arching.

Timber lagging shall be designed in accordance with AASHTO LRFD Section 8.6. The size effect factor (CF_b) shall be considered 1.0 unless a specific size is shown in the wall plans. The wet service factor (CM_b) shall be 0.85. The load applied to lagging shall be applied at the critical depth. Lagging size may be stepped over the height of the wall.

Timber lagging designed as a permanent structural element shall consist of treated Douglas Fir-Larch, grade No. 2 or better. Hem-fir wood species, due to the inadequate durability in wet condition, shall not be used for permanent timber lagging.

4. Design of Fascia Panels

Refer to Section 8.1.5D for design criteria.

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

E. Design of Structural Earth Walls**1. Pre-approved Proprietary Structural Earth Walls**

Structural earth (SE) wall systems meeting established WSDOT design and performance criteria have been listed as “preapproved” by the Bridge and Structures Office and the Materials Laboratory Geotechnical Branch. A list of current preapproved proprietary wall systems and their limitations is provided in the [Geotechnical Design Manual](#) Appendix 15D. For the SE wall shop drawing review procedure, see the [Geotechnical Design Manual](#) Chapter 15.

Refer to Section 8.1.6 for additional design criteria.

F. Design of Standard Plan Geosynthetic Walls

Details for construction are given in the [Standard Plans](#) Section D.

G. Design of Soil Nail Walls

Soil nail walls shall be designed in accordance with the FHWA Publication FHWA-NHI-14-007 “[Geotechnical Engineering Circular No. 7 Soil Nail Walls](#)”. The seismic design parameters shall be determined in accordance with the most current edition of the AASHTO SEISMIC. Typical soil nail wall details are provided in [Section 8.1](#).

H. Scour of Retaining Walls

Refer to Section 8.1.9 for design criteria.

I. Miscellaneous Items

Refer to Section 8.1.10 for design criteria.

15.8.2 Noise Barrier Walls

A. General

Design of noise barrier walls shall be based on the requirements and guidance cited herein and in the current AASHTO LRFD, AASHTO SEISMIC, AASHTO LRFD *Bridge Construction Specifications*, WSDOT General & Bridge Special Provisions and the [Standard Specifications](#) M 41-10 unless otherwise cited herein.

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

B. Loads

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

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

Seismic load shall be as follows: See Section 8.2.2 for seismic loads.

C. Design

1. Standard Plan Noise Barrier Walls

Refer to Section 8.2.3.A for design criteria.

2. Non-Standard Noise Barrier Walls

Noise barrier walls containing design parameters which exceed those used in the standard noise barrier wall design are considered to be non-standard.

a. Noise Barrier Walls on Bridges and Retaining Walls

Refer to Section 8.2.3B1 for design criteria.

15.8.3 Buried Structures

A. General Policy

Cast-in-place or precast reinforced concrete, and metal structural plate are authorized materials for Buried Structures as defined in [Section 8.3](#). If a Design-Builder intends to use alternate materials, other than reinforced concrete or metal structural plate, they shall submit an Alternative Technical Concept.

All Buried Structures shall be designed for a minimum service life of 75 years.

The Structural Clear Span, Structure Class, and Fill Depth for Buried Structures shall be as defined in [Section 8.3.1](#).

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

B. General Design Requirements

The design of Buried Structures shall be in accordance with the requirements and guidance cited herein and in the current AASHTO LRFD, AASHTO SEISMIC, the WSDOT [Geotechnical Design Manual](#), and [Standard Specifications](#), unless otherwise required in the project-specific criteria.

All Class 2 Buried Structures shall be designed for seismic effects in accordance with [Section 8.3.3.H](#), and load rated in accordance with [Section 13](#). Seismic design shall not apply for Class 1 Buried Structures.

C. Application of Loads

Buried Structures shall be designed for force effects in accordance with AASHTO LRFD, [Section 12.6.1](#), except exemption from seismic loading shall not apply for Class 2 Buried Structures.

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

The effects of live load may be neglected for;

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

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

D. Scour

Buried Structures, wingwalls, headwalls, and respective foundations shall be designed for the effects of scour as described in Section 8.1.9.

E. Corrosion

Consideration shall be given to the degradation of Buried Structure materials resulting from corrosive conditions as defined in Section 6.7. For metal structural plate structures, minimum corrosion rates and design service life analysis shall be in accordance with Section 6.7.2. For concrete structures, corrosion resistant reinforcement as defined in Section 5.1.2 shall be used in Marine or Non-Marine: Corrosive environments. The minimum cover requirements for direct exposure to salt water and coastal situations of the AASHTO LRFD shall apply.

F. Fall Protection

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

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

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

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

For details see Standard Plan C-20.41 and the WSDOT [Design Manual](#) M 22-01.

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

H. Deflection

Concrete structures with less than 2.0 feet of Fill Depth shall mitigate differential deflection between adjacent units in accordance with Section 8.3.5.A.2.

I. Control of Cracking

Reinforcement provided in accordance with AASHTO LRFD, Section 5.6.7 shall be based upon a Class 2 exposure condition.

J. Joints

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

K. Soil Cover

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

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

L. Metal Structural Plate Structures

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

Steel structural plate shall not be used in locations conforming to Marine or Non-Marine: Corrosive environments as defined in Section 6.7.1 of this *Bridge Design Manual*.

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

Where aluminum will contact concrete or grout, two coats of paint shall be applied to the aluminum at the contact surface in accordance with *Standard Specifications*, Section 7-08.3(2)D.

M. Design of Detention Vaults

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

N. Design of Tunnels

Refer to Section 8.3.8 for design criteria specific to tunnels.

15.9 Bearings and Expansion Joints

15.9.1 Expansion Joints

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 in order 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.

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.

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

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:

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

2. Thermal Effects

Uniform thermal movement range shall be calculated using the maximum and minimum anticipated bridge superstructure average temperatures in accordance with AASHTO 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 AASHTO 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](#) M 41-10, 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. In order 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.

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.

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 9.1.3C](#) below.

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

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.

3. Asphaltic Plug Joints

Asphaltic plug joints are not allowed.

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.

D. Medium Movement Range Joints

1. Steel Sliding Plate Joints

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

- i) sidewalks and crosswalks
- ii) modular expansion joint upturns at traffic barriers
- iii) 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.

2. Strip Seal Joints

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

Either a standard anchorage or a special anchorage may be used for a strip seal expansion joint. The special anchorage incorporates steel reinforcement bar loops welded to intermittent steel plates, which in turn are welded to the

steel shape. The special anchorage shall be used for very high traffic volumes or applications subject to snowplow hits. In applications highly susceptible to snowplow hits and concomitant damage, the intermittent steel plates shall be detailed to protrude $\frac{1}{4}$ " 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.

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.

E. Large Movement Range Joints

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.

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.

a. Operational Characteristics

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

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

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

b. 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.

c. 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.

d. Construction Considerations

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

15.9.2 Bearings

A. General Considerations

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

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 AASHTO LRFD. Impact need not be applied to live load forces in the design of bearings.

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:

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.

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 AASHTO LRFD *Bridge Design Specifications*.

