Chapter 12

LOADS AND LOAD FACTORS

NDOT STRUCTURES MANUAL

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Chapter 12
LOADS AND LOAD FACTORS

Sections 1 and 3 of the *LRFD Bridge Design Specifications* discuss various aspects of loads. Unless noted otherwise in Chapter 12 of the *NDOT Structures Manual*, the *LRFD Specifications* applies to loads and load factors in Nevada. Chapter 12 also presents additional information on NDOT practices.

12.1 GENERAL

12.1.1 Load Definitions

Reference: LRFD Article 3.3.2

12.1.1.1 Permanent Loads

Reference: LRFD Article 3.5

Permanent loads are loads that are always present in or on the bridge and do not change in magnitude during the life of the bridge. Specific permanent loads include:

1. Gravitational Dead Loads.

   - DC – dead load of all of the components of the superstructure and substructure, both structural and non-structural.
   - DW – dead load of additional non-integral wearing surfaces, future overlays and any utilities crossing the bridge.
   - EL – 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 – vertical earth pressure from the dead load of earth fill.

2. Earth Pressures.

   Reference: LRFD Article 3.11

   - EH – horizontal earth pressure.
   - ES – earth pressure from a permanent earth surcharge (e.g., an embankment).
   - DD – 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.

12.1.1.2 Transient Loads

Transient loads are loads that are not always present in or on the bridge or change in magnitude during the life of the bridge. Specific transient loads include:

1. Live Loads.

   Reference: LRFD Article 3.6

   - LL – static vertical gravity loads due to vehicular traffic on the roadway.
   - PL – vertical gravity loads due to pedestrian traffic on sidewalks.
   - IM – dynamic load allowance to amplify the force effects of statically applied vehicles to represent moving vehicles, traditionally called impact.
   - LS – horizontal earth pressure from vehicular traffic on the ground surface above an abutment or wall.
   - BR – horizontal vehicular braking force.
   - CE – horizontal centrifugal force from vehicles on a curved roadway.

2. Water Loads.

   Reference: LRFD Article 3.7

   - WA – pressure due to differential water levels, stream flow or buoyancy.

3. Wind Loads.

   Reference: LRFD Article 3.8

   - WS – horizontal and vertical pressure on superstructure or substructure due to wind.
   - WL – horizontal pressure on vehicles due to wind.

4. Extreme Events.

   - EQ – loads due to earthquake ground motions.

     Reference: LRFD Article 3.10

   - CT – horizontal impact loads on abutments or piers due to vehicles or trains.

     Reference: LRFD Article 3.6.5

   - CV – horizontal impact loads due to aberrant ships or barges.

     Reference: LRFD Article 3.14

- IC – horizontal static and dynamic forces due to ice action.

  Reference: LRFD Article 3.9

5. Superimposed Deformations.

   Reference: LRFD Article 3.12

   - TU – uniform temperature change due to seasonal variation.
   - TG – temperature gradient due to exposure of the bridge to solar radiation.
   - SH – differential shrinkage between different concretes or concrete and non-shrinking materials, such as metals and wood.
   - CR – creep of concrete or wood.
   - SE – the effects of settlement of substructure units on the superstructure.

6. Friction Forces.

   Reference: LRFD Article 3.13

   - FR – frictional forces on sliding surfaces from structure movements.

12.1.2 Limit States

Reference: LRFD Article 1.3.2

The *LRFD Specifications* groups the traditional design criteria together within groups termed “limit states.” The *LRFD Specifications* assigns load combinations to the various limit states.

12.1.2.1 Basic LRFD Equation

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

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n \qquad \text{(LRFD Eq. 1.3.2.1-1)}$$

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 LRFD Equations 1.3.2.1-2 and 1.3.2.1-3

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

For the Strength limit states, the *LRFD Specifications* is basically a hybrid design code in that 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 Equation is determined predominantly by applying inelastic response principles. The *LRFD Specifications* 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, which is NDOT's general policy for the design of bridges.

12.1.2.2 Load Modifier

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 LRFD 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 LRFD Equation 1.3.2.1-2; when η_i modifies a minimum load factor, it is the reciprocal of the product as indicated in LRFD Equation 1.3.2.1-3. These factors are somewhat arbitrary; their significance is in their presence in the *LRFD Specifications* and not necessarily in the accuracy of their magnitude. The LRFD factors reflect the desire to promote redundant and ductile bridges.

NDOT uses η_i values of 1.00 for all limit states, because bridges designed in accordance with the *NDOT Structures Manual* will demonstrate traditional levels of redundancy and ductility. Rather than penalize less redundant or less ductile bridges, such bridges are not encouraged. NDOT may on a case-by-case basis designate a bridge to be of special operational importance and specify an appropriate value of η_i.

The load modifier accounting for importance of LRFD Article 1.3.5, η_I, should not be confused with the importance categories for seismic design of LRFD Articles 3.10.3 and 4.7.4.3. The importance load modifier is used in the basic LRFD Equation, but the importance categories are used to determine the minimum seismic analysis requirements.

12.1.3 <u>Load Factors and Combinations</u>

Reference: LRFD Article 3.4.1

LRFD Table 3.4.1-1 provides the load factors for all of the load combinations of the *LRFD Specifications*.

