Chapter 3 Contents

Chapter 3 Loads

3.1 Scope

The current AASHTO *Load and Resistance Factor Design (LRFD) Bridge Design Specifications* and applicable AASHTO *Guide Specifications* shall be the minimum design criteria used for all bridges except as modified herein.

3.2 Definitions

The definitions in this section supplement those given in AASHTO LRFD Section 3.

Permanent Loads – Loads and forces that are, or are assumed to be, either constant upon completion of construction or varying only over a long time interval.

Transient Loads – Loads and forces that can vary over a short time interval relative to the lifetime of the structure.

Temporary Loads – Loads that occur for a limited duration during pre-service conditions

Construction Loads – Loads that occur during any construction activity and include imposed loads due to falsework and formwork, material stockpiles when permitted, and construction equipment

Tsunami Loads – Loads that occur as result of tsunami

3.3 Load Designations

Load designations follow AASHTO LRFD Article 3.3.2, and as specified in this section.

3.4 Limit States

The basic limit state equation given by AASHTO LRFD 1.3.2.1 is as:

$$\Sigma \eta_i \gamma_i Q_i \leq \phi R_n = R_r \tag{3.4-1}$$

For loads for which a maximum value of γ_i is appropriate

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$$

For loads for which a minimum value of γ_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0$$

where:

η_i = load modifier
γ_i = load factor
Q_i = force effect
ϕ = resistance factor
R_n = nominal resistance
R_r = factored resistance

The modifier, η_i is the product of factors for ductility, redundancy, and importance. For simplicity use a value of 1.0 for η_i except for the design of columns when a minimum value of γ_i is appropriate. In such a case, use η_i = 0.95. Compression members in seismic designs are proportioned and detailed to ensure the development of significant and visible inelastic deformations at the extreme event limit states before failure.

Strength IV load combination shall not be used for foundation design.

The load factor for live load in the Service III load combination shall be as specified in Section 3.5.

The tsunami load is considered an Extreme Event load with load factor of 1.0 and load combination similar to Extreme Event I.

3.5 Load Factors and Load Combinations

The limit states load combinations, and load factors (γ_i) used for structural design are in accordance with the AASHTO LRFD Table 3.4.1-1. For foundation design, loads are factored after distribution through structural analysis or modeling.

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

> γ_{LL} = 0.8 when the requirements of Sections 5.6.1 and 5.6.2 are satisfied and stress analysis is based on gross section properties.
>
> γ_{LL} = 1.0 when the requirements of Sections 5.6.1 and 5.6.2 are satisfied and stress analysis is based on transformed section properties.

In special cases that deviate from the requirements of Sections 5.6.1 and 5.6.2 and have been approved by the WSDOT Bridge Design Engineer, γ_{LL}, shall be as specified in AASHTO LRFD Table 3.4.1-4.

The Service III live load factor for load rating shall be as specified in Section 13.1.1.

The live load factor for Extreme Event-I Limit State load combination, γ_{EQ} as specified in the AASHTO LRFD Table 3.4.1-1 for all WSDOT bridges and walls shall be taken equal to 0.50. The γ_{EQ} factor applies to the live load force effect obtained from the bridge live load analysis. Associated mass of live load need not be included in the dynamic analysis.

The AASHTO LRFD allows the live load factor in Extreme Event-I load combination, γ_{EQ}, to be determined on a project specific basis. The commentary indicates that the possibility of partial live load, i.e., γ_{EQ} < 1.0, with earthquakes should be considered. The application of Turkstra's rule for combining uncorrelated loads indicates that γ_{EQ} = 0.50 is reasonable for a wide range of values of average daily truck traffic (ADTT). The NCHRP Report 489 recommends live load factor for Extreme Event-I Limit State, γ_{EQ} equal to 0.25 for all bridges. This factor shall be increased to γ_{EQ} equal to 0.50 for bridges located in main state routes and congested roads.

Since the determination of live load factor, γ_{EQ} is based on ADTT or bridges located in congested roads could be confusing and questionable, it is decided that live load factor of γ_{EQ} equal to 0.50 to be used for all WSDOT bridges regardless the bridge location or traffic congestion. The live load factor equal to zero shall be used for tsunami load combination.