D. Detailing Considerations

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

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

E. Bearing Types

1. Elastomeric Bearings

Steel reinforced elastomeric bearings shall be designed using the AASHTO LRFD 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 251 - *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 ¼₁₆-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:

- i. For prestressed concrete wide flange girders (WF36G to WF100G), the edge of the bearing pad shall be set between 1 inch minimum and 9 inch maximum inside of the edge of the girder bottom flange.
- ii. 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.

- iii. 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.
- iv. For all prestressed concrete slabs one bearing pad and corresponding grout pad is required for each end of the prestressed concrete slab. The centerline of the bearing and grout pad shall coincide with the centerline of the prestressed concrete slab. The need for steel shims shall be assessed during the bearing design.

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

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

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

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

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

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

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.

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.

a. 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.

b. 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.

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.

4. Rocker and Roller Type Bearings

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

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.

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.

7. Seismic Isolation Bearings

Seismic isolation bearings may be used, subject to the restrictions outlined in Sections 4.2.2 and 9.3.4.

F. Miscellaneous Details

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:

a. 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.

b. 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.

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.

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.

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.

I. Bearing Replacement Considerations

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

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

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

15.10 Signs, Barriers, Bridge Approach Slabs, and Utilities

15.10.1 Sign and Luminaire Supports

A. Loads

1. General

The reference used in developing the following office criteria is the AASHTO "LRFD for Structural Supports for Highway Signs, Luminaires, and Traffic Signals," First Edition Dated 2015 (including latest interims), and shall be the basis for analysis and design.

2. Dead Loads

Sign:

(Including panel and wind beams; does not include vert. bracing) 3.25 lbs/ft²

Luminaire (effective projected area of head = 3.3 sq feet) 60 lbs/each

Fluorescent Lighting 3.0 lbs/ft

Standard Signal Head 60 lbs/each

Mercury Vapor Lighting 6.0 lbs/ft

Sign Brackets Calc.

Structural Members Calc.

5 foot wide maintenance walkway:

(including sign mounting brackets and handrail) 160 lbs

Signal Head w/3 lenses:

(effective projected area with backing plate = 9.2 sq feet) 60 lbs/each

3. Live Load

A live load consisting of a single load of 500 lb distributed over 2.0 feet transversely to the member shall be used for designing members for walkways and platforms. The load shall be applied at the most critical location where a worker or equipment could be placed, see 2015 AASHTO *Specifications* Section 3.6.

4. Wind Loads

A 3 second gust wind speed shall be used in the AASHTO wind pressure equation. The 3 second wind gust map in AASHTO is based on the wind map in ANSI/ASCE 7-16.

The basic wind speed of 115 mph shall be used in computing design wind pressure using Equation 3.8.1-1 of AASHTO Section 3.8.1. This is based on the high risk category with a mean recurrence interval of 1700 years per AASHTO Table 3.8-1.

The Alternate Method of Wind Pressures given in Appendix C of the AASHTO 2015 *Specifications* shall not be used.

5. Fatigue Design

Fatigue design shall conform to AASHTO Section 11 with the exception of square and rectangular tube shape. AASHTO does not provide fatigue calculations for shapes with less than 8 sides. Therefore, calculating the Constant Amplitude Fatigue Threshold, D_T (Table 11.9.3.1-2, AASHTO 2015) was taken to be the larger outer flat to flat distance of the rectangular tube. Fatigue Categories are listed in Table 11.6-1. Overhead Cantilever and Bridge Sign and signal structures, high-mast lighting towers (HMLT), poles, and bridge mounted sign brackets shall conform to the following fatigue categories.

Fatigue Category I: Overhead cantilever sign structures (maximum span of 35 feet and no VMS installation), overhead sign bridge structures, high-mast lighting towers 55 feet or taller in height, bridge-mounted sign brackets, and all signal bridges. Gantry or pole structures used to support sensitive electronic equipment (tolling, weigh-in-motion, transmitter/receiver antennas, transponders, etc.) shall be designed for Fatigue Category I, and shall also meet any deflection limitations imposed by the electronic equipment manufacturers.

Fatigue Category II: For structures not explicitly falling into Category I or III.

Fatigue Category III: Lighting poles 50 feet or less in height with rectangular or square cross sections, or non-tapered round cross sections, and overhead cantilever traffic signals (maximum cantilever length 65 feet).

Sign bridges, cantilever sign structures, signal bridges, and overhead cantilever traffic signals mounted on bridges shall be either attached to substructure elements (e.g., crossbeam extensions) or to the bridge superstructure at pier locations. Mounting these features to bridges as described above will help to avoid resonance concerns between the bridge structure and the signing or signal structure.

CCTV camera pole shall meet deflection criteria specified on Standard Plan J-29-15 for fixed base.

The “XYZ” limitation shown in [Table 10.1.4-2](#) shall be met for Monotube Cantilevers. The “XYZ” limitation consists of the product of the sign area (XY) and the arm from the centerline of the posts to the centerline of the sign (Z). See Appendix 10.1-A2-1 for details.

6. Ice and Snow Loads

A 3 psf ice load may be applied around all the surfaces of structural supports, horizontal members, and luminaires, but applied to only one face of sign panels (Section 3.7, AASHTO 2015).

Walk-through VMS shall not be installed in areas where appreciable snow loads may accumulate on top of the sign, unless positive steps are taken to prevent snow build-up.

7. Group Load Combinations

Sign, luminaire, and signal support structures are designed using the load factors from Table 10.1.1-1, AASHTO 2015 (including latest interims).

B. Bridge Mounted Signs

1. Vertical Clearance

All new signs mounted on bridge structures shall be positioned such that the bottom of the sign or lighting bracket does not extend below the bottom of the bridge as shown in [Figure 10.1.2-1](#).

Bridge mounted sign brackets shall be designed to account for the weight of added lights, and for the wind effects on the lights to ensure bracket adequacy if lighting is attached in the future.

2. Geometrics

- a. Signs shall be installed at approximate right angles to approaching motorists. For structures above a tangent section of roadway, signs shall be designed to provide a sign skew within 5 degrees from perpendicular to the lower roadway (see [Figure 10.1.2-2](#)).
- b. For structures located on or just beyond a horizontal curve of the lower roadway, signs shall be designed to provide a sign chord skew within 5 degrees from perpendicular to the chord-point determined by the approach speed (see [Figure 10.1.2-3](#)).
- c. The top of the sign shall be level.

3. Aesthetics

- a. The support structure shall not extend beyond the limits of the sign.
- b. The sign support shall be detailed in such a manner that will permit the sign and lighting bracket to be installed level.

4. Sign Placement

- a. Signs shall never be placed under bridge deck overhangs or directly under the dripline of the bridge.
- b. A minimum of 2 inches of clearance shall be provided between back side of the sign support and edge of the bridge, see [Figure 10.1.2-5](#).
- c. VMS units shall not be installed on bridges.
- d. Top arm lengths exceeding 7'-0", or any arm length with load demands exceeding the capacity of a 4" diameter standard pipe shall not be used.

