Steel Bridge Design Handbook

CHAPTER 7

Loads and Load Combinations

February 2022

Smarter.
Stronger.
Steel.

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Foreword

The Steel Bridge Design Handbook covers a full range of topics and design examples to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. The Handbook has a long history, dating back to the 1970s in various forms and publications. The more recent editions of the Handbook were developed and maintained by the Federal Highway Administration (FHWA) Office of Bridges and Structures as FHWA Report No. FHWA-IF-12-052 published in November 2012, and FHWA Report No. FHWA-HIF-16-002 published in December 2015. The previous development and maintenance of the Handbook by the FHWA, their consultants, and their technical reviewers is gratefully appreciated and acknowledged.

This current edition of the Handbook is maintained by the National Steel Bridge Alliance (NSBA), a division of the American Institute of Steel Construction (AISC). This Handbook, published in 2021, has been updated and revised to be consistent with the 9th edition of the AASHTO LRFD Bridge Design Specifications which was released in 2020. The updates and revisions to various chapters and design examples have been performed, as noted, by HDR, M.A. Grubb & Associates, Don White, Ph.D., and NSBA. Furthermore, the updates and revisions have been reviewed independently by Francesco Russo, Ph.D., P.E., Brandon Chavel, Ph.D., P.E., and NSBA.

The Handbook consists of 19 chapters and 6 design examples. The chapters and design examples of the Handbook are published separately for ease of use, and available for free download at the NSBA website, www.aisc.org/nsba.

The users of the Steel Bridge Design Handbook are encouraged to submit ideas and suggestions for enhancements that can be implemented in future editions to the NSBA and AISC at solutions@aisc.org.

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Steel Bridge Design Handbook:
Loads and Load Combinations

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1.0 INTRODUCTION

Sections 1 and 3 of the *AASHTO LRFD Bridge Design Specifications*, *9th Edition*, (referred to herein as AASHTO LRFD BDS) [1] discuss various aspects of loads. The load factors are tabulated in Table 3.4.1-1 of the AASHTO LRFD BDS, and are associated with various limit states and further various load combinations within the limit states. This volume discusses the various components of load and provides information beyond that contained in the AASHTO LRFD BDS that will be useful to the designer. It also discusses and reviews the various limit-state load combinations to assist the designer in avoiding non-governing load combinations.

2.0 LOADS

Loads within the context of the AASHTO LRFD BDS are categorized as permanent or transient loads. This categorization is necessary because the AASHTO LRFD BDS represents a "reliability-based" design methodology which evaluates the statistical probability of occurrence of the loading effects (and the statistical reliability of the resistance effects). In other words, the specifications recognize there is some uncertainty associated with the estimated values of the loading effects; loads can be larger than the nominal value (the value of load calculated as specified in the AASHTO LRFD BDS) or smaller than the nominal value. The load factors specified in the AASHTO LRFD BDS reflect this uncertainty by adjusting the nominal estimated value of various loads.

Transient loads occur intermittently, and thus the smallest value of a transient load is, by definition, zero. Permanent loads occur continuously, and generally any variation in the magnitude of a permanent load from its estimated value is due to either an intentional change in conditions or a misestimation of the value. However, in some cases permanent loads can cause relieving force effects in the structure, so it is necessary to consider a minimum estimate of the load in addition to considering a maximum estimate. Thus, for permanent loads there are maximum load factors (generally greater than 1.0) and minimum load factors (generally less than 1.0).

2.1 Permanent Loads

2.1.1 General

Permanent loads are defined by AASHTO (American Association of State Highway and Transportation Officials) as "loads and forces that are, or are assumed to be, either constant upon completion of construction or varying only over a long time interval". The AASHTO LRFD BDS specifies 10 types of permanent loads, which include direct gravity loads, loads indirectly caused by gravity loads, "locked-in" loads resulting from the construction process, and certain loads due to superimposed deformations. This section describes each of the 10 permanent load types as well as their applicability to the design of a bridge structure.

2.1.2 Gravitational Dead Loads

DC represents the dead load of structural components, as well as any non-structural attachments.

Component dead loads associated with composite girder-slab bridges consist of non-composite and composite components, typically termed DC_1 and DC_2, respectively. Dead loads applied to the non-composite cross section (i.e., the girder alone) include the self-weight of the girder and the weight of the wet concrete, forms and other construction loads typically required to place the deck. The concrete dead load should include allowances for haunches over the girders. Where steel stay-in-place formwork is used, the designer shall account for the steel form weight and any additional concrete in the flues of the formwork.

When calculating the distribution of the weight of plastic concrete (i.e., concrete prior to hardening) to the girders, including that of any integral sacrificial wearing surface, the formwork is typically assumed to be simply supported between beams and cantilevered beyond the exterior beams.