The base construction temperature may be taken as 64°F for the determination of Temperature Load.

The load factors γ_{TG} and γ_{SE} are to be determined on a project specific basis in accordance with AASHTO LRFD 3.4.1 and 3.12. Load Factors for Permanent Loads, γ_p are provided in AASHTO LRFD Table 3.4.1-2.

The load factor for down drag loads shall be as specified in the AASHTO LRFD Table 3.4.1-2. The Geotechnical Report will provide the down drag force (*DD*). The down drag force is a load applied to the pile/shaft with the load factor specified in the Geotechnical Report. Generally, live loads (*LL*) are less than the down drag force and should be omitted when considering down drag forces.

The Load Factors for Superimposed Deformations are provided in Table 3.5-3 below.

Table 3.5-3 Load Factors for Superimposed Deformations

	PS	CR, SH
Superstructure	1.0	1.0
Substructures that are fixed at the base and have a longitudinal connection with the superstructure (such as a hinged or integral connection) and analyzed using the gross moment of inertial (I_g)	0.5	0.5
All other substructure supporting Superstructure analyzed using either gross moment of inertia (I_g) or the effective cracked moment of inertia ($I_{effective}$)	1.0	1.0

3.5.1 Load Factors for Substructure

Table 3.5-4 below provides general guidelines for when to use the maximum or minimum shaft/pile/column permanent load factors for axial capacity, uplift, and lateral loading.

In general, substructure design should use unfactored loads to obtain force distribution in the structure. Moment, shear and axial force responses are then factored for final structural design. All forces and load factors are as defined previously.

Table 3.5-4 Minimum/Maximum Substructure Load Factors for Strength Limit State

Axial Capacity	Uplift	Lateral Loading
DC_{max}, DW_{max}	DC_{min}, DW_{min}	DC_{max}, DW_{max}
DC_{max}, DW_{max} for causing shear	DC_{max}, DW_{max} for causing shear	DC_{max}, DW_{max} causing shear
DC_{min}, DW_{min} for resisting shear	DC_{min}, DW_{min} *for resisting shear*	DC_{min}, DW_{min} resisting shear
DC_{max}, DW_{max} for causing moments	DC_{max}, DW_{max} for causing moments	DC_{max}, DW_{max} for causing moments
DC_{min}, DW_{min} for resisting moments	DC_{min}, DW_{min} for resisting moments	DC_{min}, DW_{min} for resisting moments
EV_{max}	EV_{min}	EV_{max}
DD = varies	DD = varies	DD = varies
EH_{max}	EH_{max} if causes uplift	EH_{max}

In the table above, “causing moment” and “causing shear” are taken to be the moment and shear causing axial, uplift, and lateral loading respectively. “Resisting” is taking to mean those force effects that are diminishing axial capacity, uplift, and lateral loading.

3.6 Loads and Load Factors for Construction

Unless otherwise specified, the load factor for construction loads and for any associated dynamic effects shall not be less than 1.5 in Strength I.

When investigating Strength Load Combinations I, III, and V during construction, load factors for the weight of the structure and appurtenances, *DC* and *DW*, shall not be taken to be less than 1.25 when investigating for maximized force effects. Strength IV investigations shall use the appropriate factors from AASHTO LRFD Tables 3.4.1-1 and 3.4.1-2.

Where evaluation of construction deflections is required by the contract documents, Load Combination Service I shall apply. Construction dead loads shall be considered as part of the permanent load and construction transient loads considered part of the live load. The associated permitted deflections shall be included in the contract documents.

For falsework and formwork design loads, see *Standard Specifications* Section 6-02.3(17)A. The base construction temperature shall be taken as 64°F for the determination of Temperature Load.

3.7 Load Factors for Post-tensioning

3.7.1 Post-tensioning Effects from Superstructure

When cast-in-place, post-tensioned superstructure is constructed monolithic with the piers, the substructure design should consider frame moments and shears caused by elastic shortening and creep of the superstructure upon application of the axial post-tensioning force at the bridge ends. Frame moments and shears thus obtained should be added algebraically to the values obtained from the primary and secondary moment diagrams applied to the superstructure.