5. Installation

- a. Adhesive anchors or cast-in-place ASTM F593 Type 304, Group 1 Condition CW anchor rods shall be used to install the sign brackets on the structure. Size and minimum installation depth shall be given in the plans or specifications. The adhesive anchors shall be installed normal to the concrete surface, and shall not be core drilled. Adhesive anchors shall not be placed through the webs or flanges of prestressed or post-tensioned girders. Adhesive anchors shall not be used at overhead locations other than with horizontal hole/anchor alignment.
- b. Bridge mounted sign structures shall not be placed on bridges with steel superstructures.

6. Installing/Replacing Sign Panels on Existing Bridge Mounted Sign Brackets

When installing a new sign panel on an existing bridge mounted sign bracket, the installation shall conform to the following.

- a. All hardware shall be replaced in accordance with [Standard Specifications](#) Section 9-28.11.
- b. The area of the new sign panel shall not exceed the area of the originally designed sign panel.
- c. The WSDOT inspection report for the bridge mounted sign bracket shall be reviewed to ensure the assembly is in good condition. If there is no inspection report, then an inspection shall be performed to establish the current condition of the assembly.

7. Material Specifications

- a. Material specifications shall be as shown in [Bridge Standard Drawings 10.1-A6-1](#).
- b. All non-stainless steel parts shall be galvanized in accordance with AASHTO M111 after fabrication. Bolts and hardware shall be galvanized in accordance with AASHTO M232.

8. Detailing

For standard bridge mounted sign bracket details see Bridge Standard Drawings 10.1-A6-1 to 10.1-A6-5. All information shown in the Layout (Bridge Standard Drawing 10.1-A6-1) shall be included on the contract plans. When attaching the lower bracket arm to concrete I-girders, concrete, box/tub girders, or steel I-girders, use Bridge Standard Drawing 10.1-A6-4A, 10.1-A6-4B, or 10.1-A6-4C, respectively.

C. Monotube Sign Structures Mounted on Bridges

1. Design Loads

Design loads for the supports of the Sign Bridges shall be calculated based on assuming a 12 foot deep sign over the entire roadway width, under the sign bridge, regardless of the sign area initially placed on the sign bridge. For Cantilever design loads, guidelines specified in [Section 10.1.1](#) shall be followed. The design loads shall follow the same criteria as described in [Section 10.1.1](#). Loads from the sign bridge shall be included in the design of the supporting bridge.

In cases where a sign structure is mounted on a bridge, the sign structure, from the anchor bolt group and above, shall be designed to *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, First Edition, dated 2015, including interims. The concrete around the anchor bolt group and the connecting elements to the bridge structure shall be designed to the specifications in this manual and AASHTO LRFD. Loads from the sign structure design code shall be taken as unfactored loads for use in *AASHTO LRFD Bridge Design Specifications*.

2. Vertical Clearance

Vertical clearance for Monotube Sign Structures shall be 20'-0" minimum from the bottom of the lowest sign to the highest point in the traveled lanes. See Bridge Standard Drawings [10.1-A1-1](#), [10.1A2-1](#), and [10.1-A3-1](#) for sample locations of Minimum Vertical Clearances.

3. Geometrics

Sign structures shall be placed at approximate right angles to approaching motorists. Dimensions and details of sign structures are shown in the WSDOT Standard Plans [G-60.10](#), [G-60.20](#), [G-60.30](#), [G-70.10](#), [G-70.20](#), [G-70.30](#) and Bridge Standard Drawings [10.1-A1-1](#), [10.1-A1-2](#), and [10.1-A1-3](#) and [10.1-A1-1](#), [10.1-A1-2](#), and [10.1-A1-3](#). When maintenance walkways are included, refer to Standard Plans [G-95.10](#), [G-95.20](#), and [G-95.30](#).

D. Monotube Sign Structures

1. Sign Bridge Conventional Design

[Table 10.1.4-1](#) provides the conventional structural design information to be used for a Sign Bridge Layout, [Bridge Standard Drawings 10.1-A1-1](#); along with the Structural Detail sheets, which are [Bridge Standard Drawings 10.1-A1-1](#) and [10.1-A1-3](#); and General Notes, [Bridge Standard Drawings 10.1-A5-1](#); and Miscellaneous Details, [Appendix 10.1-A5-2](#). Sign bridge span lengths shall not exceed 180-feet.

2. Cantilever Conventional Design

Table 10.1.4-2 provides the conventional structural design information to be used for a Cantilever Layout, Bridge Standard Drawings 10.1-A2-1; along with the Structural Detail sheets, which are Bridge Standard Drawings 10.1-A2-2 and 10.1-A2-3; and General Notes, Bridge Standard Drawings 10.1-A5-1; and Miscellaneous Details, Bridge Standard Drawings 10.1-A5-2. Cantilever arm lengths shall not exceed 35-feet. Cantilever sign structures shall not be used to support VMS signs.

3. Balanced Cantilever Conventional Design

Bridge Standard Drawings 10.1-A3-1; along with the Structural Detail sheets, Bridge Standard Drawings 10.1-A3-2 and 10.1-A3-3, General Notes, Bridge Standard Drawings 10.1-A5-1; and Miscellaneous Details, Bridge Standard Drawings 10.1-A5-2 provides the conventional structural design information to be used for a Balanced Cantilever Layout, Balanced Cantilevers are typically for VMS sign applications and shall have the sign positioned so that no less than $\frac{1}{3}$ of the sign dead load resides on either side of the post.

4. Monotube Sheet Guidelines

The following guidelines apply when using the Monotube Sign Structure Bridge Standard Drawings 10.1-A1-1, 10.1-A1-2, and 10.1-A1-3; 10.1-A2-1, 10.1-A2-2, and 10.1-A2-3; 10.1-A3-1, 10.1-A3-2, and 10.1-A3-3; 10.1A4-1, 10.1A4-2, and 10.1A4-3; and 10.1-A5-1, 10.1-A5-2, and 10.1-A5-3.

Each sign structure shall be detailed to specify:

- i. Sign structure base Elevation, Station, and Number.
- ii. Type of Foundation 1, 2, or 3 shall be used for the Monotube Sign Structures, unless a non-conventional design is required. The average Lateral Bearing Pressure for each foundation shall be noted on the Foundation sheet(s).
- iii. If applicable, label the Elevation View "Looking Back on Stationing".

5. VMS Installation

- a. VMS units shall not be installed on unbalanced cantilever structures.
- b. VMS installation on Sign Bridge structures designed in accordance with AASHTO 2015 shall be installed in accordance with the following:
 - i. On spans 120 feet and greater up to two VMS units may be installed with a maximum weight of 4,000 lbs. each. Maintenance walkways may be installed between VMS units, but may not exceed 160 lbs/ft, or exceed 50 percent of the structure span length.

- ii. On spans less than 120 feet up to three VMS units may be installed with a maximum weight of 4,000 lbs. each. Maintenance walkways may be installed between VMS units, but may not exceed 160 lbs/ft.
- c. The number of VMS installed on Sign Bridge structures designed prior to AASHTO 2015 shall be reduced by one as defined in D.2-a and b.