Component dead loads applied to the composite cross section (i.e., the girder with the composite slab) include the weight of any curb, rail, sidewalk or barrier placed after the deck concrete has hardened, as well as any appurtenances such as lights, signs, walls, etc.

DW represents the dead load of additional non-integral wearing surfaces, future overlays, and any utilities supported by the bridge.

An allowance for a future wearing surface over the entire deck area between the gutter lines is typically included as a composite dead load. Many Owner-agency design policies specify the value of the future wearing surface allowance.

The dead loads applied after the deck has cured, DC_2 and DW, are sometimes termed superimposed dead loads.

Permanent loads which occur more or less uniformly over the width of the bridge deck, such as wearing surfaces, are typically assumed to be distributed equally among all girders. Concentrated permanent loads, such as the self-weight of barrier rails, sidewalks, median barriers, etc. are typically assumed to be distributed among a limited number of nearby girders. For example, it is common to assume that the self-weight of a barrier rail is distributed to only the exterior girder, or to a small number of nearby girders, rather than being uniformly distributed among all girders; many Owner-agencies design policies provide guidance on acceptable assumptions for the distribution of the self-weight of barrier rails. In some cases, such as wider bridges, staged construction, or heavier barrier rails, walls, or utilities, the bridge designer may consider performing a refined analysis to more accurately determine the distribution of superimposed dead loads.

EL represents the accumulated lock-in, or residual, force effects resulting from the construction process, including the jacking apart of components in cantilever construction.

EV represents the vertical earth pressure from the dead load of earth fill.

2.1.3 Earth Pressures (see Article 3.11)

EH represents the load due to horizontal earth pressure.

ES represents the load due to earth pressure from a permanent earth surcharge (e.g., an embankment).

DD represents the loads developed along the vertical sides of a deep-foundation element tending to drag it downward (called “downdrag”), typically due to consolidation of soft soils underneath embankments reducing its resistance. Deep foundations (i.e., driven piles and drilled shafts) through unconsolidated soil layers may be subject to downdrag, also known as negative skin friction. If possible, the bridge designer should detail the deep foundation to mitigate the effects of downdrag; otherwise, it is necessary to design considering downdrag.

As discussed later in this document, the permanent force effects in superstructure design are factored by the maximum permanent-load load factors almost exclusively. The most common exception is the check for uplift of a bearing. In substructure design, the permanent force effects are routinely factored by the maximum or minimum permanent-load load factors from the AASHTO LRFD BDS Table 3.4.1-2 as appropriate.

2.1.4 Permanent Loads due to Superimposed Deformations (See Article 3.12)

CR represents the load induced by creep in concrete or wood.

SH represents the load induced by differential shrinkage between concretes of different age or composition, and between concrete and other materials, such as steel and wood.

PS represents the load due to secondary forces from post-tensioning for strength limit states and/or total prestress forces for service limit states.

2.2 Transient Loads

2.2.1 General

Transient loads are defined by AASHTO as "loads and forces that can vary over a short time interval relative to the lifetime of the structure" [1]. The AASHTO LRFD BDS recognizes 18 transient loads. The transient loads include loading effects associated with vehicular or pedestrian loading (LL, IM, BR, CE, LS, PS), environmental conditions such as thermal, wind, and water loading (TG, TU, WS, WL, WA, IC), friction in sliding bearings (FR), settlement (SE), and various extreme event conditions (BL, CT, CV, EQ). This section describes the transient loads recognized in the AASHTO LRFD BDS.

2.2.2 Live Loads (see Article 3.6)

LL represents the vertical gravity loads due to vehicular traffic on the roadway, treated as static loads.

For short and medium span bridges, vehicular live load is the most significant component of load.

The HL-93 live-load model is a notional load in that it is not a true representation of actual truck weights. Instead, the force effects (i.e., the moments and shears) due to the superposition of vehicular and lane load within a single design lane are considered to be a reasonably accurate representation of the force effects due to actual trucks.

The components of the HL-93 notional load are:

- a vehicle, either a 72-kip three-axle design truck (equivalent to the historical HS20-44 truck of the *AASHTO Standard Specifications for Highway Bridges* (referred to herein as the Standard Specifications) [2] or a 50-kip design tandem (equivalent to the Alternate Loading of the Standard Specifications); and

- a 0.64 k/ft uniformly distributed lane load (similar to the lane load of the Standard Specifications, but acting concurrently with the vehicle without any of the previous associated concentrated loads).

