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Steel Bridge Design Handbook

Loads and Load Combinations

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16. Abstract
Sections 1 and 3 of the *AASHTO LRFD Bridge Design Specifications, 5th Edition* discuss various aspects of loads. The load factors are tabulated in Table 3.4.1-1 of the AASHTO LRFD and are associated with various limit states and further various load combinations within the limit states. This module discusses the various components of load and provides information beyond that contained in the AASHTO LRFD 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.

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Steel Bridge Design Handbook: Loads and Load Combinations

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FOREWORD

It took an act of Congress to provide funding for the development of this comprehensive handbook in steel bridge design. This 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 is based on the Fifth Edition, including the 2010 Interims, of the AASHTO LRFD Bridge Design Specifications. The hard work of the National Steel Bridge Alliance (NSBA) and prime consultant, HDR Engineering and their sub-consultants in producing this handbook is gratefully acknowledged. This is the culmination of seven years of effort beginning in 2005.

The new *Steel Bridge Design Handbook* is divided into several topics and design examples as follows:

- Bridge Steels and Their Properties
- Bridge Fabrication
- Steel Bridge Shop Drawings
- Structural Behavior
- Selecting the Right Bridge Type
- Stringer Bridges
- Loads and Combinations
- Structural Analysis
- Redundancy
- Limit States
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- Design Example: Three-span Continuous Curved Tub-Girder Bridge

These topics and design examples are published separately for ease of use, and available for free download at the NSBA and FHWA websites: http://www.steelbridges.org, and http://www.fhwa.dot.gov/bridge, respectively.

The contributions and constructive review comments during the preparation of the handbook from many engineering processionals are very much appreciated. The readers are encouraged to submit ideas and suggestions for enhancements of future edition of the handbook to Myint Lwin at the following address: Federal Highway Administration, 1200 New Jersey Avenue, S.E., Washington, DC 20590.

Myint Lwin

M. Myint Lwin, Director
Office of Bridge Technology

1.0 INTRODUCTION

Sections 1 and 3 of the *AASHTO LRFD Bridge Design Specifications*, *5th Edition*, (referred to herein *as* AASHTO LRFD (5th Edition, 2010)) (1) discuss various aspects of loads. The load factors are tabulated in Table 3.4.1-1 of the AASHTO LRFD (5th Edition, 2010), and are associated with various limit states and further various load combinations within the limit states. This module discusses the various components of load and provides information beyond that contained in the AASHTO LRFD (5th Edition, 2010) 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 (5th Edition, 2010) are categorized as permanent or transient loads. This categorization is necessary due to the probabilistic nature of the specifications. Due to uncertainty, loads can be larger than the nominal value (the value of load calculated as specified in the AASHTO LRFD (5th Edition, 2010)) or less than the nominal value. In the case of transient loads, lower values are of no consequence since not placing the transient load on the structure at all will govern. Permanent loads are always there however, so lesser values may be important (for example, when considering retaining wall sliding or overturning). For permanent loads, minimum load factors are specified as well as maximum load factors. Thus, the categorization of loads as permanent or transient is significant within the context of a probability-based specification.

2.1 Permanent Loads

2.1.1 General

Permanent loads are loads that are always present in or on the structure and do not change in magnitude during its life. The AASHTO LRFD (5th Edition, 2010) specifies seven components of permanent loads, which are either direct gravity loads or caused by gravity loads. Prestressing is considered, in general, to be part of the resistance of a component and has been omitted from the list of permanent loads in Section 3. However, when designing anchorages for prestressing tendons, the prestressing force is the only load effect, and it should appear on the load side of the AASHTO LRFD (5th Edition, 2010) Equation.

2.1.2 Gravitational Dead Loads

DC is the dead load of all of the components of the superstructure and substructure, both structural and non-structural.

Component dead loads associated with composite girder-slab bridges consist of composite and non-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.

For the distribution of the weight of plastic concrete to the girders, including that of an integral sacrificial wearing surface, assume that the formwork is simply supported between interior beams and cantilevered over 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.