E. Foundations

1. Monotube Sign Structure Foundation Types

The foundation type to be used shall be based on the geotechnical investigation performed and geotechnical report completed by the Geotechnical Engineer of record. Standard foundation designs for standard plan truss-type sign structures are provided in Standard Plans G-60.20, G-60.30, G-70.20, and G-70.30. Monotube sign structure foundations are Bridge Design Office conventional designs and shall be as described in the following paragraphs:

- a. Foundation Type 1, is the preferred foundation type. A foundation Type 1 consists of a drilled shaft with its shaft cap. The design of the shaft depths shown in the Bridge Standard Drawings are based on a lateral bearing pressure of 2,500 psf. The designer shall check these shaft depths using LRFD methodology. For Type 1 foundation details and shaft depths see Bridge Standard Drawings 10.1-A4-1 and 10.1-A4-2. The geotechnical report for Foundation Type 1 should include the soil friction angle, soil unit weight, allowable bearing pressure and temporary casing if required. Temporary casing shall be properly detailed in all Foundation Type 1 sheets if the Geotechnical Engineer requires them.
- b. Foundation Type 2 is designed for a lateral bearing pressure of 2,500 psf. See [Bridge Standard Drawing 10.1-A4-3](#) for Bridge Design Office conventional Foundation Type 2 design information. The designer shall check these shaft depths using LRFD methodology.
- c. Foundation Type 3 replaces the foundation Type 2 for poor soil conditions where the lateral bearing pressure is between 2,500 psf and 1,500 psf. See [Bridge Standard Drawing 10.1-A4-3](#) for Bridge Design Office conventional Foundation Type 3 design information. The designer shall check these shaft depths using LRFD methodology.
- d. Barrier Shape Foundations are foundations that include a barrier shape cap on the top portion of Foundation Types 1, 2, and 3. Foundation details shall be modified to include Barrier Shape Cap details. See [Bridge Standard Drawing 10.1-A5-1](#) details a single slope barrier.

2. Luminaire, Signal Standard, and Camera Pole Foundation Types

Luminaire foundation options are shown on WSDOT Standard Plan J-28.30. Signal Standard and Camera Pole foundation options are provided on WSDOT Standard Plans J-26.10 and J-29.10 respectively.

3. Foundation Design

Shaft type foundations constructed in soil for sign bridges, cantilever sign structures, luminaires, signal standards and strain poles shall be designed in accordance with the current edition of the *AASHTO LRFD Standard Specifications For Highway Signs, Luminaires, and Traffic Signals*; Section 13.16; Drilled Shafts.

No provisions for foundation torsional capacity are provided in Section 10.13 of the *AASHTO Standard Specifications for Highway Signs, Luminaires, and Traffic Signals*. The following approach can be used to calculate torsional capacity of sign structure, luminaire, and signal standard foundations:

Torsional Capacity, ϕT_n ,

$$\phi T_n = F \cdot \tan \phi D$$

Where:

- F = Total force normal to shaft surface (kip)
- D = Diameter of shaft (feet)
- ϕ = Soil to foundation contact friction angle (degree), use smallest for variable soils

a. Monotube Sign Bridge and Cantilever Sign Structures Foundation Type 1 Design

The standard embedment depth “Z”, shown in the table on Monotube Sign Structure [Bridge Standard Drawing 10.1-A4-1](#), shall be used as a minimum embedment depth and shall be increased if the shaft is placed on a sloped surface, or if the allowable lateral bearing pressures are reduced from the standard 2500 psf. The standard depth assumed that the top 4 feet of the **cast-in-place (C.I.P.)** cap is not included in the lateral resistance (i.e., shaft depth “D” in the code mentioned above), but is included in the overturning length of the sign structure. The sign structure shaft foundation GSPs under Section 8-21 in the RFP Appendix shall apply for all Foundation Type 1 shafts.

b. Monotube Sign Structures Foundation Type 2 and 3

These foundation designs are Bridge Design Office convention and shall not be adjusted.

c. Monotube Sign Structures Non-Conventional Design Foundations

The Geotechnical Engineer of record shall identify any locations where the foundation types (1, 2, or 3) will not work. At these locations, the design forces are calculated, using the AASHTO LRFD *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, and applied at the bottom of the structure base plate. These forces are then considered service loads and the non-conventional design foundation is designed with the appropriate Service, Strength, and Extreme Load Combination Limit States and current design practices of the AASHTO LRFD and this manual. The anchor rod array shall be used from [Tables 10.1.4-1](#) and [10.1.4-2](#) and shall be long enough to develop the rods into the confined concrete core of the foundation. The rod length and the reinforcement for concrete confinement, shown in the top four feet of the Foundation Type 1, shall be used as a minimum.

d. Signal Foundation Design

The traffic signal standard GSPs in the RFP Section 8-20 shall apply for foundations in substandard soils.

F. Truss Sign Bridges: Foundation Sheet Design Guidelines

If a Truss sign structure is used, refer to WSDOT [Standard Plans](#) for foundation details. There are four items that should be addressed when using the WSDOT [Standard Plans](#), which are outlined below.

- a. Determine conduit needs. If none exist, delete all references to conduit. If conduit is required, verify as to size and quantity.
- b. Show sign bridge base elevation, number, dimension and station.
- c. The concrete barrier transition section shall be in accordance with the [Standard Plans](#).
- d. The quantities shall be based on the [Standard Plans](#) details as needed.

15.10.2 Bridge Traffic Barriers**A. General Guidelines and Policy**

The design criteria for traffic barriers on structures shall be in accordance with Chapter 13 of the AASHTO LRFD with the following supplemental guidelines:

The minimum traffic barrier height shall be 42 inches to meet the “Fall Protection” requirements in Section 10.2.1.

WSDOT standard 42-inch high Single Slope concrete barrier shall be used for barriers on new bridges, bridge approach slabs, retaining walls, Structural Earth Wall traffic barriers, and Geosynthetic wall traffic barriers. WSDOT standard 34-inch or 42-inch Single Slope concrete barrier shall be used for barriers on existing bridges and bridge rehabilitation projects.

Use of 32-inch or 42-inch F-Shape concrete barrier shall be limited for continuity off structure or within a corridor. Use of Pedestrian concrete barrier shall be limited for locations with sidewalk. Use of 42-inch combination barrier (32-inch or 34-inch concrete barrier increased in height by metal railing) may be used only if allowed by the RFP Criteria. See Chapter 10 Appendix for available WSDOT standard bridge barrier designs.

Barriers shall be designed for minimum Test Level 4 (TL-4) design criteria regardless of the barrier height. The Test Level shall be specified in the Plans.

A Test Level 5 (TL-5) barrier shall be used on new structures for "T" intersections, for barriers on structures with a radius of curvature less than 500 feet (TL-4 is acceptable for barrier on the inside of the curve), locations with Average Daily Truck Traffic (ADTT) greater than 10 percent, and locations with approach speeds at 50 mph or greater (e.g. freeway off-ramps).

See AASHTO LRFD Chapter 13 for additional Test Level selection criteria.