The force effects of the traditional HS-20 truck alone are less than that of the AASHTO legal loads. Thus, a heavier vehicle was deemed appropriate for design. Originally, a longer 57-ton vehicle (termed the HTL-57) was developed to model the force effects of trucks on our nation's highways at the time of the development of early drafts of the 1st Edition of the AASHTO LRFD BDS. Ultimately, however, it was deemed objectionable to specify a super-legal truck in the national design specifications. Instead, the concept of superimposing the design vehicle force effects and the design lane force effects to produce moments and shears representative of real trucks on the highways was developed for later drafts and subsequent editions of the AASHTO LRFD BDS. The moments and shears produced by the HL-93 notional load model are essentially equivalent to those of the more realistic 57-ton truck.

The multiple presence factor of 1.0 for two loaded lanes, as given in Table 3.6.1.1.2-1, is the result of the AASHTO LRFD BDS calibration for the notional load, which has been normalized relative to the occurrence of two side-by-side, fully correlated, or identical, vehicles. The multiple presence factor of 1.2 for one loaded lane should be used where a single design tandem or single design truck governs, such as in overhangs, decks, etc. The multiple-presence factors should never be applied to fatigue loads nor any other vehicle of relatively known weight such as a legal or permit load.

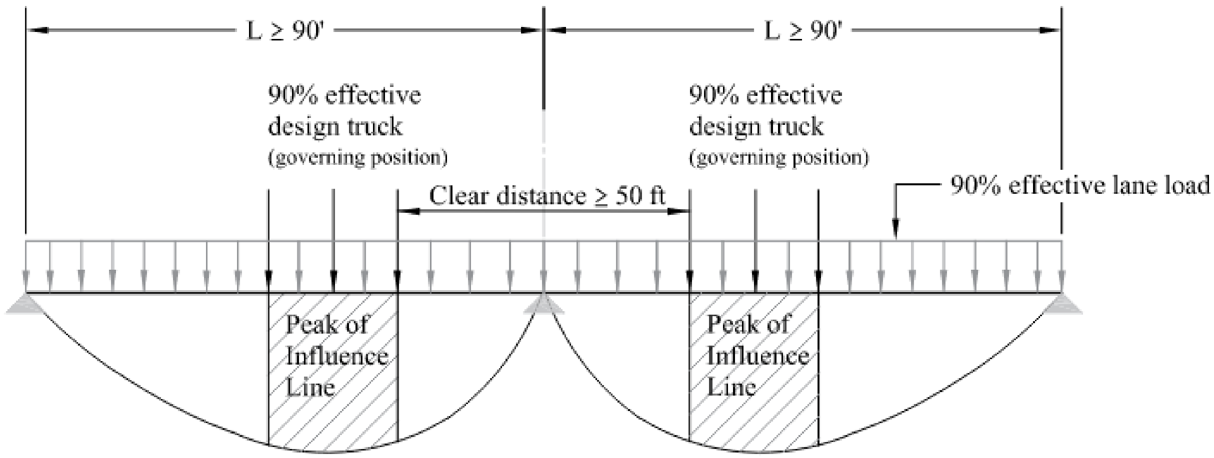
The AASHTO LRFD BDS retains the traditional design lane width of 12 ft and the traditional spacing of the axles and wheels of the HS-20 truck. Both vehicles (the design truck and design tandem) and the lane load occupy a 10-ft width placed transversely within the design lane for maximum effect, as specified in Article 3.6.1.3.

The combination of the lane load and a single vehicle (either a design truck or a design tandem) does not always adequately represent the real-life loading effect in negative-moment regions for a variety of span lengths. Thus, a special load case has been specified in the AASHTO LRFD BDS to calculate these effects. 90 percent of the effect of two design trucks, with a fixed rear axle spacing of 14 ft and a clear distance not less than 50 ft between the lead axle of one truck and the rear axle of the other truck, superimposed upon 90 percent of the effect of the lane load, all within a single design lane approximates a statistically valid representation of negative moment and interior reactions due to heavy trucks. This sequence of highway loading is specified for negative moment and interior reactions only. This sequence is not extended to other structures or portions of structures. While not explicitly mentioned, it is generally assumed that "interior reactions" includes piers and supports at joint locations. While each individual bearing support is a simple reaction, similar to that of an abutment, two trucks can still be present on either side of the joint, causing increased reactions similar to pier supports at continuous span locations. The Owner's interpretation of these provisions should be verified during design.