When cast-in-place or precast, post-tensioned superstructure are supported on sliding bearings at some of the piers, the design of those piers should include the longitudinal force from friction on the bearings generated as the superstructure shortens during jacking. When post-tensioning is complete, the full permanent reaction from this effect should be included in the governing AASHTO load combinations for the pier under design.

3.7.2 Secondary Forces from Post-tensioning, PS

The application of post-tensioning forces on a continuous structure produces reactions at the structure's support and internal forces that are collectively called secondary forces.

Secondary prestressing forces (i.e. secondary moments) are the force effects in continuous members, as a result of continuous post-tensioning. In frame analysis software, the secondary moments are generally obtained by subtracting the primary moment (P*e) from the total PS moment obtained by applying an equivalent static load which represents the forces due to post-tensioning. A load factor, γ_{PS}, of 1.0 is appropriate for the superstructure. For fixed columns a 50 percent reduction in PS force effects could be used given the elasto-plastic characteristics of the soil surrounding the foundation elements.

3.8 Permanent Loads

The design unit weights of common permanent loads are provided in Table 3.8-1.

Table 3.8-1 Permanent Loads

Item	Load
Precast Pre-tensioned or Post-tensioned Spliced Girders including 10 lb/ft^3 allowance for reinforcement	165 lb/ft^3
All Other Normal-Weight Reinforced Concrete including 5 lb/ft^3 allowance for reinforcement	155 lb/ft^3
Unreinforced Concrete	145 lb/ft^3
Concrete Overlay	150 lb/ ft^3
Lightweight or specified density concrete without allowance for reinforcement	110-135 lb/ft^3
Stay-in-Place Form for Box Girder (applied to slab area less overhangs and webs)	5 lb/ft^2
Traffic Barrier (32″ – F Shape) (Normal weight concrete)	460 lb/ft
Traffic Barrier (42″ – F Shape) (Normal weight concrete)	710 lb/ft
Traffic Barrier (34″ – Single Slope) (Normal weight concrete)	490 lb/ft
Traffic Barrier (42″ – Single Slope) (Normal weight concrete)	670 lb/ft
Wearing Surface – Hot Mix Asphalt (HMA)/Asphalt Concrete Pavement (ACP)	140 lb/ft^3
Soil, Compact	125 lb/ft^3

For lightweight concrete barrier, multiply the normal weight concrete barrier weight from Table 3.8-1 by the ratio of the actual material weight to the unit weight of normal weight concrete (155 lb/ft^3).

3.8.1 *Deck Overlay Requirement*

Vehicular traffic will generate wear and rutting on a concrete bridge deck over the life of a bridge. One option to correct excessive wear is to add a Hot Mix Asphalt (HMA) overlay on top of the existing concrete deck. This type of overlay requires less construction time and is less expensive compared to removing a portion of the deck and adding a modified concrete overlay. The initial bridge design needs to incorporate the future overlay dead load. 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.

3.8.2 *Distribution of Permanent Loads*

The dead load of one traffic barrier is divided uniformly among the three nearest girders. When there are six or fewer girders, traffic barrier loads are equally distributed to all girders.

Sidewalk loads are distributed by the same method as traffic barrier loads. However, if a wide sidewalk is directly supported by more than three girders, the sidewalk load shall be distributed equally to all supporting girders.

3.9 Live Loads

3.9.1 Design Live Load

Live load design criteria are specified in the lower right corner of the bridge preliminary plan sheet. The Bridge Preliminary Plan Engineer determines the criteria using the following guideline:

- New bridges and bridge widening with addition of substructure – HL-93
- Bridge superstructure widening with no addition of substructure – Live load criteria of the original design
- Temporary widening of existing bridges: Live load criteria of the existing bridge
- Detour and other temporary bridges, except as required by Section 10.13.2 – 75 percent of HL-93

The application of design vehicular live loads shall be as specified in AASHTO LRFD 3.6.1.3. The design tandem, or "low boy", defined in AASHTO LRFD C3.6.1.3.1 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 AASTHO LRFD 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.

3.9.2 Loading for Live Load Deflection Evaluation

The loading for live load deflection criteria is defined in AASHTO LRFD 3.6.1.3.2. Live load deflections for the Service I limit state shall satisfy the requirements of AASHTO LRFD 2.5.2.6.2.