B. Design Criteria

1. Structural Capacity

AASHTO LRFD Appendix A13 shall be used to design barriers and their supporting elements (i.e. deck).

Concrete barriers shall be designed using yield line analysis as described in AASHTO LRFD Section A13.3.1.

Deck overhangs supporting concrete barriers shall be designed in accordance with AASHTO LRFD Section A13.4 as modified by Section 10.2.4.A.

2. Geometry

The traffic face geometry is part of the crash test and shall not be modified.

Concrete clear cover shall meet minimum concrete cover requirements and shall be increased to accommodate rustication grooves or patterns for architectural reasons. Concrete cover shall be increased to 2½" for traffic face of barrier for slip form method of construction.

The 3" toe dimension of the F-Shape barrier shall be increased to accommodate HMA overlays to a maximum of 6".

For designing and detailing bridge decks with a superelevation of 8 percent or less, the exterior barrier and/or the median barrier shall be oriented perpendicular to the bridge deck. Bridge decks with a superelevation of more than 8 percent, the barrier on the low side of the bridge and/or median barrier shall be oriented perpendicular to an 8 percent superelevated bridge deck, and the barrier on the high side of the bridge shall be oriented perpendicular to the bridge deck.

3. Miscellaneous Design Information

Steel reinforcement bars S1 and S2 or S3 and S4 and W1 and W2 (or equivalent bars) from Chapter 10 Appendix barrier standard drawings shall be included in the Bar List.

Steel reinforcement bars S1, S2, S3, S4, AS1, AS2, and W1 (or equivalent bars) from Chapter 10 Appendix barrier standard drawings shall be epoxy coated.

Any modifications to Chapter 10 Appendix barrier standard drawings or to WSDOT *Standard Plans* shall not alter or compromise the structural integrity and/or crash test performance of barrier. Modifications shall be submitted for review and acceptance by WSDOT for concurrence with design policies.

15.10.3 At Grade Concrete Barriers

Differential grade concrete barriers with a grade difference greater than 4'-0" shall be designed as reinforced concrete retaining walls with a traffic barrier at the top and a barrier shape at the cut face.

Differential grade concrete barriers with a grade difference 4'-0" or less shall be designed in accordance to AASHTO LRFD barrier loading with the following guidelines.

Full depth expansion joints with shear dowels at the top shall be provided at 120' maximum spacing.

Barrier shall be continuous or have shear connections between barrier sections if precast.

A. Differential Grade Concrete Barrier Design Criteria

1. Structural Capacity

The structural capacity of the differential grade concrete barrier shall be designed for the required Test Level (TL) vehicle impact design forces in accordance with AASHTO LRFD Chapters 5 and 13. The minimum Test Level shall be TL-3.

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

The torsion capacity of the differential grade concrete barrier shall be equal to or greater than the traffic barrier moment generated by the TL impact forces applied to the top of the barrier.

2. Global Stability

Global stability shall be in accordance with [Section 10.3.1.A](#).

3. Geometry

The top of the differential grade concrete barrier shall have a minimum width of 6" with a minimum 6" clear distance to each side of luminaire or sign pole if mounted on top of the differential grade concrete traffic barrier. The transition flare rate shall follow the [Design Manual](#) M 22-01.

Barrier bottom shall be embedded a minimum 6" below roadway. Roadway subgrade and ballast shall be extended below whole width of differential grade concrete barrier.

B. Traffic Barrier Moment Slab Design Criteria

1. Structural Capacity

The structural capacity of the traffic barrier moment slab shall be designed for the required TL impact forces in accordance with AASHTO LRFD Chapters 5 and 13. 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 AASHTO LRFD 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.

2. Global Stability

See [Section 10.3.2.B.2](#).

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.

4. Soil Reinforcement

Design of the soil reinforcement shall be in accordance with the [Geotechnical Design Manual](#) Chapter 15.

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.*

C. Precast Concrete Barrier

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

15.10.4 Bridge Traffic Barrier Rehabilitation

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.

B. 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.

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 AASHTO LRFD then the following minimum equivalent Extreme Event (CT) traffic barrier loading can be used:

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

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

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

A. General Guidelines and Policy

Pedestrian and bicycle/pedestrian railings shall be designed in accordance with AASHTO LRFD 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 296-155.

See [Section 10.5.2](#) for available bridge railing designs.

15.10.6 Bridge Approach Slabs

Bridge approach slabs are required for all new and widened bridges, and for all buried structures when soil cover is not provided.

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

A. Bridge Approach Slab Design Criteria

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

1. The bridge approach slab is designed as a slab in accordance with AASHTO LRFD. (Strength Limit State, IM = 1.33, no skew).
2. The support at the roadway end is assumed to be a uniform soil reaction with a bearing length that is approximately $\frac{1}{3}$ the length of the approach slab, or $25' / 3 = 8'$.
3. The Effective Span Length (Seff), regardless of approach length, is assumed to be: $25' \text{ approach} - 8' = 17'$.

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

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.

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](#).

D. Approach Anchors and Expansion Joints

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

L Type Abutments

Use a pinned connection in accordance with [Section 10.6.5](#).

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.

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.

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](#).

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

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.

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.

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.

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.

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.

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.

4. Utility Blockouts

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

5. Gas Lines or Volatile Fluids

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

6. Water Lines

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

7. Sewer Lines

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

8. Electrical (Power and Communications)

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

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.

D. Traffic Barrier Conduit

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

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

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

Pull boxes shall be provided at a maximum spacing of 180 feet. For fiber optics only, spacing shall not exceed 360 feet. The pull box size shall conform to the specifications of the National Electric Code or be a minimum of 8 inches by 8 inches by 18 inches to facilitate pulling of wires. Galvanized steel pull boxes (or junction boxes) shall meet the specifications of the "NEMA Type 4X" standard for stationary-form barrier, shall meet the specifications of the "NEMA 3R" and be adjustable in depth for slip form barrier, and the NEMA junction box type shall be stated on the plans. Stainless steel pull boxes may be used as an option to the galvanized steel.

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

E. Conduit Types

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

F. Utility Supports

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

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

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

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

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

a. 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.

b. Surface Mounting

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

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

15.10.9 Review Procedure for Utility Installations on Existing Structures

Utility installations on existing bridges shall be reviewed in accordance with [Section 10.9](#).

15.10.10 Anchors for Permanent Attachments

The design procedure for cast-in-place and post-installed anchors shall be in accordance with AASHTO LRFD 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

All drainage system expansion joints shall be watertight.

1. Geometrics

Bridges shall have a minimum transverse slope of .02'/feet and a minimum longitudinal slope of 0.5 percent.

2. Hydrology

Hydrological calculations are made using the rational equation. A 10 year storm event with a 5 minute duration is the intensity used for all inlets except for sag vertical curves where a 50 year storm intensity is required.

3. On Bridge Systems

Drains shall only be placed on bridge structures when required by bridge deck drainage hydraulics analysis and where alignment and superelevation geometry cannot be adjusted to compensate. The minimum pipe diameter shall be 6 inches with no bends greater than 45° within the system.

15.11 Detailing Practices

Structural detailing shall meet the requirements of this section.