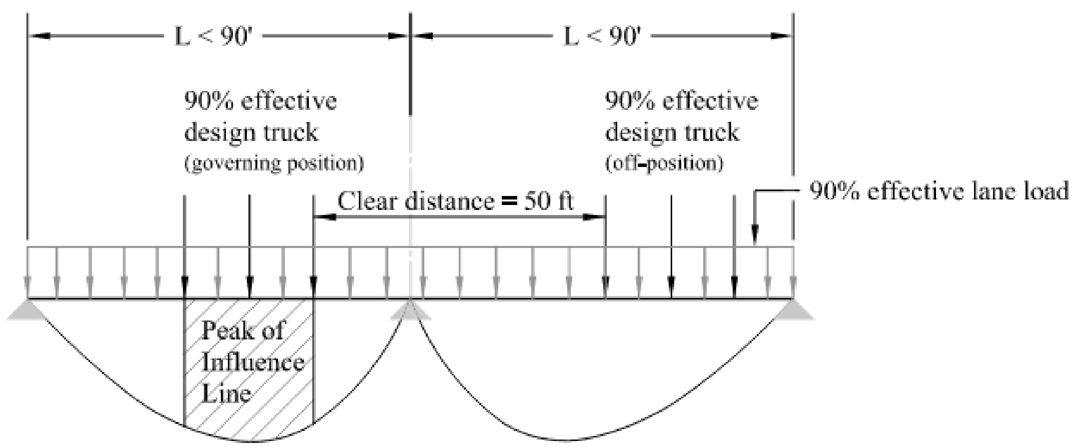
In positioning the two trucks to calculate negative moment or the interior reaction over an interior support, spans should be at least approximately 90 ft in length to be able to position a truck in each span's governing position (over the peak of the influence line, see Figure 1a). If the spans are larger than 90 ft in length, the trucks remain in the governing positions but, if they are smaller than 90 ft, the maximum force effect can only be attained by trial-and-error with either one or both trucks in off-positions (i.e., non-governing positions for each individual span away from the peak of the influence line, see Figure 1b). This is not to say that the special two-truck load case does not govern, just that the trucks will not be positioned over the maximum influence-line ordinate. The truck in the first span of the two-span continuous bridge (in Figure 1b) is in the governing position for the span; the truck in the second span falls to the right of the spans governing position based upon the influence line for negative moment over the pier.

The AASHTO LRFD BDS defines the notional live load for fatigue for a particular bridge component by specifying both a magnitude and a frequency. The magnitude of the fatigue load is consistent with the design truck or axles specified in Article 3.6.1.2.2, but with a constant spacing of 30 ft between the 32.0 kip axles. The frequency of the fatigue load is taken as the greatest single-lane average daily truck traffic ($ADTT_{SL}$) and is used for all components of the bridge, even though some lanes may carry a lesser number of trucks. When information regarding the directionality of truck traffic is unavailable, designing for 55% of the bi-directional ADTT is recommended.

The design live load applied for checking strength, service and fatigue limits is not the same configuration used for checking the optional AASHTO LRFD BDS live load deflections limits in Article 2.5.2.6.2. Unless dictated otherwise by the Owner, the live load for the optional deflection check is specified in Article 3.6.1.3.2, which uses the maximum result of the design truck alone or 25% of the design truck combined with the design lane load. Per Article 2.5.2.6.2, the live load should include the dynamic load allowance, IM, and multiple presence factors per Article 3.6.1.1.2.



a) Governing truck positions for spans larger than 90 ft



b) Possible truck positions for maximum negative moment effects for spans smaller than 90 ft

Figure 1 Influence line for a two-span continuous bridge

PL represents the vertical gravity loads due to pedestrian traffic on sidewalks, taken as 75 psf for sidewalks wider than 2.0 feet.

IM represents the dynamic load allowance to amplify the force effects of statically applied vehicles to represent moving vehicles, traditionally called impact. Note that the dynamic load allowances (IM) specified in Article 3.6.2.1 is applicable only to the design trucks, the design tandems, and the fatigue truck load, excluding centrifugal and braking forces. The dynamic load allowance should not be applied to the uniformly distributed lane load. Note also that the dynamic load allowance need not be applied to foundation components that are entirely below ground.

LS represents the horizontal earth pressure resulting from the surcharge effect of vehicular traffic on the ground surface above an abutment or wall.

BR represents the horizontal vehicular braking force. Two formulations for the braking force are provided; the design braking force is taken as the greater of the two values. Braking forces are typically distributed among the substructure units based on bearing fixity and/or bearing stiffness and substructure stiffness. For very simple configurations, the distribution can often be determined by inspection. For more complicated configurations with multiple spans and multiple supports a simplified relative stiffness analysis may be useful determining the distribution of the braking force among the substructure units. Note that the bearings at each substructure unit should also be designed to resist the load being transferred to those substructure units.

CE represents the horizontal centrifugal force from vehicles on a curved roadway.

2.2.3 Water Loads (see Article 3.7)

WA represents the pressure due to differential water levels, stream flow, buoyancy or wave action. Simplified equations are provided in the AASHTO LRFD BDS for most routine situations; unique cases may warrant a more refined analysis.