12.1.3.1 Strength Load Combinations

The load factors for the Strength load combinations are calibrated based upon structural reliability theory and represent the uncertainty of their associated loads. The significance of the Strength load combinations can be simplified as follows:

1. <u>Strength I Load Combination</u>. 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.

2. Strength II Load Combination. In the *LRFD Specifications*, this load combination represents an owner-specified permit load model. This live-load event has less uncertainty than random traffic and, thus, a lower live-load load factor. This load combination is used for design in conjunction with the permit live load design vehicle (P loads) discussed in Section 12.3.2.7

3. 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 is assumed to cross the bridge.

4. 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 *LRFD Specifications* 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 200 ft in length. Thus, this load combination will only be necessary in relatively rare cases.

5. 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.

For components not traditionally governed by wind force effects, the Strength III and Strength V load combinations should not govern. Generally, the Strength I and Strength II load combinations will govern for a typical multi-girder highway bridge.

12.1.3.2 Service Load Combinations

Unlike the Strength load combinations, the Service load combinations are material dependent. The following applies:

1. Service I Load Combination. This load combination 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.

2. 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. Service III Load Combination. This load combination is applied for controlling tensile stresses in prestressed concrete superstructure components under vehicular traffic loads. The Service III load combination need not apply to the design permit live load design vehicle.

4. 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.

12.1.3.3 Extreme-Event 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). The following applies:

1. Extreme-Event I Load Combination. This load combination is applied to earthquakes. Because of the high seismicity in specific regions of Nevada, this load combination often governs design. Earthquakes in conjunction with scour (which is considered a change in foundation condition, not a load) can result in a very costly design solution if severe scour is anticipated. In this case, NDOT practice is to combine one-half of the total design scour (sum of contraction, local and long-term scour) with the full seismic loading.

2. Extreme-Event II Load Combination. This load combination is applied to various types of collisions (vessel, vehicular or ice) applied individually.

12.1.3.4 Fatigue-and-Fracture Load Combination

The Fatigue-and-Fracture load combination, although strictly applicable to all types of superstructures, only affects the steel elements, components and connections of a limited number of steel superstructures. Chapter 15 discusses fatigue and fracture for steel.

12.1.3.5 Application of Multiple-Valued Load Factors

12.1.3.5.1 *Maximum and Minimum Permanent-Load Load Factors*

In LRFD 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 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. LRFD 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 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 AASHTO *Standard Specifications* treated uplift as a separate load combination. With the introduction of maximum and minimum load factors, the *LRFD Specifications* 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 LRFD 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, q_{ult}, 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, Q_{τ}, again making the limit state more critical. The application of these maximum and minimum load factors is required for foundation and substructure design; see Chapters 17 and 18.

12.1.3.5.2 *Load Factors for Superimposed Deformations*

The load factors for the superimposed deformations (TU, CR, SH) 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.

12.2 PERMANENT LOADS

12.2.1 General

Reference: LRFD Article 3.5

The *LRFD Specifications* specifies seven components of permanent loads, which are either direct gravity loads or caused by gravity loads. The primary forces from prestressing are considered to be part of the resistance of a component and has been omitted from the list of permanent loads in Section 3 of the *LRFD Specifications*. 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 LRFD Equation. The permanent load EL includes secondary forces from pre-tensioning or post-tensioning. As specified in LRFD Table 3.4.1-2, use a constant load factor of 1.0 for both maximum and minimum load factors for EL.

As discussed in Section 12.1.3.5.1, the permanent force effects in superstructure design are factored by the maximum permanent-load load factors almost exclusively, with the most common exception being 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 LRFD Table 3.4.1-2 as appropriate.

12.2.2 Deck Slab Dead Load

12.2.2.1 General

Loads applied to the composite cross section (i.e., the girder with the slab over it) include the weight of any raised median, rail, sidewalk or barrier placed after the deck concrete has hardened. Include a uniform load of 38 psf to account for a wearing surface over the entire deck area between the face of rails or sidewalks.

12.2.2.2 Composite Girders

Reference: LRFD Articles 6.10.1.1.1 and 9.7.4

Bridge deck slab dead load (DL) for design consists of composite and non-composite components. Loads applied to the non-composite cross section (i.e., the girder alone) include the weight of the plastic concrete, forms and other construction loads typically required to place the deck. Calculate the non-composite DL using the full-slab volume including haunches.

Where steel stay-in-place formwork is used, the designer shall account for the steel form weight and any additional concrete in the flutes of the formwork. The combined weight of the form and concrete in the flutes shall not exceed 15 psf.

12.2.2.3 Cast-in-Place Concrete Box Girders

The designer shall account for the weight of lost deck forms by including an additional load of 12 psf.

12.2.3 Distribution of Dead Load to Girders

Reference: LRFD Article 4.6.2.2.1

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 girders and cantilevered over the exterior girders.

Superimposed dead loads (e.g., curbs, barriers, sidewalks, parapets, railings, future wearing surfaces) placed after the deck slab has cured may be distributed equally to all girders as traditionally specified by AASHTO except for girder bridges with more than six girders. For wider bridges with more than six girders, assume that the superimposed dead loads of sidewalks, parapets or railings are carried by the three girders immediately under and adjacent to the load. In some cases, such as staged construction and heavier utilities, the bridge designer should conduct a more refined analysis, as discussed in Section13.2, to determine a more accurate distribution of superimposed dead loads.

For cast-in-place concrete box girders, assume equal distribution across the full bridge deck width.