3.9.3 Distribution to Superstructure

3.9.3.A Multi Girder Superstructure

The live load distribution factor for exterior girder of multi girder bridges designated in AASHTO LRFD Table 4.6.2.2.1-1 as type a, b, c, e, k and also i, j if sufficiently connected to act as a unit, 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 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 special analysis based on the conventional approximation of loads on piles as described in AASHTO LRFD C4.6.2.2.2d shall not be used unless the effectiveness of diaphragms on the lateral distribution of truck load is investigated. In accordance with the AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 and later, the special analysis is only applicable to steel beam-slab bridge cross-sections with diaphragms or cross-frames.

3.9.3.B Concrete Box Girders

The load distribution factor for multi-cell cast in place concrete box girders shall be for interior girders from AASHTO LRFD Table 4.6.2.2.2b-1 for bending moment, and Table 4.6.2.2.3a-1 for shear. The live load distribution factor for interior girders shall then be multiplied by the number of webs to obtain the design live load for the entire superstructure. The live load distribution need not exceed the total number of design lanes. The correction factor for live load distribution for skewed support as specified in AASHTO LRFD Table 4.6.2.2.2e-1 for bending moment and AASTHO LRFD Table 4.6.2.2.3c-1 for shear shall apply.

$DF = N_b \times Df_i$ Live load distribution factor for multi-cell box girder (3.9.4-1)

Where:

Df_i = Live load distribution factor for interior web

N_b = Number of webs

3.9.3.C Multiple Presence Factors

A reduction factor will be applied in the substructure design for multiple lane loadings in accordance with AASHTO LRFD 3.6.1.1.2.

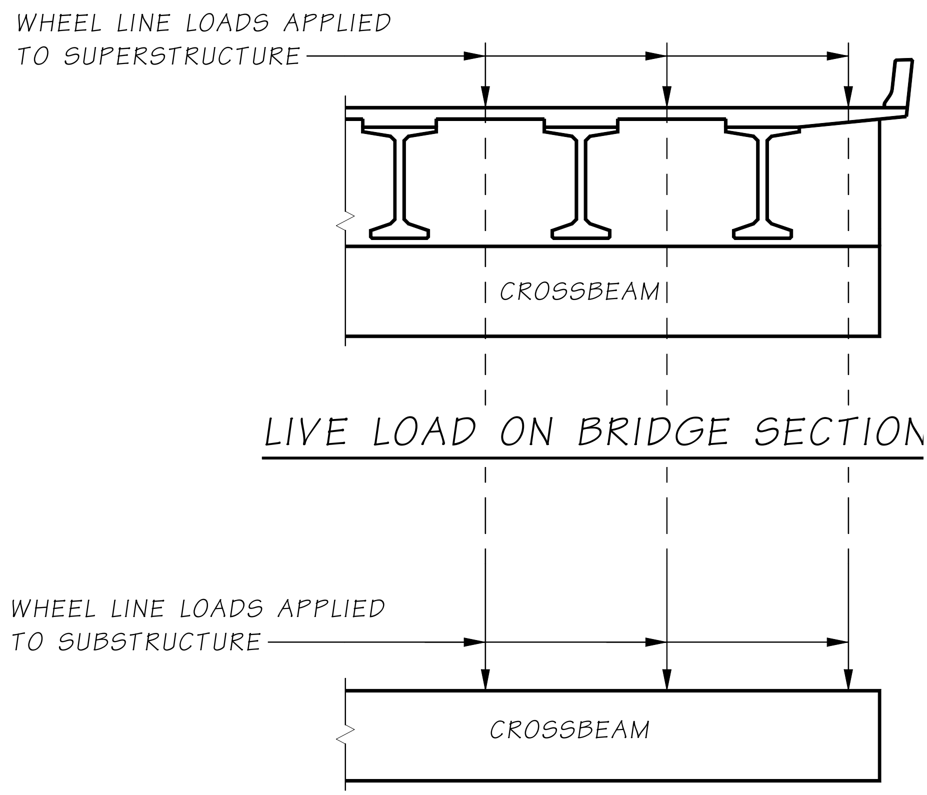
3.9.3.D Distribution to Substructure

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

3.9.3.E Distribution to Crossbeam

The design and load rating live loading are distributed to the substructure by placing wheel line reactions in lane configurations that generate the maximum force effects in the substructure. A wheel line reaction is one-half of the reaction of a single lane of live load. For integral and hinged continuity diaphragms, live loads are considered to act directly on the substructure without further distribution through the superstructure as illustrated in Figure 3.9-1. For girder configurations where there is a clear load path through the girders to the cross beam, such as at expansion piers with girders supported on individual bearings, live load reactions are applied through the bearings. Normally, substructure design will not consider live load torsion or lateral distribution. Sidesway effects shall be considered.