DW is 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 may be included as a composite dead load.

The dead loads applied after the deck has cured, DC_2 and DW, are sometimes termed superimposed dead loads. These superimposed dead loads may be distributed equally to all girders as traditionally specified by the AASHTO LRFD (5^{th} Edition, 2010). In some cases, such as wider bridges, staged construction or heavier utilities, the bridge designer should conduct a more representative analysis to determine a more accurate distribution of superimposed dead loads. For a typical bridge, the barriers could more realistically be assumed to be supported by the exterior girders alone.

EL is the accumulated lock-in, or residual, force effects resulting from the construction process, including the secondary forces from post-tensioning (which are not gravitational dead loads).

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

2.1.3 Earth Pressures (see Article 3.11)

EH is the horizontal earth pressure.

ES is the earth pressure from a permanent earth surcharge (e.g., an embankment).

DD are the loads developed along the vertical sides of a deep-foundation element tending to drag it downward 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, DD. Calculate this additional load as a skin-friction effect. 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 Table 3.4.1-2 as appropriate.

2.2 Transient Loads

2.2.1 General

Transient loads are loads that are not always present in or on the bridge or change in magnitude during the life of the bridge. The AASHTO LRFD (5^{th} Edition, 2010) recognizes 19 transient loads. Static water pressure, stream pressure, buoyancy and wave action are designated as water load, WA. Creep, settlement, shrinkage and temperature (CR, SE, SH, TU and TG) are elevated in importance to "loads," being superimposed deformations which, if restrained, will result in force effects. For example, restraint strains due to uniform-temperature increase induce compression forces. The AASHTO LRFD (5th Edition, 2010) has considerably increased the vehicular braking force (BR) to reflect the improvements in the mechanical capability of modern trucks in comparison with the traditional values of the *AASHTO Standard Specifications for Highway Bridges* (referred to herein as the Standard Specifications) (2).

2.2.2 Live Loads (see Article 3.6)

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

For short and medium span bridges, which are predominant, 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 a more 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 (to those familiar with the Standard Specifications, the HS20-44 truck) or a 50-kip design tandem, similar to the Alternate Loading, both of the Standard Specifications and the AASHTO LRFD (5th Edition, 2010); 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 legal loads. Thus, a heavier vehicle is 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 the 1st Edition of the AASHTO LRFD Specifications. Ultimately, however, it was deemed objectionable to specify a super-legal truck in the AASHTO LRFD (5th Edition, 2010). 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. 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 (5th Edition, 2010) 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 (5^{th} Edition, 2010) 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 (5^{th} Edition, 2010) to calculate these effects. Two design trucks, with a fixed rear axle spacing of 14 ft and a clear distance not less than 50 ft between them, superimposed upon the lane load, all within a single design lane and adjusted by a factor of 0.90 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.

In positioning the two trucks to calculate negative moment or the interior reaction over an internal support of a continuous girder, 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). 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). 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. See Figure 1 below. The truck in the first span of the two-span continuous bridge (in the figure) 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 (5^{th} Edition, 2010) 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 consists of a single design truck per bridge with a load factor of 0.75 (i.e., the factored force effects are equivalent to those of an HS-15 truck). This single-factored design truck produces a considerable reduction in the stress range in comparison with the stress ranges of the Standard Specifications. However, fatigue designs using the Specifications are virtually identical to those of the Standard Specifications. This equivalence is accomplished through an increase in the frequency from values on the order of two million cycles in the Standard Specifications, which represented "design" cycles, to frequencies on the order of tens and hundreds of millions of cycles, which represent actual cycles in the Specifications. The increase in number of cycles compensates for the reduction in stress range, yet both cases fall on the resistance curve producing a similar fatigue design.