12.2.4 Downdrag on Deep Foundations

Reference: LRFD Article 3.11

Deep foundations (i.e., driven piles and drilled shafts) through unconsolidated soil layers may be subject to downdrag, DD. Downdrag is a load 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. 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. Section 17.3.3.1 discusses mitigation methods.

12.2.5 Differential Settlement

Differential settlement between adjacent substructure units or transversely across a single substructure unit induces stresses in continuous structures and deflections in simple structures. Although most bridges can easily resist these stresses and deflections, the potential effects of differential settlement should be considered for all structures. The effects of differential settlement in the longitudinal direction need not be considered if its magnitude is ½ in or less. The effects of differential settlement in the transverse direction should be considered on a case-by-case basis.

12.3 TRANSIENT LOADS

12.3.1 General

The *LRFD Specifications* recognizes 19 transient loads. Static water pressure, stream pressure, buoyancy and wave action are integrated 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, restrained strains due to uniform-temperature increase induces compression forces. The *LRFD Specifications* 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*.

12.3.2 Vehicular Live Load (LL)

12.3.2.1 General

Reference: LRFD Articles 3.6.1.1, 3.6.1.2 and 3.6.1.3

For short and medium span bridges, which predominate in Nevada, vehicular live load is the most significant component of load. Dead loads become more significant for long-span bridges. Long-span bridges are defined as those governed by the Strength IV load combination where the dead load is seven times or more greater than the live load.

12.3.2.2 The Nature of the Notional 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 true representation of the force effects due to actual trucks.

The components of the HL-93 notional load are:

- a vehicle, either the familiar HS-20 truck, now called the design truck, or a 50-kip design tandem, similar to the Alternate Loading, both of the *Standard Specifications*; and

- a 0.64 k/ft uniformly distributed lane load, similar to the lane load of the *Standard Specifications*, but without any of the previous associated concentrated loads.

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.

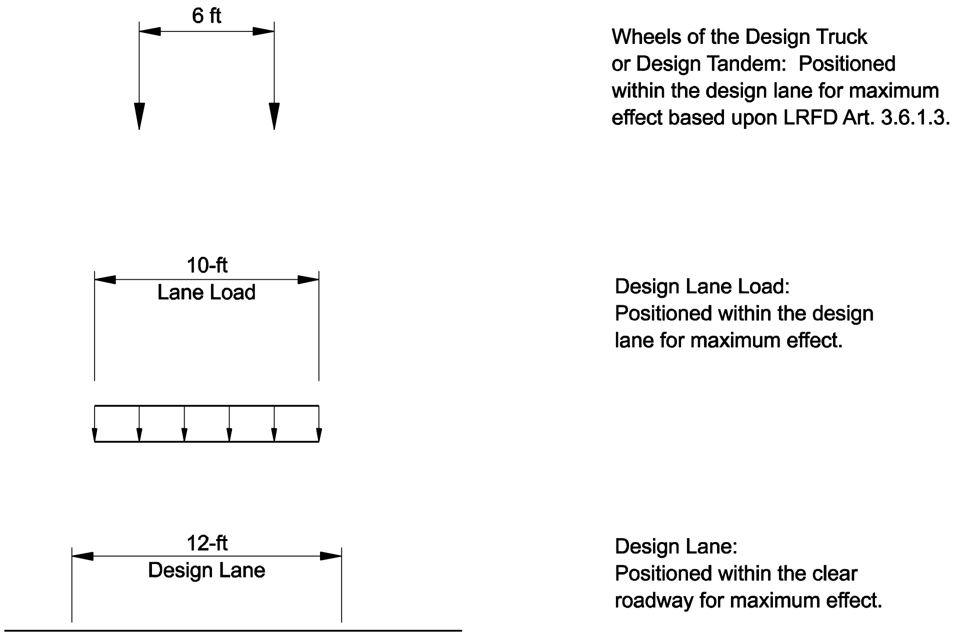
The force effects of the traditional HS-20 truck alone are less than that of the legal loads. Thus, a heavier vehicle is appropriate for design. As specified for the HL-93 live-load model, the concept of superimposing the design vehicle force effects and the design lane force effects was developed to yield moments and shears representative of real trucks on the highways. The moments and shears produced by the HL-93 load model are essentially equivalent to those of a 57-ton truck.

12.3.2.3 Multiple Presence Factors

The multiple presence factor of 1.0 for two loaded lanes, as given in LRFD Table 3.6.1.1.2-1, is the result of the *LRFD Specifications'* 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 not be applied to fatigue loads.

12.3.2.4 Application of Vehicles and Lanes

The *LRFD Specifications* 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 LRFD Article 3.6.1.3 and illustrated schematically in Figure 12.3-A.