Figure 3.9-1 Live Load Distribution to Substructure



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. Live load placement is dependent on the member under design. Some examples of live load placement are as follows: the exterior vehicle wheel is placed 2 feet from the curb for maximum crossbeam cantilever moment or maximum eccentric foundation moment.

For crossbeam design between supports, the lanes are placed to obtain the maximum positive moment in the member; then re-located to obtain the maximum shear or negative moment in the member.

For column design, the design lanes are placed to obtain the maximum transverse moment at the top of the column; then re-located to obtain the maximum axial force of the column.

3.9.4 Bridge Load Rating

Bridge designers are responsible for Design, Legal, and Permit load rating of new bridges in accordance with the National Bridge Inspection Standards (NBIS) and the AASHTO *Manual Bridge Evaluation*. See Chapter 13 for detailed information on loading requirements for bridge load rating.

3.10 Pedestrian Loads

Pedestrian bridges shall be designed in accordance with the requirements of the AASHTO *LFRD Guide Specifications for the Design of Pedestrian Bridges*, dated December 2009.

Seismic design of pedestrian bridges shall be performed in accordance with the requirements of the AASHTO SEISMIC.

Pedestrian live load on vehicular bridge shall be as specified in LRFD 3.6.1.6.

Pedestrian live loads on sidewalks shall be distributed to the same girders as the sidewalk dead load.

3.11 Wind Loads

3.11.1 *Wind Load to Superstructure*

For the usual girder and slab bridges having individual span length of not more than 150 ft and a maximum height of 33 feet above low ground or water level, the following simplified wind pressure on structure (*WS*), could be used in lieu of the general method described in AASHTO LRFD 3.8.1.2:

Table 3.11.1-1 Wind Pressure (kip per square foot)

Limit State	Wind Exposure Category					
	B		C		D	
	Transverse	Longitudinal	Transverse	Longitudinal	Transverse	Longitudinal
Strength III	0.029	0.007	0.040	0.010	0.046	0.012
Strength V	0.021	0.005	0.021	0.005	0.021	0.005
Service I	0.016	0.004	0.016	0.004	0.016	0.004
Service IV	0.016	0.004	0.023	0.006	0.026	0.007

Both forces shall be applied simultaneously.

For the usual girder and slab bridges having individual span length of not more than 150 feet and a maximum height of 33 feet above low ground or water level, the following simplified wind pressure on vehicle (*WL*), could be used in lieu of the general method described in AASHTO LRFD 3.8.1.3:

- 0.10 kip per linear foot, transverse
- 0.04 kip per linear foot, longitudinal

Both forces shall be applied simultaneously.

3.11.2 *Wind Load to Substructure*

Wind forces shall be applied to the substructure units in accordance with the loadings specified in AASHTO LRFD. Transverse stiffness of the superstructure may be considered, as necessary, to properly distribute loads to the substructure provided that the superstructure can sustain such loads. Vertical wind pressure, per AASHTO LRFD 3.8.2, shall be included in the design where appropriate, for example, on single column piers. Wind loads shall be applied through shear keys or other positive means from the superstructure to the substructure.

Wind loads shall be distributed to the piers and abutments in accordance with the laws of statics. Transverse wind loads can be applied directly to the piers assuming the superstructure to act as a rigid beam. For large structures a more appropriate result might be obtained by considering the superstructure to act as a flexible beam on elastic supports.

3.11.3 *Wind on Noise Walls*

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

3.12 Loads on Buried Structures

Loads, live load distribution, and seismic design of buried structures shall be in accordance with the requirements of Section 8.3.

3.13 Earthquake Effects

Earthquake loads see Chapter 4.