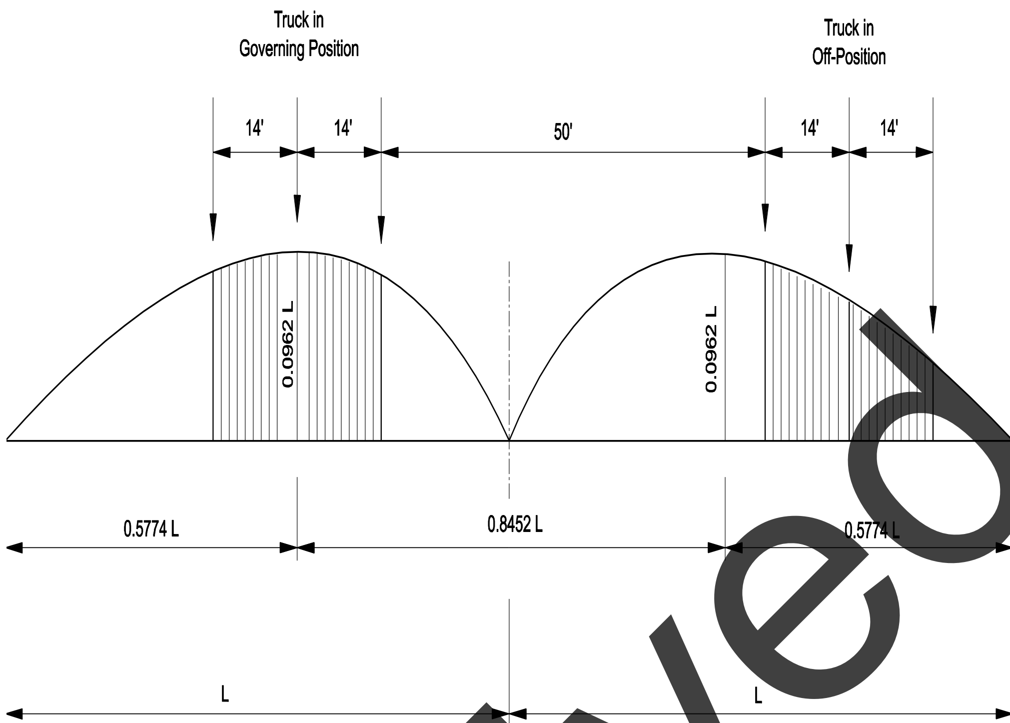

Figure 1 Influence line for a two-span continuous bridge

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 allowance (IM) of 0.33 is applicable only to the design trucks and the design tandems, but not to the uniformly distributed lane load.

LS is the horizontal earth pressure from vehicular traffic on the ground surface above an abutment or wall.

Where reinforced-concrete approach slabs are provided at bridge ends, live-load surcharge need not be considered on the abutment; however, the bridge designer shall consider the reactions on the abutment due to the axle loads on the approach slabs. The abutments must be able to resist the reactions due to axle loads on an approach slab.

Where approach slabs are not provided, the abutments must be able to resist the lateral pressure due to the live-load surcharge just as a retaining wall.

BR is the horizontal vehicular braking force.

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

2.2.3 Water Loads (see Article 3.7)

WA is the pressure due to differential water levels, stream flow or buoyancy.

2.2.4 Wind Loads (see Article 3.8)

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

WL is the horizontal pressure on vehicles due to wind.

2.2.5 Extreme-Event Loads

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

2.2.6 Superimposed Deformations (see Article 3.12)

TU is the uniform temperature change due to seasonal variation.

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

SH is the differential shrinkage between different concretes or concrete and non-shrinking materials, such as metals and wood.

CR is the creep of concrete or wood.

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

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.

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 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 (5th Edition, 2010) 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 specified, as their magnitude and placement can be very contractor and project specific. Nonetheless, the AASHTO LRFD (5th Edition, 2010) suggests load factors for the various load components during construction as shown below in Table 1. The commentary to the AASHTO LRFD (5th Edition, 2010) states that these load factors "should not relieve the contractor of responsibility for safety and damage control during construction."

Jacking forces during bearing replacement also fall into this category of loads discussed but not included formally in the load combinations. The AASHTO LRFD (5th Edition, 2010) 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 of 1.75 should also be included in the factored design force.