PLACEMENT OF THE DESIGN LOADS WITHIN THE DESIGN LANES

Figure 12.3-A

12.3.2.5 Special Load Applications

12.3.2.5.1 *Two Design Trucks in a Single Lane for Negative Moment and Interior Reactions*

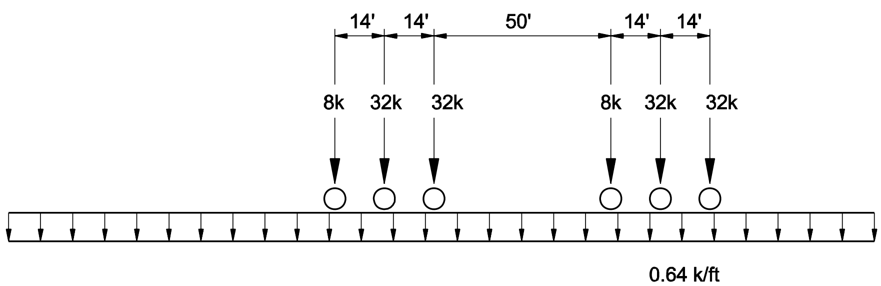
Reference: LRFD Article 3.6.1.3.1

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 of two heavy vehicles closely following one another, interspersed with other lighter traffic. Thus, a special load case has been specified in the *LRFD Specifications* to calculate these force 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 closely spaced heavy trucks. This sequence of highway loading is specified for negative moment and reactions at interior piers due to the shape of the influence lines for such force effects. This sequence is not extended to other structures or portions of structures because it is not expected to govern for other influence-line shapes. This loading is illustrated in Figure 12.3-B.

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 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). These effects are illustrated in Figure 12.3-C.

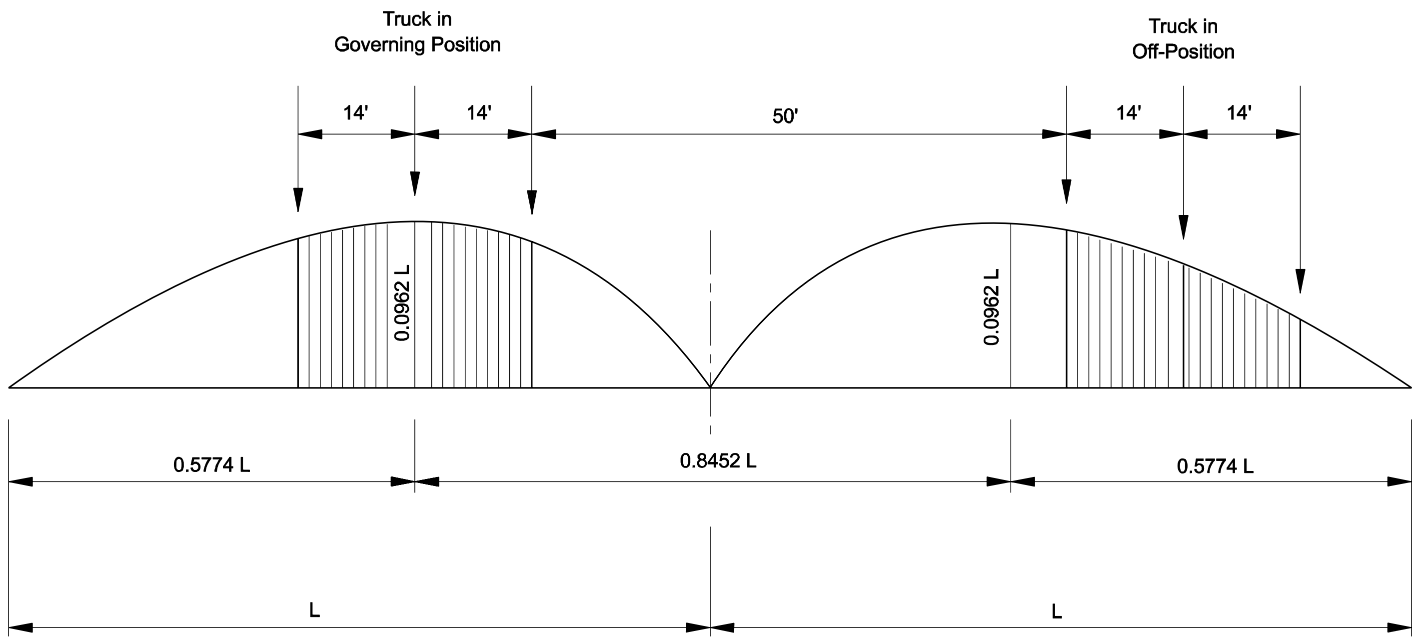
12.3.2.5.2 *Application of Horizontal Superstructure Forces to the Substructure*

The transfer of horizontal superstructure forces to the substructure depends on the type of superstructure to substructure connection. Centrifugal force (CE), braking force (BR) and wind on live load (WL) are all assumed to act horizontally at a distance of 6 ft above the roadway. Connections can be fixed, pinned or free for both moment and shear.



SPECIAL LOADING FOR NEGATIVE MOMENT AND INTERIOR REACTIONS OF CONTINUOUS SPANS

Figure 12.3-B



APPLICATION OF DESIGN VEHICULAR LIVE LOAD – LRFD ARTICLE 3.6.1.3

Figure 12.3-C

If the horizontal superstructure force is being applied to the substructure through a pinned connection, there is no moment transfer. The designer should apply the superstructure force to the substructure at the connection.

For a fixed or moment connection, apply the superstructure horizontal force with an additional moment to the substructure. The additional moment is equal to the horizontal force times the distance between the force's line of action and the point of application.

12.3.2.6 Wheel Load for Deck Design

Reference: LRFD Article 3.6.1.3.3

Bridge decks shall be designed to carry axles consisting of two 20-kip wheels with dynamic allowance, alone or in combination with the lane load as appropriate. This axle load is consistent with the HS-25 truck.