2.2.4 Wind Loads (see Article 3.8)

WS represents the horizontal and vertical pressure on superstructure or substructure due to wind.

WL represents the horizontal pressure on vehicles due to wind.

For most routine bridges, transverse wind loads are distributed among substructure units based on tributary span length assumptions, while longitudinal wind loads are distributed in a manner similar to that described for braking forces in Section 2.2.2. For more complicated designs, a more refined analysis may be warranted to determine distribution of wind loads to the various substructure units.

2.2.5 Extreme-Event Loads

BL represents the intentional or unintentional force due to construction blasting (see Article 3.15).

EQ represents loads due to earthquake ground motions (see Article 3.10).

CT represents horizontal impact loads on abutments or piers due to vehicles or trains (see Article 3.6.5).

CV represents horizontal impact loads due to aberrant ships or barges (see Article 3.14).

IC represents the horizontal static and dynamic forces due to ice action (see Article 3.9).

2.2.6 Transient Loads due to Superimposed Deformations (see Article 3.12)

TU represents the uniform temperature change due to seasonal variation.

TG represents the temperature gradient due to exposure of the bridge to solar radiation.

SE represents the effects of settlement of substructure units on the superstructure.

Settlement and temperature (SE, TU and TG) are classified as transient loads due to superimposed deformations which, if restrained, will result in force effects. For example, when the expansion of a bridge superstructure during a uniform increase in the temperature of the structure is restrained at the bearings, a compression force will be induced in the superstructure and shear forces will be induced in the restraining substructure units.

Typically, superimposed deformations are not considered in the design of typical steel girder bridges other than the use of TU to size joints and bearings. However, when more than one pier is provided with fixed bearings in a multi-span continuous design, shear forces will be induced at the fixed piers and should be accounted for in the substructure design.

2.2.7 Friction Forces (see Article 3.13)

FR represents the frictional forces on sliding surfaces from structure movements.

The bridge designer should adjust the frictional forces from sliding bearings to account for unintended additional friction forces due to the future degradation of the coefficient of friction of the sliding surfaces. Consider the horizontal force due to friction conservatively. Include friction forces where design loads would increase, but neglect friction forces where design loads would decrease.

Typically, friction forces enter only into the design of bearings and substructures for typical steel girder bridges.

2.2.8 Other Loads (see Articles 3.4.2 and 3.4.3.1)

Two other load components are discussed in the AASHTO LRFD BDS but are not explicitly included in the table of load combinations. As such, these loads are not included in any load combinations but should be applied at the discretion of the designer.

Construction loads are not explicitly specified, as their magnitude and placement can be very contractor and project specific. Nonetheless, the AASHTO LRFD BDS and the *AASHTO Guide Design Specifications for Bridge Temporary Works* (referred to herein as the Specifications for

Bridge Temporary Works) [4] suggest minimum load factors for the various load components during construction as summarized below in Table 1. Article 3.4.2.1 from the AASHTO LRFD BDS states that these load factors “should not relieve the contractor of responsibility for safety and damage control during construction.” Furthermore, the *Specifications for Bridge Temporary Works* states that the given loads “are not all-inclusive; therefore, their selection will require judgment in many situations. Design should be based on the load combination causing the most unfavorable effect.”

An important construction-specific strength limit load case required in AASHTO LRFD involves the checking of stability and strength of primary steel superstructure components during construction. Loads which are applied to the fully erected steelwork shall be considered for this additional load combination. This is typically called the “constructability checks” for the girder and are addressed in Article 6.10.3 of the AASHTO LRFD BDS.

Jacking forces during bearing replacement also fall into this category of loads discussed but not included formally in the load combinations. The AASHTO LRFD BDS recommends that the factored design force be equal to 1.3 times the permanent-load reaction at the bearing. If the jacking occurs under traffic, the live-load reaction times the load factor for live load should also be included in the factored design force.

Table 1 Minimum load factors during construction

LOAD COMPONENT	LOAD COMBINATION	LOAD FACTOR
Dead Load	Strength I & III	1.25
Construction Loads Including Dynamic Effects	Strength I	1.50
Wind Load	Strength III	See Note 1
Dead Load & Construction Loads Including Dynamic Effects[2]	--	1.40
1. Strength III load factor for wind as specified by the Owner 2. Applies only to primary steel superstructure components		

3.0 LOAD COMBINATIONS

3.1 Reliability-based Design

The AASHTO LRFD BDS is based upon the theory of structural reliability, and the strength load combinations are developed to achieve uniform reliability of structural components regardless of the type of material used. When the load factors and the resistance factors of the AASHTO LRFD BDS are applied in design, a uniform level of reliability or safety is achieved. The magnitudes of the factors derived to achieve this uniform safety are the major difference between the Load and Resistance Factor Design (LRFD) method used in the AASHTO LRFD BDS and Load Factor Design (LFD) method used in the previous Standard Specifications. Further details, including the basis of the LRFD calibration, can be found in NCHRP Report 368 [5].