15.11.1 Standard Practices

A. Drawing Orientation and Layout Control

- Contract plans shall be printed, sealed, signed and submitted on 11" × 17" paper. Alternatively, the Contract plans may be submitted in an electronic format accepted by WSDOT encrypted with a valid electronic signature.
- Drawings shall be organized so the intent of the drawing is easily understood.
 1. North arrow shall be placed on layouts and footing/foundation layouts.
 2. Related details shall be grouped together in an orderly arrangement, lined up horizontally and vertically and drawn to the same scale.
 3. The Plan view layout of structures shall be oriented from left to right in the direction of increasing state route mileposts. For layouts of existing bridges undergoing widening, expansion joint or thrie beam retrofit, or other structural modification, this orientation requirement may result in the bridge layout being opposite from what is shown in the original plans. In such cases, the bridge layout orientation and pier identification shall be laid out to be consistent with the WSDOT Bridge Preservation Office inspection records.
 4. Except for the Layout, wall elevations are to show the exposed face regardless of direction of stationing. The Layout sheet stationing shall read increasing left to right. The elevation sheets shall represent the view in the field as the wall is being built.

B. Lettering

1. General

- i. Lettering shall be upper case only, slanted at approximately 68 degrees. General text is to be approximately $\frac{1}{8}$ " high.
- ii. Text shall be oriented so as to be read from the bottom or right edge of the sheet.
- iii. Detail titles shall be a similar font as general text, about twice as high and of a heavier weight. They shall be underlined with a single line having the same weight as the lettering.
- iv. The True Type fonts "BridgeTech Italic" (BRIDT__.TTF, BRIDRG__.TTF) shall be used, exclusive of title blocks, and may be downloaded for use by the Design-Builder from the WSDOT Bridge and Structures web site.

2. Dimensioning

- i. A dimension shall be shown once on a drawing. Duplication and unnecessary dimensions shall be avoided.
- ii. All dimension figures shall be placed above the dimension line, so that they may be read from the bottom or the right edge of the sheet.
- iii. When details or structural elements are complex, utilize two drawings, one for dimensions and the other for reinforcing bar details.
- iv. Dimensions 12 inches or more shall be given in feet and inches unless the item dimensioned is conventionally designated in inches (for example, 16" \varnothing pipe).
- v. Dimensions that are less than one inch over an even foot, the fraction shall be preceded by a zero (for example, 3'-0 $\frac{3}{4}$ ").
- vi. Dimensions shall be placed outside the view, preferably to the right or below. However, in the interest of clarity and simplicity it may be necessary to place them otherwise.

C. Line Work

1. All line work shall be of sufficient size, weight, and clarity so that it can be easily read on a 11" \times 17" sheet.
2. The line style used for a particular structural outline, centerline, etc., shall be kept consistent wherever that line is shown within a set of plans.
3. Line work shall have appropriate gradations of width to give line contrast. Care shall be taken that the thin lines are dense enough to show clearly when reproduced.
4. When drawing structural sections showing reinforcing steel, the outline of the sections shall be a heavier line weight than the reinforcement steel.
5. The order of line precedence (which of a pair of crossing lines is broken) shall be as follows:
 - i. Dimension lines are never broken.
 - ii. Leader line from a callout.
 - iii. Extension line.

D. Scale

1. CAD Sheet Models shall be configured using standard architectural or engineering scales for referencing. All details shall be accurately represented at scale. Scales shall not be shown in the plans.
2. The minimum scale for a section detail with rebar shall be $\frac{3}{8}'' = 1'$. The minimum scale to be used on steel details shall be $\frac{3}{4}'' = 1'$.

E. Graphic Symbols

Graphic symbols shall be in accordance with the following:

1. Structural steel shapes: See the [AISC Steel Construction Manual](#).
2. Welding symbols: See the [Lincoln Welding Chart](#).
3. Symbols for hatching different materials are shown on [Appendix 11.1-A2](#).

F. Structural Sections, Views and Details

1. Whenever possible, sections and views shall be taken looking to the right, ahead on stationing, or down.
2. The orientation of a detail drawing shall be identical to that of the plan, elevation, etc., from which it is taken. Where there is a skew in the bridge any sections shall be taken from plan views.
3. The default view orientation is to be looking ahead on stationing. Other orientations shall be noted.
4. A circle divided into upper and lower halves shall identify structural sections, views, and details. Examples are shown in [Appendix 11.1-A3](#).
5. Breaks in lines are allowable provided that their intent is clear.

G. Miscellaneous

1. Callout arrows shall come off either the beginning or end of the sentence. This means the top line of text for arrows coming off the left of the callout or the bottom line of text for arrows pointing right.

H. RFC Revisions

1. Changes made to Released For Construction (RFC) plan sheets shall be clouded with the exception of table entries which shall be shaded in accordance with the [Plans Preparation Manual Appendix 5](#). Subsequent changes shall be clouded or shaded and the clouding and shading from previous changes shall be removed.
2. Changes shall be marked with a number in a circle in a triangle.
3. Changes shall be noted in the revision block at the bottom of the sheet using Lucida Console font 12pt.

I. Title Block

1. The project title shall be displayed in the plan sheet title block. The title consists of Line 1 specifying the highway route number(s), Line 2 and possibly Line 3 specifying the title verbiage. Bridge structures shall have a fourth line, in a smaller font, to specify the bridge name and number in accordance with the [Bridge List M 23-09](#) and [Sections 2.3.1.A](#) and [2.3.2.A](#).
2. The highway route number(s) in Line 1 shall be consistent with WSDOT naming practice. Interstate routes (5, 82, 90, 182, 205, 405, and 705) shall be specified as I-(number). US routes (2, 12, 97, 97A, 101, 195, 197, 395, and 730) shall be specified as US (number). All other routes shall be specified as SR (number). Projects including two highway routes shall include both route numbers in Line 1, as in "US 2 And I-5". Projects including three or more highway routes shall be specified with the lowest numbered route, followed by "Et Al", as in "SR 14 Et Al".

J. Reinforcement Detailing

1. Contract documents shall convey all necessary information for fabrication of reinforcing steel. In accordance with [Standard Specifications](#) Section 6-02.3(24), reinforcing steel details shown in the bar list shall be verifiable in the plans and other contract documents.
2. Size, spacing, orientation and location of reinforcement shall be shown on the plan sheets.
3. Reinforcement shall be identified by mark numbers inside a rectangle. Reinforcing bar marks shall be called out at least twice. The reinforcement including the spacing is called out in one view (such as a plan or elevation). The reinforcement without the spacing is called out again in at least one other view taken from a different angle (such as a section).
4. Epoxy coating for reinforcement shall be shown in the plans by noting an E inside a triangle.
5. The spacing for reinforcement shall be on a dimension line with extension lines. Do not point to a single bar and call out the spacing. Reinforcement spacing callouts shall include a distance. If the distance is an unusual number, give a maximum spacing. Do not use "equal spaces" as in "23 equal spaces = 18'-9\". Also, never use the word "about" as in 23 spaces @ about 10" = 18'-9\". Instead these should read "23 spaces @ 10" max. = 18'-9\".
6. Reinforcement geometry shall be clear in plan details. Congested areas, oddly bent bars, etc. can be clarified with additional views/details/sections or adjacent bending diagrams. In bending diagrams, reinforcement dimensions shall be given out-to-out. It may be necessary to show edges of reinforcement with two parallel edge lines to clearly show working points and dimensions.
7. Reinforcement lengths, angles, etc. need not be called out when they can be determined from structural member sizes, cover requirements, etc. Anchorage,

embedment and extension lengths of reinforcement shall be dimensioned in the plans.