12.3.2.7 Permit Loads for Design (P Load)

NDOT has adopted one of the Caltrans "Standard Permit Design Vehicles" for the design of structures to provide a minimum permit-load capacity on all highway structures to account for vehicles that exceed the legal limits and that operate on highways and structures under special transportation permits. This load is commonly called the "P" load. Typically, all State-owned bridges are designed for the Strength II, Service I and Service II load combinations with the P load in all lanes. The application of the P load to non-State owned bridges is determined on a case-by-case basis.

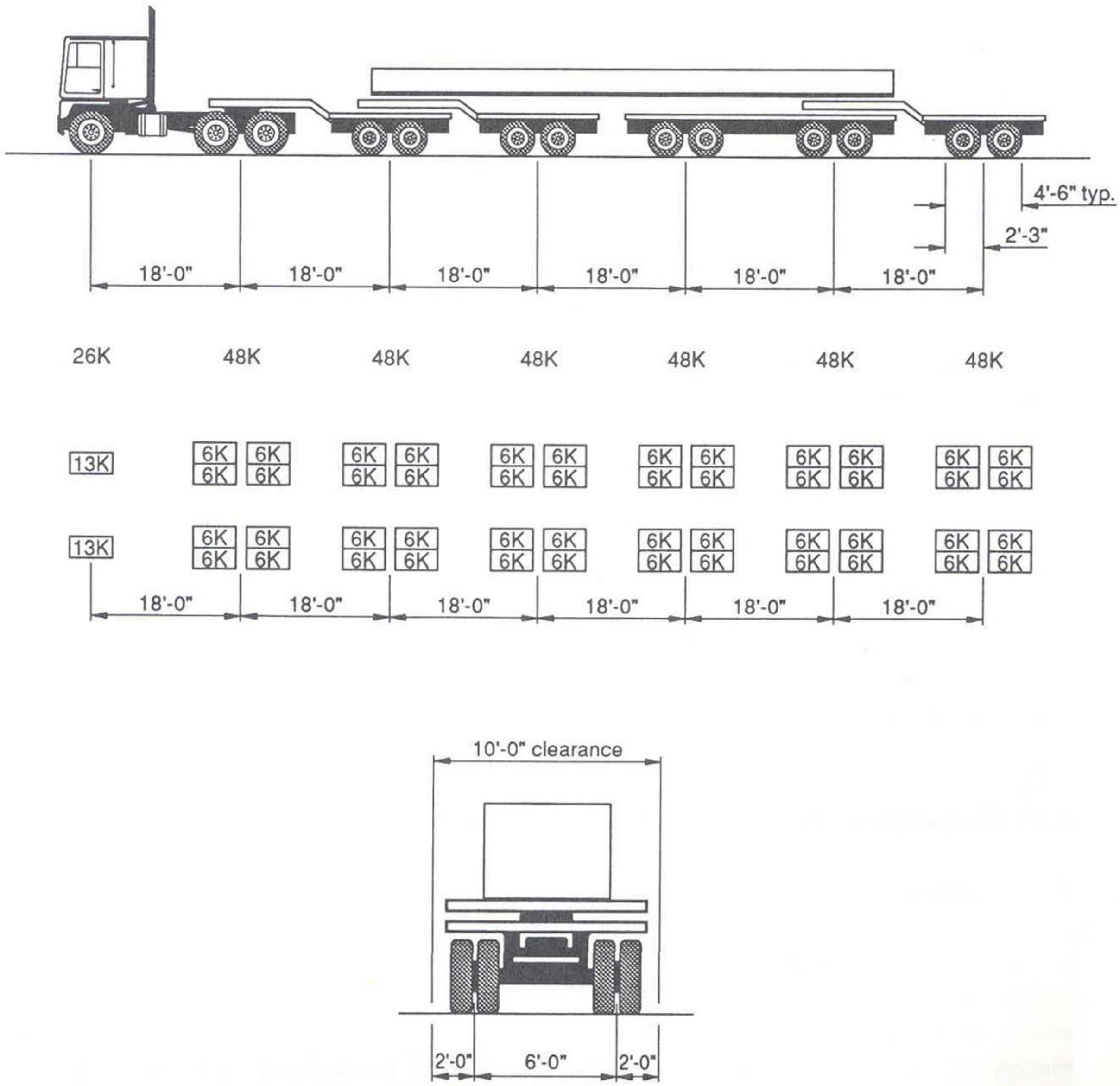
The P load, specifically the Caltrans P-13, is illustrated in Figure 12.3-D.