3.14 Substructure, Scour and Earth Pressure

For substructure, scour, and earth pressure loads see Chapter 7.

3.15 Force Effects Due to Superimposed Deformations

PS, *CR*, *SH*, *TU* and *TG* are superimposed deformations. Load factors for *PS*, *CR*, and *SH*, are as shown in Table 3.5-3. In non-segmental structures: *PS*, *CR*, and *SH* are symbolically factored by a value of 1.0 in the strength limit state but are actually designed for in the service limit state. For substructure in the strength limit state, the value of 0.50 for γ_{PS}, γ_{CR}, γ_{SH}, and γ_{TU} may be used when calculating force effects in non-segmental structures, but shall be taken in conjunction with the gross moment of inertia in the columns or piers. The larger of the values provided for load factor of *TU* shall be used for deformations and the smaller values for all other effects. The calculation of displacements for *TU* loads utilizes a factor greater than 1.0 to avoid under sizing joints, expansion devices, and bearings.

The current AASHTO LRFD requires a load factor of 1.2 on *CR*, *SH*, and *TU* deformations, and 0.5 on other *CR*/*SH*/*TU* force effects. The lower value had been rationalized as dissipation of these force effects over time, particularly in the columns and piers.

Changing the load factors for creep and shrinkage is not straight-forward because *CR*, *SH* are "superimposed deformations", that is, force effects due to a change in material behavior that cause a change in the statical system. For safety and simplicity in design, they are treated as loads--despite not being measurable at time t = 0. However, behavior is nonlinear and application of the load factor must also be considered. Some software will run service load analysis twice: once with and once without *CR*, *SH* effects. The *CR* and *SH* can then be isolated by subtracting the results of the two runs. Other software will couple the *CR* and *SH* with the dead load, giving a shrinkage- or creep-adjusted dead load.

The proposed compromise is to assign creep and shrinkage the same load factor as the *DC* loads but permit a factor of 1.0 if the project-specific creep coefficient can be determined and is then used in the linear analysis software.

Thermal and shrinkage loadings are induced by movements of the structure and can result from several sources. Movements due to temperature changes are calculated using coefficients of thermal expansion of 0.000006 feet/foot per degree for concrete and 0.0000065 feet/foot per degree for steel. Reinforced concrete shrinks at the rate of 0.0002 feet/foot.

3.16 Other Loads

3.16.1 Buoyancy

The effects of submergence of a portion of the substructure are to be calculated, both for designing piling for uplift and for realizing economy in footing design.

3.16.2 Collision Force on Bridge Substructure

See AASHTO LRFD 3.6.5 and 3.14

3.16.3 Collision Force on Traffic Barrier

See AASHTO LRFD 3.6.5.1

3.16.4 Force from Stream Current, Floating Ice, and Drift

See AASHTO LRFD 3.9

3.16.5 Ice Load

In accordance with WSDOT HQ Hydraulics Office criteria, an ice thickness of 12″ shall be used for stream flow forces on piers throughout Washington State.

3.16.6 Uniform Temperature Load

The design thermal movement associated with a uniform temperature change may be calculated using the ranges of temperature as specified herein. The temperature ranges shown below reflect the difference between the extended lower and upper boundary to be used to calculate thermal deformation effects.

- Concrete Bridges (All Regions): 0° to 100°
- Steel Bridges (Eastern Washington): −30° to 120°
- Steel Bridges (Western Washington): 0° to 120°

3.16.7 Vehicular Collision Force: CT

Abutments and piers located within the clear zone as defined by the AASHTO *Roadside Design Guide* shall be investigated for collision. Collision shall be addressed by either providing structural resistance or by redirecting or absorbing the collision load. The provisions of AASHTO LRFD 2.3.2.2.1 shall apply as appropriate.

Where the design choice is to provide structural resistance, the pier or abutment shall be designed for an equivalent static force of 600 kip, which is assumed to act in a direction between 0 and 15 degrees with the edge of the pavement in a horizontal plane, between 2.0 and 5.0 ft above the ground, whichever produces the critical shear or moment in the pier component and the connections to the foundation or pier cap.