3.2 Limit States

3.2.1 Basic LRFD Equation

Components and connections of a bridge must be designed to satisfy the basic LRFD equation for all limit states:

$$\sum_i \eta_i \gamma_i Q_i \leq \phi R_n = R_r \quad \text{(Equation 1.3.2.1-1)}$$

where:

γ_i = load factor

Q_i = load or force effect

ϕ = resistance factor

R_n = nominal resistance

η_i = load modifier as defined in Equations 1.3.2.1-2 and 1.3.2.1-3

R_r = factored resistance: ϕR_n

The left-hand side of Equation 1.3.2.1-1 in the AASHTO LRFD BDS is the sum of the factored load (force) effects acting on a component or connection; the right-hand side is the factored nominal resistance of the component or connection for those effects. The equation must be considered for all applicable limit state load combinations. Similarly, the equation is applicable to both superstructures and substructures.

For the strength limit states, the AASHTO LRFD BDS is basically a hybrid design code in that, for the most part, the force effect on the left-hand side of the LRFD Equation is based upon elastic structural response, while resistance on the right-hand side of the LRFD Equation can take advantage of inelastic response principles. The AASHTO LRFD BDS has adopted the hybrid nature of strength design on the assumption that the inelastic aspects of structural performance are generally relatively insignificant because of non-critical redistribution of force effects. This non-criticality is developed by providing adequate redundancy and ductility of the structures.

3.2.2 Load Modifiers

The load modifier η_i relates the factors η_D, η_R and η_I to ductility, redundancy and operational importance, respectively. The location of η_i on the load side of the AASHTO LRFD BDS Equation 1.3.2.1-1 may appear counterintuitive as ductility, redundancy and operational importance seem to be more related to resistance than to load. However, η_i is on the load side for a logical reason. When η_i modifies a maximum load factor, it is the product of the factors as indicated in Equation 1.3.2.1-2; when η_i modifies a minimum load factor, it is the reciprocal of the product as indicated in Equation 1.3.2.1-3. The AASHTO LRFD BDS factors, η_D, η_R and η_I are based on a 5% stepwise positive or negative adjustment, reflecting unfavorable or favorable conditions. These factors are somewhat arbitrary; their significance is in their presence in the AASHTO LRFD BDS and not necessarily in the accuracy of their magnitude. The AASHTO LRFD BDS factors reflect the desire to promote redundant and ductile bridges.

In practice, η_i values of 1.00 are generally used for all limit states, because bridges designed in accordance with the AASHTO LRFD BDS demonstrate traditional levels of redundancy and ductility. Rather than penalize less redundant or less ductile bridges, such bridges are typically not acceptable. On a case-by-case basis, the Owner can designate a bridge to be of operational importance and specify an appropriate value of η_I. Some Owners may also designate certain structure types as having reduced redundancy or ductility and specify appropriate values of η_D or η_R, again on a case-by-case basis.

The load modifier accounting for Operational Importance (η_I), as specified in Article 1.3.5, should not be confused with the importance categories for vessel collision of Article 3.14 nor the bridge category classifications for seismic design of Article 3.10.

3.2.3 Load Factors

3.2.3.1 Development of Load Factors

The load factors were defined using the load statistics (mean and coefficient of variation) so that each factored component of load has an equal probability of being exceeded. The magnitudes of the individual load factors by themselves have no significance. Their relative magnitude in comparison with one another indicates the relative uncertainty of the load component. For example, in the Strength I load combination, the live-load load factor of 1.75 indicates that live load has more uncertainty than component dead load which is assigned a maximum load factor of only 1.25.

3.2.3.2 Maximum/Minimum Permanent Load Factors

In Table 3.4.1-1, the variable γ_P represents load factors for all of the permanent loads in Strength limit state load combinations, shown in the first column of load factors. This variable γ_P reflects that the Strength limit state load factors for the various permanent loads are not single constants, but they can have two extreme values. Table 3.4.1-2 provides these two extreme values for the various permanent load factors, maximum and minimum. Permanent loads are always present on the bridge, but the nature of uncertainty is that the actual loads may be more or less than the nominal specified design values. Therefore, maximum and minimum load factors reflect this uncertainty.