12.3.2.8 Fatigue Loads

Reference: LRFD Articles 3.6.1.4.1, 3.6.1.4.2

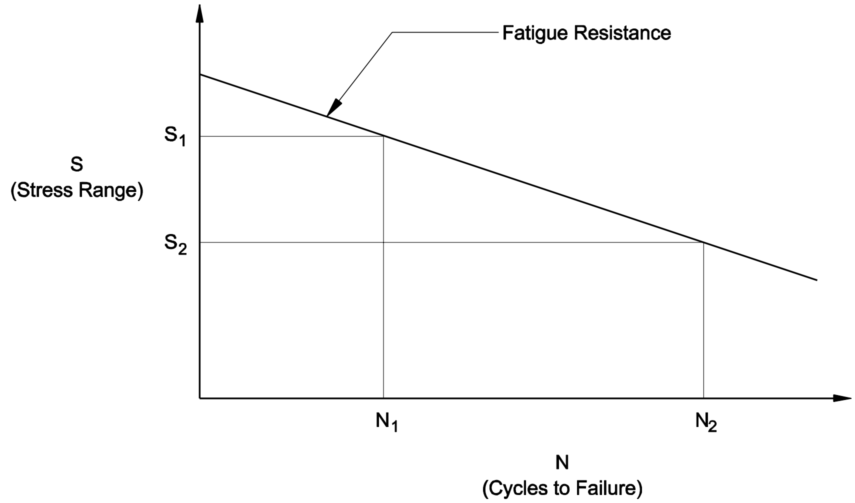
The *LRFD Specifications* defines the fatigue load for a particular bridge component by specifying both a magnitude and a frequency. The magnitude of the fatigue load consists of a single design truck per bridge with a load factor of 0.75 (i.e., the factored force effects are equivalent to those of an HS-15 truck). This single-factored design truck produces a considerable reduction in the stress range in comparison with the stress ranges of the AASHTO *Standard Specifications*. However, fatigue designs using the *LRFD 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 *LRFD Specifications*.

This change to more realistic stress ranges and cycles, illustrated in the S-N curve (a log-log plot of stress range versus cycle to failure) of Figure 12.3-E, increases the designer's understanding of the extremely long fatigue lives of steel bridges. In Figure 12.3-E, S_1 represents the controlling stress range for multiple lanes of strength-magnitude loading typically in accordance with the *Standard Specifications*, with N_1 being its corresponding number of design cycles. S_2 represents the controlling stress range for a single fatigue truck in accordance



PERMIT DESIGN LIVE LOADS
(For P-13 Vehicle)

Figure 12.3-D



COMPARISON OF THE FATIGUE LOADS OF THE *LRFD SPECIFICATIONS* AND *STANDARD SPECIFICATIONS*

Figure 12.3-E

with the *LRFD Specifications*, with N_2 being its corresponding number of actual cycles. The increase in the number of cycles compensates for the reduction in stress range, yet both cases fall on the resistance curve producing a similar fatigue design.

The bridge designer shall also apply P loads, also with a load factor of 0.75, to the fatigue design for structural steel. In lieu of better information, the average daily truck traffic in a single lane, $ADTT_{SL}$, for the P load shall be taken as 10 trucks per day.

12.3.2.9 Distribution of Live Load to Piers

Reference: LRFD Article 3.6.1.3.1

To promote uniformity of distribution of live load to piers and other substructure components, the following procedure is suggested unless a more exact distribution of loads is used:

1. <u>Live-Load Distribution Factor</u>. The live-load distribution factor for each girder shall be determined assuming that the deck is acting as a simple girder between interior girders and as a cantilever spanning from the first interior girder over the exterior girder.

2. <u>Live Load on Design Lanes</u>. Design lanes shall be placed on the bridge to produce the maximum force effect for the component under investigation. Separate loadings of the HL-93 live load or the P load shall be placed within an individual design lane to likewise produce the maximum effect. The bridge designer shall consider one, two, three or more design lanes in conjunction with the multiple presence factors of LRFD Table 3.6.1.1.2-1, as can be accommodated on the roadway width.

3. Reaction on Piers. For piers with drop caps, live loads are transmitted to the pier through the girder bearings, and the cap shall be designed using the shears determined from the girder line analysis. For integral caps, the designer may distribute the live load to the cap using a wheel line method, a girder and axle method, or a combination of the two. The wheel line method and the girder and axle method are described in Example 12.3-1. For both drop caps and integral caps, the designer shall analyze multiple lane positions to maximize load effects (e.g., side-by-side lanes to maximize negative cap bending at an interior pier support, lanes placed in every other cap span to maximize positive bending).

* * * * * * * * * *

Example 12.3-1 — Live Load Placement on Integral Bent Caps

Given:

- Two-span bridge, 145-ft and 160-ft spans, zero skew, box girder depth of 6′-6″
- Girder spacing = 9′-4″
- Column Spacing = 18′8″ (with zero skew, pier is normal to bridge centerline)
- From the superstructure analysis, the reaction at the center pier for a single HL-93 lane with both spans loaded was determined to be 200k
- HL-93 loading is depicted in this example. Treat permit loads in a similar fashion. Apply superstructure dead load to the integral cap at girder lines

a. Wheel Line Method (Simplified Approach)

Determine wheel line loads from HL-93 lane reaction:

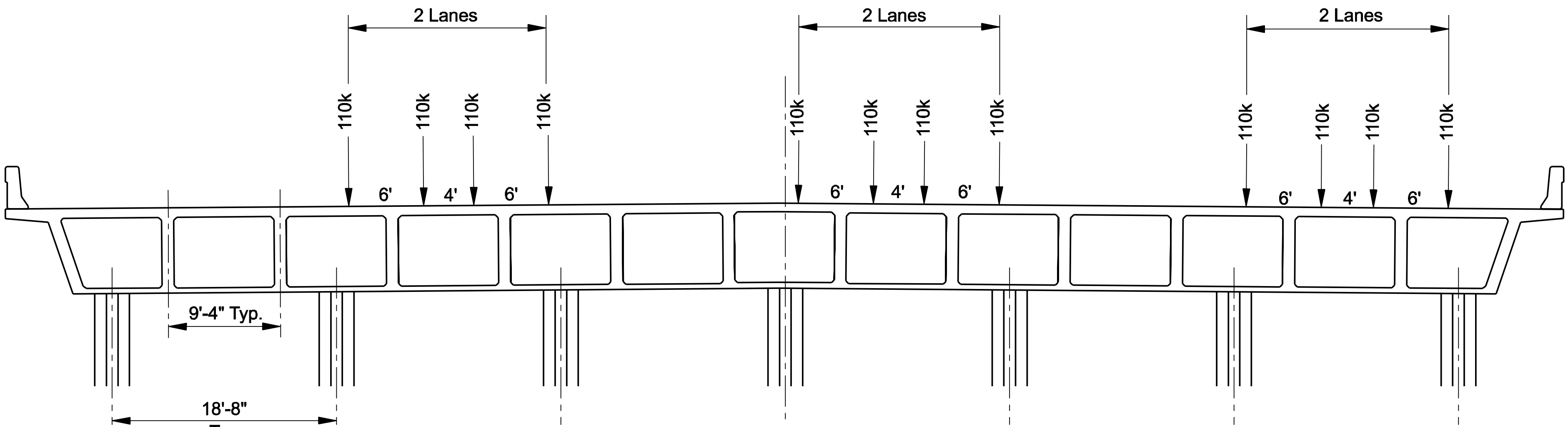
$$W_{HL\text{-}93} = \tfrac{1}{2}\,(\text{lane reaction})$$
$$= \tfrac{1}{2}\,(220\text{k}) = 110\text{ k}$$

Wheel lines are applied 6 ft apart in a lane and 4 ft apart between lanes. As positioned in Figure 12.3-F(a), wheel lines are located to maximize positive bending in the cap beam. Analyze additional wheel line patterns to maximize load effects along the length of the cap beam (i.e., to develop moment and shear envelopes). A “train” of wheel lines running across the cap as a moving load is an easy approach to generating the envelopes.

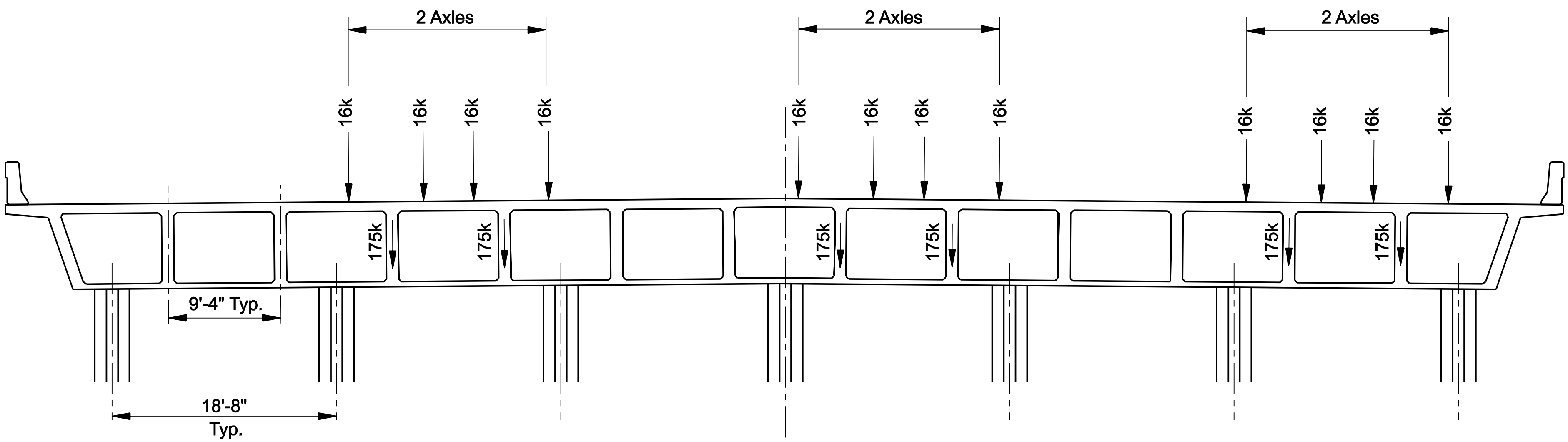
b. Girder and Axle Method (Refined Approach)

This refined approach recognizes that the majority of the lane load is transferred to the cap through the girder lines while a portion of the lane load could be positioned anywhere on the cap as an axle passes over. To represent this condition, the lane loading is divided between that which reaches the cap through the girders and that which is caused by the heaviest axle from the design vehicle applied directly to the cap. Determine the loads to girders assuming that the deck is simply supported between girder lines. From the full lane load, subtract the heaviest vehicle axle for direct application to the pier cap.