8. Standard hooks in accordance with AASHTO LRFD Section 5.10.2.1 need not be dimensioned or called out, but shall be drawn with the proper angle (90°, 135° or 180°). Seismic hooks per AASHTO LRFD Section 5.10.2.2 (used for transverse reinforcement in regions of expected plastic hinges) shall be called out on the plans whenever they are used.
9. The location, length and stagger of lap splices shall be shown on the plan sheets. Tables of applicable lap splice lengths are acceptable with associated stagger requirements. Type, location and stagger of mechanical and welded splices of reinforcement shall be shown.
10. Where concrete cover requirements differ from those given in the standard notes or *Standard Specifications* Section 6-02.3(24)C, they shall be shown in the plans. It shall be clear whether the cover requirement refers to ties and stirrups or the main longitudinal bars.

15.11.2 Bridge Office Standard Drawings and Office Examples

A. General

The Bridge and Structures Office provides standard drawings and example sheets of various common bridge elements.

B. Use of Standards

The standard drawings shall be considered as nothing more than examples of items like girders or traffic barriers which are often used and are very similar from job to job.

They shall be modified to fit the particular aspects of the structure. They are not intended to be included in a plan set without close scrutiny for applicability to the job.

15.11.3 Plan Sheets

Plan sheets shall be assembled in the order of construction and shall include the items listed below.

- | | |
|---------------------------------------|---|
| • Layout | • Framing Plan |
| • General Notes/Construction Sequence | • Typical Section |
| • Footing/Foundation Layout | • Girders/Diaphragms |
| • Piles/Shafes | • Bridge Deck Reinforcement (Plan and transverse section) |
| • Abutments | • Expansion Joints (if needed) |
| • Intermediate Piers/Bents | • Traffic Barrier |
| • Bearing Details | • Bridge Approach Slab |

A. Layout

- The Layout sheet shall contain, but is not limited to:
 - Plan View with ascending stations from left to right
 - Elevation View shown as an outside view of the bridge and shall be visually aligned with the plan view.
- Alignment lines, vertical curves and roadway superelevation diagrams.
 - Test hole locations (designated by $\frac{3}{16}$ inch circles, quartered) to plan view.
 - Elevation view of footings, seals, piles, etc. Show elevation at Bottom of footing and, if applicable, the type and size of piling.
 - General notes above legend on right hand side, usually in place of the typical section.
 - Title “LAYOUT” in the title block and sheet number in the space provided.
 - Other features, such as lighting, conduit, signs, excavation, riprap, etc. as determined by the designer.

B. General Notes/Construction Sequence

- The general notes sheet shall contain, but is not limited to those shown in Section 11.1.3

C. Footing Layout

- An abutment with a spread footing has a Footing Layout. An abutment with piles and pile cap has a Foundation Layout.
- The Footing Layout is a plan of the bridge whose details are limited to those needed to locate the footings. The intent of the footing layout is to minimize the possibility of error at this initial stage of construction.
- The Foundation Layout is a plan of the bridge whose details are limited to those needed to locate the shafts or piles. The intent of the Foundation layout is to minimize the possibility of error at this initial stage of construction.
- Other related information and/or details such as pedestal sizes, and column sizes are considered part of the pier drawing and should not be included in the footing layout.
- The Footing Layout should be shown on the layout sheet if space allows. It need not be in the same scale. When the general notes and footing layout cannot be included on the first (layout) sheet, the footing layout should be included on the second sheet.
- Longitudinally, footings should be located using the survey line to reference such items as the footing, centerline pier, centerline column, or centerline bearing, etc.
- When seals are required, their locations and sizes should be clearly indicated on the footing layout.
- The Wall Foundation Plan for retaining walls is similar to the Footing Plan for bridges except that it also shows dimensions to the front face of wall.

- Appendix 11.1-A4 is an example of a footing layout showing:
 - The basic information needed.
 - The method of detailing from the survey line.

D. Piles/Shfts**E. Abutment**

- Bridge elements that have not yet been built will not be shown. For example, the superstructure is not to be shown, dashed or not, on any substructure details.
- Elevation information for seals and piles or shafts may be shown on the abutment or pier sheets.
- Views are to be oriented so that they represent what the contractor or inspector would most likely see on the ground. Pier 1 elevation is often shown looking back on stationing. A note should be added under the Elevation Pier 1 title saying "Shown looking back on stationing".

F. Intermediate Piers/Bents

- Each pier shall be detailed separately as a general rule. If the intermediate piers are identical except for height, then they can be shown together.

F. Bearing Details**G. Framing Plan**

- Girder Lines must be identified in the plan view (Gir. A, Gir. B, etc.).

H. Typical Section

- Girder spacing, which is tied to the bridge construction baseline
- Bridge deck thickness, as well as web and bottom slab thicknesses for box girders
- "A" dimension
- Limits of pigmented sealer
- Profile grade and pivot point and cross slopes
- Utility locations
- Curb to curb roadway width
- Soffit and drip groove geometry

I. Girders/Diaphragms

- Prestressed girder sheets can be copied from the Bridge and Structures Office library but they shall be modified to match the project requirements.

J. Bridge Deck Reinforcement

- Plan and transverse section views

K. Expansion Joints**L. Traffic Barrier**

- Traffic barrier sheets can be copied from the [Standard Plans](#) but they must be modified to match the project requirements.

M. Bridge Approach Slab

- Approach slab sheets can be copied from the [Standard Plans](#) and modified as necessary for the project.

N. Barlist

- The barlist sheets do not require stamping because they are not officially part of the contract plan set.

15.11.5 Structural Steel**A. General**

Flat pieces of steel are termed plates, bars, sheets or strips, depending on the dimensions.

B. Barsw

Up to 6 inches wide, 0.203 inch ($\frac{3}{16}$ -inch) and over in thickness, or 6 inches to 8 inches wide, 0.230 inch ($\frac{7}{32}$ -inch) and over in thickness.

C. Plates

Over 8 inches wide, 0.230 inch ($\frac{7}{32}$ -inch) and over in thickness, or over 48 inches wide, 0.180 in ($\frac{11}{64}$ -inch) and over in thickness.

D. Strips

Thinner pieces up to 12 inches wide are strips and over 12 inches are sheets. A complete table of classification may be found in the AISC *Manual of Steel Construction*, 8th Ed. Page 6-3.

E. Labeling

The following table shows the usual method of labeling some of the most frequently used structural steel shapes. Note that the inches symbol (") is omitted, but the foot symbol (') is used for length including lengths less than a foot.