Each of the following substructure components are considered to have adequate structural resistance to bridge collapse due to vehicular impacts:

1. Substructure components that are backed by soil (e.g., abutments).
2. Reinforced concrete pier components that are at least 3.0-ft thick and have a concrete cross-sectional area greater than 30.0 ft^2 as measured in the horizontal plane at all elevations from the top of the pier foundation to a height of at least 5.0 ft above the grade.

3. Pier systems with three or more columns where the designer shows by calculation that the superstructure will not collapse with one column missing when subjected to the full dead load with a 1.1 load factor and the live load in the permanent travel lanes with a load factor of 1.0.
4. Piers supporting a bridge superstructure where it is shown by calculation that the superstructure will not collapse with one column missing when subjected to the full dead load with a 1.1 load factor and the live load in the permanent travel lanes with a load factor of 1.0.
5. Pier walls and multi-column piers with struts between columns that have been designed and detailed as longitudinal traffic barriers according to Section 13.

Where the design choice is to redirect or absorb the collision load, protection shall consist of one of the following:

- For new or retrofit construction, a minimum 42.0-in. high crash tested rigid TL-5 barrier, as specified in AASHTO LRFD Section 13, located such that the top edge of the traffic face of the barrier is 3.25 ft or more from the face of the pier component being protected.
- For retrofit construction, a minimum 42.0-in. high crash tested rigid TL-5 barrier may be placed closer than 3.25 ft from the top edge of the traffic face of the barrier to the nearest traffic face of the pier component being protected when there is no other practical option. Such rigid barriers shall be structurally and geometrically capable of surviving the crash test as specified in AASHTO LRFD Section 13.

3.16.8 Bridges Subjected to Tsunami Effects

The AASHTO *Guide Specifications for Bridges Subject to Tsunami Effects* are intended for the design and construction of conventional bridges to resist the effects of tsunami waves. The conventional bridges are taken as those that:

- Have slab, beam, or box girder superstructures
- Are supported by pier and abutment substructures
- Are founded on shallow or deep foundations
- Have minimal changes in elevation along the bridge length
- Are straight in plan view (including skewed but not curved)

The tsunami design requirements for non-conventional bridges shall be considered on a case-by-case basis and consultation with the Bridge and Structures Office.

Determination of whether the tsunami hazard applies shall be made at the bridge preliminary plan or conceptual plan stage using the above-mentioned tools and shall be noted in the Job File. Bridge preliminary plan or conceptual plan shall specify the most probably bridge soffit elevation and superstructure depth, and substructure/foundation type for tsunami hazard determination.

Where possible, all new bridges shall be designed so that the tsunami wave does not contact the superstructure. If this is not possible the bridge shall be designed to ensure that the bridge can resist the tsunami hazard loading as specified in the AASHTO *Guide Specifications for Bridges Subject to Tsunami Effects*.

A bridge superstructure shall be considered for the tsunami hazard if the inundation at Mean High Water (MHW) is high enough to contact the bridge superstructure soffit based on the modeled inundation depth. Substructure design should consider loading applied through the superstructure and loads applied directly to the substructure. Both scour and geotechnical hazards due to tsunami inundation shall be investigated during final design.

The critical tsunami parameters including wave direction, velocity and depth, can be obtained from the Washington State Department of Natural Resources (DNR), at: Tsunami Hazard Maps - WA or Tsunami Inundation Database Interactive Web Portal at: Tsunami Inundation Database Portal — The B. John Garrick Institute for the Risk Sciences (ucla.edu).

The performance requirement for bridges subjected to tsunami effects is life safety no-collapse as defined in Chapter 4 for seismic performance requirement. Higher performance for post-tsunami serviceability may be considered on a case-by-case basis for recovery bridges as defined in Chapter 4.

Simultaneous consideration of tsunami and seismic, or tsunami and scour are not recommended tsunami design of conventional bridges.

The provisions outlined in Section 7 of the AASHTO *Guide Specifications for Bridges Subject to Tsunami Effects* for reduction of tsunami loading, accommodation of forces, geometric proportioning, venting, and sacrificial elements could be used with the approval of the Bridge Design Engineer regardless of the contracting method.

3.99 References

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, November 2020.
2. AASHTO Guide Specifications for Bridges Subjected to Tsunami Effects, 1st Edition, January 2022.