Figure 12.3-G represents girder and axle load placement to produce maximum positive bending in the cap. From statics:



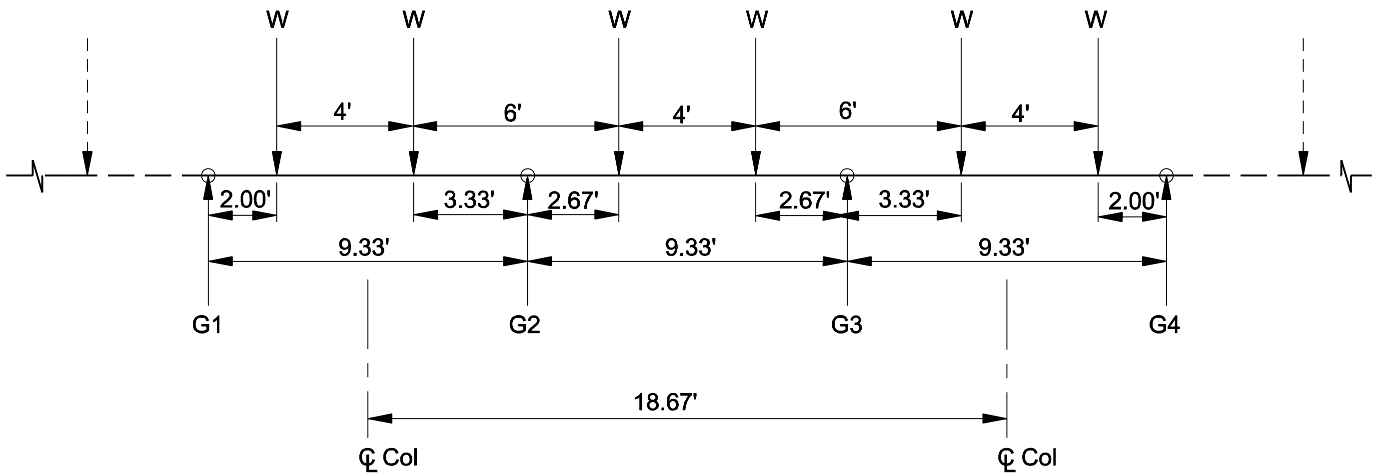
(a) WHEEL LINE METHOD



(b) GIRDER AND AXLE METHOD

LIVE LOAD APPLICATION
(Integral Cap)

Figure 12.3-F



PARTIAL CAP ELEVATION

Figure 12.3-G

G2 = G3	=	W + 6W/9.33 + 2W/9.33
	=	1.86W
	=	0.93 lanes

HL-93 axle	=	32k
G2 = G3	=	0.93 (220k – 32k)
	=	175k

See Figure 12.3-F(b) for placement of loads across the integral cap.

* * * * * * * * * *

12.3.2.10 Sidewalk Loading

Reference: LRFD Article 3.6.1.6

Where sidewalks are present on the bridge, the bridge designer shall design for the dead load and pedestrian live load on the sidewalk; however, the full width of the bridge, including sidewalks, shall also be designed for the traffic live load assuming that traffic can mount the sidewalk.

Pedestrian and traffic loads will not be applied together. Sidewalks separated from traffic lanes by barrier rail shall also be designed for vehicular loads due to the potential for future widening.

12.3.2.11 Vehicular Collision Force (CT)

Reference: LRFD Article 3.6.5

Bridge abutments and piers over highways or railroads within a distance of:

- 30 ft to the edge of the roadway, or
- 50 ft to the centerline of the railroad track

shall be protected as specified in LRFD Article 3.6.5.1. If this is deemed to be impractical and with the approval of the Chief Structures Engineer, the abutment or pier shall be designed for a collision force of 400 kips acting in a horizontal plane in any direction at a distance of 4 ft above ground, as specified in LRFD Article 3.6.5.2.

12.3.3 Friction Forces (FR)

Reference: LRFD Article 3.13

LRFD Article 3.13 discusses the determination of horizontal friction forces from an expansion bearing sliding on its bearing plate on the supporting substructure component.

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.

12.3.4 Thermal Loads

Reference: LRFD Article 3.12.2

The bridge designer shall use Procedure A of LRFD Article 3.12.2.1 to determine the appropriate design thermal range. For Nevada-specific ranges of temperatures and procedures, see Section 19.1.

12.3.5 Earthquake Effects

Reference: LRFD Article 3.10

The seismic provisions of the *LRFD Specifications* shall be applied to bridge design in Nevada. The seismicity of Nevada varies greatly across the State. Nevada includes all four seismic zones specified in the *LRFD Specifications*. Earthquake force effects shall be determined in accordance with LRFD Article 3.10; however, the minimum seismic coefficients shown in Figure 12.3-H shall be applied unless otherwise approved by the Chief Structures Engineer.

Other Chapters in the *NDOT Structures Manual* present NDOT's seismic detailing practices. For example, Chapter 15 presents NDOT's seismic detailing practices for steel superstructures.

County	**Peak Ground Acceleration (PGA) Coefficient**	**Short-Period Spectral Acceleration Coefficient (S_s)**	**Long-Period Spectral Acceleration Coefficient (S_l)**
Carson City, Douglas, Esmerelda, Washoe	0.50	1.25	0.50
Lyon, Mineral, Storey	0.40	1.00	0.40
Churchill, Nye	0.35	0.80	0.30
Eureka, Lander, Lincoln, Pershing	0.25	0.60	0.20
Clark, Elko, Humboldt, White Pine	0.15	0.40	0.15

MINIMUM SEISMIC COEFFICIENTS BY COUNTY

Figure 12.3-H

12.3.6 Live-Load Surcharge (LS)

Reference: LRFD Article 3.11.6.2

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. Because approach slabs are required at all bridges in Nevada, live-load surcharge is not used for abutments.

Retaining walls that retain soil supporting a roadway must be able to resist the lateral pressure due to the live-load surcharge. See Section 23.1 for retaining walls.