The designer should select the appropriate maximum or minimum permanent-load load factors (γ_P) to produce the more critical load effect. For example, in continuous superstructures with relatively short-end spans, transient live load in the end span causes the bearing to be more compressed, while transient live load in the second span causes the bearing to be less compressed and perhaps lift up. To check the maximum compression force in the bearing, place the live load in the end span and use the maximum DC load factor of 1.25 for all spans. To check possible uplift of the bearing, place the live load in the second span and use the minimum DC load factor of 0.90 for all spans.

Superstructure design uses the maximum permanent-load load factors almost exclusively, with the most common exception being uplift of a bearing as discussed above. The Standard Specifications treated uplift as a separate load combination. With the introduction of maximum and minimum load factors, the AASHTO LRFD BDS has generalized load situations such as uplift where a permanent load (in this case a dead load) reduces the overall force effect (in this case a reaction). Permanent load factors, either maximum or minimum, must be selected for each load combination to produce extreme force effects.

Substructure design routinely uses the maximum and minimum permanent-load load factors from Table 3.4.1-2. An illustrative yet simple example is a spread footing supporting a cantilever retaining wall. When checking bearing, the weight of the soil (EV) over the heel is factored up by the maximum load factor, 1.35, because greater EV increases the bearing pressure making the limit state more critical. When checking sliding, EV is factored by the minimum load factor, 1.00, because lesser EV decreases the resistance to sliding again making the limit state more critical. The application of these maximum and minimum load factors is required for substructure and foundation design.

3.2.3.3 Load Factors for Superimposed Deformations due to Uniform Temperature Change (TU)

The load factors for the superimposed deformations related to TU for the Strength limit states have two specified values -- a load factor of 0.5 for most effects, and a load factor of 1.2 for the calculation of deformation. The greater value of 1.2 is used to calculate unrestrained deformations (e.g., a simple span expanding freely with rising temperature). For example, the 1.2

factor is typically used to calculate thermal movements of the superstructure when sizing expansion joints.

The lower value of 0.5 is used for the elastic calculation of stress and reflects the inelastic response of the structure due to restrained deformations. For example, 0.5 times the temperature rise would be used to elastically calculate the stresses in a constrained structure; this reflects a conservative estimate of the reduction in stiffness of the substructure unit to account for cracked section properties in concrete columns. Using 1.2 times the temperature rise in an elastic calculation would overestimate the stresses in the structure. The structure resists the temperature inelastically through redistribution of the elastic stresses. A third value should be used in certain situations for the TU load factor for Strength limit states when analyzing the substructure. Steel columns or piers shall use a load factor of 1.0. For concrete substructures, the 0.5 load factor (typically used for the calculation of stress) may be used for the calculation of the force effects corresponding to the gross moment of inertia of the columns or piers. The 1.0 load factor should be used in conjunction with a partially cracked moment of inertia, determined from refined analysis.

3.2.4 Strength Limit State Load Combinations

3.2.4.1 General

The load factors for the Strength load combinations are calibrated based upon structural reliability theory and represent the uncertainty of their associated loads. Larger load factors indicate more uncertainty; smaller load factors less uncertainty. The significance of the Strength limit state load combinations can be simplified as discussed in the following articles. The Commentary for Article 3.4.1 in the AASHTO LRFD BDS often provides valuable insights helpful in understanding the intent and appropriate use of each of these load combinations.

3.2.4.2 Strength I Load Combination

This load combination represents normal vehicular use of the bridge in its 75-year design life. During this live-load event, the effect of wind is considered to be negligible.

3.2.4.3 Strength II Load Combination

This load combination represents an owner-specified permit load model. This live-load event will have less uncertainty than random traffic and, thus, a lower live-load load factor. If the Owner does not specify a permit load for design purposes, this load combination need not be considered. During this live load event, the effect of wind is considered to be negligible.

3.2.4.4 Strength III Load Combination

This load combination is applicable to bridge structures exposed to the design wind speed at the location of the bridge. During high winds, it is unlikely that any significant live load would cross the bridge.

3.2.4.5 Strength IV Load Combination

This load combination represents an extra safeguard for bridge superstructures where the unfactored dead load exceeds seven times the unfactored live load. Thus, the only significant load factor would be the 1.25 dead-load maximum load factor. For additional safety, and based solely on engineering judgment, the AASHTO LRFD BDS has arbitrarily increased the load factor for DC to 1.5. This load combination need only be considered for superstructure components, and only when the unfactored dead-load force effect is more than seven times the unfactored live-load force effect. This load combination typically governs only for longer spans, greater than approximately 200 feet in length.

3.2.4.6 Strength V Load Combination

This load combination represents the simultaneous occurrence of normal vehicular use of the bridge and a 80 mph wind event, with load factors of 1.35 and 1.0 respectively.