PLATES	PL	1/2	x	3/4	x	5'-6	ANGLES	L	6	x	5	x	3/4	x	2'-1
	GROUP SYMBOL	THICKNESS IN INCHES		WIDTH IN INCHES		LENGTH IN FEET AND INCHES		GROUP SYMBOL	LONG LEG IN INCHES		SHORT LEG IN INCHES		THICKNESS IN INCHES		LENGTH IN FEET AND INCHES
FLAT BARS	BAR	2	x	3/4	x	0'-6	RECTANGULAR HSS	HSS	6	x	5	x	1/4	x	3'-2
	GROUP SYMBOL	WIDTH IN INCHES		THICKNESS IN INCHES		LENGTH IN FEET AND INCHES		GROUP SYMBOL	WIDTH IN INCHES		WIDTH IN INCHES		WALL THICKNESS IN INCHES		LENGTH IN FEET AND INCHES
SQUARE BARS	BAR	2	Ø	x	3'-4		CIRCULAR HSS	HSS	3.000	x	0.250	x	2'-5		
	GROUP SYMBOL	SIZE IN INCHES	CONVENTION FOR "SQUARE"		LENGTH IN FEET AND INCHES			GROUP SYMBOL	OUTSIDE DIAM. IN INCHES		WALL THICKNESS IN INCHES		LENGTH IN FEET AND INCHES		
ROUND BARS	BAR	2	Ø	x	0'-4		PIPES	1 1/2"Ø	STD	PIPE					
	GROUP SYMBOL	SIZE IN INCHES	CONVENTION FOR "ROUND"		LENGTH IN FEET AND INCHES			NOMINAL DIAM. IN INCHES	DESIGNATION	GROUP SYMBOL					

15.11.6 Aluminum Section Designations

The designations used in the tables are suggested for general use.

Section	Designation	Example
I-Beams	I DEPTH \times WT	14 \times 3.28
Wide-Flange Sections	WF DEPTH \times WT	WF4 \times 4.76
Wide-Flange Sections, Army-Navy Series	WF(A-N) DEPTH \times WT	WF(A-N)4 \times 1.79
American Standard Channels	C DEPTH \times WT	C4 \times 1.85
Special Channels	CS DEPTH \times WT	CS4 \times 3.32
Wing Channels	CS(WING) WIDTH \times WT	CS(WING)4 \times 0.90
Army-Navy Channels	C(A-N) DEPTH \times WT	C(A-N)4 \times 1.58
Angles	L LL \times LL \times TH	L3 \times 3 \times 0.25
Square End Angles	LS LL \times LL \times TH	LS2 \times 2 \times 0.187
Bulb Angles	BULB L LL1 \times LL2 \times TH1 \times TH2	BULB L4 \times 3.5 \times 0.375 \times 0.375
Bulb Angle, Army-Navy Series	BULB L(A-N) LL1 \times LL2 \times TH1 \times TH2	BULB L(A-N) 3 \times 2 \times 0.188 \times 0.188
Tees	T DEPTH \times WIDTH \times WT	T4 \times 4 \times 3.43
Army-Navy Tees	T(A-N) DEPTH \times WIDTH \times WT	T(A-N)4 \times 4 \times 2.27
Zees	Z DEPTH \times WIDTH \times WT	Z4 \times 3.06 \times 2.85
Plates	PL TH \times WIDTH	PL $\frac{1}{4}$ \times 8
Rods	RD DIA	RD 1
Square Bars	SQ SDIM	SQ 4
Rectangle Bars	RECT TH \times WIDTH	RECT $\frac{1}{4}$ \times 4
Round Tubes	ODIA OD \times TH WALL	4OD \times 0.125 WALL
Square Tubes	ODIM SQ \times TH WALL	3SQ \times 0.219 WALL
Rectangle Tubes	DEPTH \times WIDTH RECT \times TH WALL	4 \times 1.5 RECT \times 0.104 WALL
WT - WEIGHT in LB/FT based on density of 0.098 TH - THICKNESS, LL - LEG LENGTH, DIA - DIAMETER ODIA - OUTSIDE DIAMETER, ODIM - OUTSIDE DIMENSION SDIM - SIDE DIMENSION All lengths are in inches		

15.11.7 Abbreviations

Abbreviations shall be defined in the contract documents.

15.12 Bridge Load Rating

15.12.1 General

Load ratings shall be completed for all new, widened, or rehabilitated bridges where the rehabilitation alters the load carrying capacity of the structure. Load ratings shall be done immediately after the design is completed and rating calculations shall be filed separately in accordance with [Section 13.4](#) and files shall be forwarded to WSDOT's Load Rating Engineer. A final stamped and signed load rating shall be provided at least 90-days prior to opening any bridge requiring load rating to traffic. Final approval of the load rating of a bridge shall rest with WSDOT's Load Rating Engineer.

New bridges shall be rated based on the Load and Resistance Factor Rating (LRFR) method in accordance with the *AASHTO Manual For Bridge Evaluation* (MBE), [Chapter 13](#) of this manual and this chapter. NBI ratings shall be based on the HL-93 truck and shall be reported as a rating factor.

15.12.2 Load Rating Software

For prestressed concrete girder bridges and X-Beams, use BridgeLink to load rate structural elements. For all other cases where BridgeLink cannot be used, such as but not limited to, or steel structures or segmental boxes, CSiBridge shall be used. Obtain WSDOT's Load Rating Engineer's approval for the use of any other software prior to commencing any work.

For more complex structures such as steel curved girders and arches, different software may be used to analyze the loads after obtaining approval from WSDOT's Load Rating Engineer. Acceptable software currently includes CSiBridge.

Loads and capacities shall be tabulated in a manner that will make it simple for WSDOT to manipulate the data in the future. Method of tabulation shall be approved by WSDOT's Load Rating Engineer prior to commencing any work. Microsoft Excel shall be used for tabulation, and all cells in the spreadsheets shall be unlocked and any hidden code or functions shall be explained thoroughly in the report. Hand calculations shall be provided to verify all spreadsheets.

15.13 Appendices

[Appendix 15.2-A1](#) Conceptual Plan Checklist

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

- _____ 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

AASHTO LRFD *Bridge Design Specifications*, Latest Edition and Interims. American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.

AASHTO *Guide Specifications for LRFD Seismic Bridge Design (SEISMIC)*, Latest Edition and Interims. American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.

AASHTO [*Standard Specifications for Highway Bridges*](#), 17th edition

AASHTO *Manual for Bridge Evaluation*

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

WSDOT [*Bridge Inspection Manual*](#) M 36-64

WSDOT [*Geotechnical Design Manual*](#) M 46-03

WSDOT [*Design Manual*](#) M 22-01

WSDOT [*Construction Manual*](#) M 41-01

WSDOT [*Local Agency Guidelines*](#) M 36-63

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

WSDOT [*Standard Plans*](#) M 21.01

WSDOT [*Hydraulics Manual*](#) M 23-03

Washington Utilities and Transportation Commission [*Clearance Rules and Regulations Governing Common Carrier Railroads*](#)

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

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