Table 1 Load factors during construction

LOAD COMPONENT	LOAD FACTOR
Dead Load	1.25
Construction Loads Equipment Dynamic Effects	1.5
Wind	1.25
All Other Loads	1.0

3.0 LOAD COMBINATIONS

3.1 Reliability-based Design

The AASHTO LRFD (5th Edition, 2010) are based upon the theory of structural reliability in that the strength load combinations are developed to achieve uniform reliability of all structural components of all types of materials. When the load factors and the resistance factors of the AASHTO LRFD (5th Edition, 2010) 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 load and resistance factor design and load factor design.

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 (5th Edition, 2010) 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 (5th Edition, 2010) 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 is determined predominantly by applying inelastic response principles. The AASHTO LRFD (5th Edition, 2010) has adopted the hybrid nature of strength design on the assumption that the inelastic component of structural performance will always remain relatively small because of non-critical redistribution of force effects. This non-criticality is assured 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. The location of η_i on the load side of the AASHTO LRFD (5^{th} Edition, 2010) Equation may appear counterintuitive because it appears to be more related to resistance than to load. η_i is on the load side for a logistical 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 (5^{th} Edition, 2010) 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 (5^{th} Edition, 2010) and not necessarily in the accuracy of their magnitude. The AASHTO LRFD (5^{th} Edition, 2010) factors reflect the desire to promote redundant and ductile bridges.

In practice, η_i values of 1.00 are used for all limit states, because bridges designed in accordance with the AASHTO LRFD (5^{th} Edition, 2010) 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.

The load modifier accounting for importance of Article 1.3.5, η_I, 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 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, shown in the first column of load factors. This variable γ_P reflects that the Strength and Extreme-Event 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 (5^{th} Edition, 2010) 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

The load factors for the superimposed deformations for the Strength limit states also have two specified values -- a load factor of 0.5 for the calculation of stress, 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). The lower value of 0.5 for the elastic calculation of stress reflects the inelastic response of the structure due to restrained deformations. For example, one-half of the temperature rise would be used to elastically calculate the stresses in a constrained structure. 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.

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.

3.2.4.2 Strength I Load Combination

This load combination represents random traffic and the heaviest truck to cross the bridge in its 75-year design life. During this live-load event, a significant wind is not considered probable.

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.

3.2.4.4 Strength III Load Combination.

This load combination represents the most severe wind during the bridge's 75-year design life. During this severe wind event, no 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 (5th Edition, 2010) has arbitrarily increased the load factor for DC to 1.5. This load combination need not be considered for any component except a superstructure component, and never where the unfactored dead-load force effect is less than seven times the unfactored live-load force effect. This load combination typically governs only for longer spans, approximately greater than approximately 200 feet in length. Thus, this load combination will be necessary only in relatively rare cases.

3.2.4.6 Strength V Load Combination

This load combination represents the simultaneous occurrence of a "normal" live-load event and a "55-mph" wind event with load factors of 1.35 and 0.4, 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 indicated above 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, akin to the "overload check" of the Standard Specifications is applied for controlling cracking in reinforced concrete components and compressive stresses in prestressed concrete components. This load combination is also used to calculate deflections and settlements of superstructure and substructure components.

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.

3.2.5.4 Service III Load Combination

This load combination is applied for controlling tensile stresses in prestressed concrete superstructure components under vehicular traffic loads.

3.2.5.5 Service IV Load Combination

This load combination is applied for controlling tensile stresses in prestressed concrete substructure components under wind loads. For components not traditionally governed by wind effects, this load combination should not govern.

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 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.

3.2.6.2 Extreme Event II Load Combination

This load combination is applied to various types of collisions (e.g., vessel, vehicular, or ice) applied individually. 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 state load combination, although strictly applicable to all types of superstructures, only governs the design of the steel elements, components, and connections of a limited number of steel superstructures.

4.0 REFERENCES

1. AASHTO, (2010). AASHTO LRFD Bridge Design Specifications; 5th Edition, AASHTO, Washington D.C.

2. AASHTO, (2002). Standard Specifications for Highway Bridges, 17th Edition, AASHTO, Washington D.C.