3.2.4.7 Typical Strength Design Practice

For components not traditionally governed by wind force effects, the Strengths III and V Load Combinations should not govern. Unless Strengths II and IV as previously described are needed, for a typical multi-girder highway overpass the Strength I Load Combination will generally be the only combination requiring design calculations.

3.2.5 Service Limit State Load Combinations

3.2.5.1 General

Unlike the Strength limit state load combinations, the Service limit state load combinations are, for the most part, material specific.

3.2.5.2 Service I Load Combination

This load combination is applicable to normal operational use of the bridge, with a 70 mph wind and all loads taken at their nominal values. Service I is also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders.

3.2.5.3 Service II Load Combination

This load combination is applied for controlling permanent deformations of compact steel sections and the “slip” of slip-critical (i.e., friction-type) bolted steel connections due to vehicular live load. If a disproportionate number of permit loads is expected at a specific site, an increase in the Service II load factor should be considered.

3.2.5.4 Service III Load Combination

This load combination is applicable to the longitudinal analysis of tensile stresses in prestressed concrete superstructure components. The objective of Service III is to control cracking and principal tension in the webs of segmental concrete girders under vehicular traffic loads.

3.2.5.5 Service IV Load Combination

This load combination is only applicable for tensile stresses in prestressed concrete columns, with the intent to control cracking.

3.2.6 Extreme Event Limit State Load Combinations

The Extreme-Event limit states differ from the Strength limit states because the event for which the bridge and its components are designed has a greater return period than the 75-year design life of the bridge (or a much lower frequency of occurrence than the loads of the strength limit state load combinations). The following applies:

3.2.6.1 Extreme Event I Load Combination

This load combination is applied to earthquakes. The factor for live load (γ_{EQ}) shall be determined on a project-specific basis.

3.2.6.2 Extreme Event II Load Combination

This load combination is applied to various types of collisions, as well as check floods and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, CT. These collisions are typically from a vessel, vehicle or ice impacting the bridge's substructure.

3.2.7 Fatigue & Fracture Limit State Load Combinations

The Fatigue and Fracture limit states differ from any of the other combinations previously described because the focus is centered around a member subjected to countless repetitions (referred to as cycles) of a "normal" live load in an average climate, rather than a "worst-case" live load or during an extreme weather event. In the AASHTO LRFD BDS (8^{th} Edition, 2017), the load factors for Fatigue limit states were increased based on a reassessment of current truck traffic. The reassessment included statistical evaluation of extensive Weigh-In-Motion (WIM) data that more accurately reflects current truck traffic in the U.S. As a result of the increase in these load factors, designs that satisfied the fatigue provisions of previous editions of the LRFD Specifications and the older Standard Specifications may not satisfy the fatigue provisions of the current LRFD Specifications. See the discussion of fatigue and fracture limit states in NSBA's *Steel Bridge Design Handbook: Limit States* [6] for additional information, including the history of the development of the AASHTO fatigue provisions.

The Fatigue limit state applies restrictions to the stress range encountered in a member subject to an anticipated number of stress range cycles, while the Fracture limit state provides a set of material toughness requirements based on the AASHTO Materials Specifications [3]. Charpy V-notch impact energy requirements are provided in Table C6.6.2.1-1 of AASHTO LRFD BDS. The Fatigue limit state is intended to limit crack development and growth under repetitive live loads, preventing fracture during the design life of the bridge.

3.2.7.1 Fatigue I Load Combination

This fatigue and fracture load combination is related to infinite load-induced fatigue life. See the discussion of infinite fatigue life design in NSBA's *Steel Bridge Design Handbook: Limit States* [6] for future guidance on this load combination.

3.2.7.2 Fatigue II Load Combination

This fatigue and fracture load combination is related to finite load-induced fatigue life. See the discussion of finite fatigue life design in NSBA's *Steel Bridge Design Handbook: Limit States* [6] for future guidance on this load combination.

4.0 REFERENCES

1. AASHTO. *LRFD Bridge Design Specifications, 9th Edition.* Washington D.C.: American Association of State Highway and Transportation Officials, 2020.

2. AASHTO. *Standard Specifications for Highway Bridges, 17th Edition.* Washington D.C.: American Association of State Highway and Transportation Officials, 2002.

3. AASHTO. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing, 40th Edition.* Washington D.C.: American Association of State Highway and Transportation Officials, 2020.

4. AASHTO. *Guide Design Specifications for Bridge Temporary Works, 2nd Edition*. Washington D.C.: American Association of State Highway and Transportation Officials, 2017.

5. Nowak, A. S. *Calibration of LRFD Bridge Design Code*. NCHRP Report 368, Transportation Research Board, National Research Council, Washington, DC, 1999.

6. NSBA. *Steel Bridge Design Handbook: Limit States*. National Steel Bridge Alliance, 2021.