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Highway Bridge Loads and Load Distribution

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6.1 Introduction

This chapter deals with highway bridge loads and load distribution as specified in the AASHTO Load and Resistance Factor Design (LRFD) Specifications [1]. Stream flow, ice loads, vessel collision loads, loads for barrier design, loads for anchored and mechanically stabilized walls, seismic forces, and loads due to soil–structure interaction will be addressed in subsequent chapters. Load combinations are discussed in Chapter 5.

When proceeding from one component to another in bridge design, the controlling load and the controlling factored load combination will change. For example, permit vehicles, factored and combined for one load group, may control girder design for bending in one location. The standard design vehicular live load, factored and combined for a different load group, may control girder design for shear in another location. Still other loads, such as those due to seismic events, may control column and footing design.

Note that in this chapter, superstructure refers to the deck, beams or truss elements, and any other appurtenances above the bridge soffit. Substructure refers to those components that support loads from the superstructure and transfer load to the ground, such as bent caps, columns, pier walls, footings, piles, pile extensions, and caissons. Longitudinal refers to the axis parallel to the direction of traffic. Transverse refers to the axis perpendicular to the longitudinal axis.

6.2 Permanent Loads

The LRFD Specification refers to the weights of the following as "permanent loads":

- The structure
- Formwork which becomes part of the structure
- Utility ducts or casings and contents
- Signs
- Concrete barriers
- Wearing surface and/or potential deck overlay(s)
- Other elements deemed permanent loads by the design engineer and owner
- Earth pressure, earth surcharge, and downdrag

The permanent load is distributed to the girders by assigning to each all loads from superstructure elements within half the distance to the adjacent girder. This includes the dead load of the girder itself and the soffit, in the case of box girder structures. The dead loads due to concrete barrier, sidewalks and curbs, and sound walls, however, may be equally distributed to all girders.

6.3 Vehicular Live Loads

The design vehicular live load was replaced in 1993 because of heavier truck configurations on the road today, and because a statistically representative, notional load was needed to achieve a "consistent level of safety." The notional load that was found to best represent "exclusion vehicles," i.e., trucks with loading configurations greater than allowed but routinely granted permits by agency bridge rating personnel, was adopted by AASHTO and named "Highway Load '93" or HL93. The mean and standard deviation of truck traffic was determined and used in the calibration of the load factors for HL93. It is notional in that it does not represent any specific vehicle [2].

The distribution of loads per the LRFD Specification is more complex than in the Standard Specifications for Highway Bridge Design [3]. This change is warranted because of the complexity in bridges today, increased knowledge of load paths, and technology available to be more rational in performing design calculations. The end result will be more appropriately designed structures.

6.3.1 Design Vehicular Live Load

The AASHTO "design vehicular live load," HL93, is a combination of a "design truck" or "design tandem" and a "design lane." The design truck is the former Highway Semitrailer 20-ton design truck (HS20-44) adopted by AASHO (now AASHTO) in 1944 and used in the previous Standard Specification. Similarly, the design lane is the HS20 lane loading from the AASHTO Standard Specifications. A shorter, but heavier, design tandem is new to AASHTO and is combined with the design lane if a worse condition is created than with the design truck. Superstructures with very short spans, especially those less than 12 m in length, are often controlled by the tandem combination.

The AASHTO design truck is shown in Figure 6.1. The variable axle spacing between the 145 kN loads is adjusted to create a critical condition for the design of each location in the structure. In the transverse direction, the design truck is 3 m wide and may be placed anywhere in the standard 3.6-m-wide lane. The wheel load, however, may not be positioned any closer than 0.6 m from the lane line, or 0.3 m from the face of curb, barrier, or railing.

The AASHTO design tandem consists of two 110-kN axles spaced at 1.2 m on center. The AASHTO design lane loading is equal to 9.3 N/mm and emulates a caravan of trucks. Similar to the truck loading, the lane load is spread over a 3-m-wide area in the standard 3.6-m lane. The lane loading is not interrupted except when creating an extreme force effect such as in "patch" loading of alternate spans. Only the axles contributing to the extreme being sought are loaded.

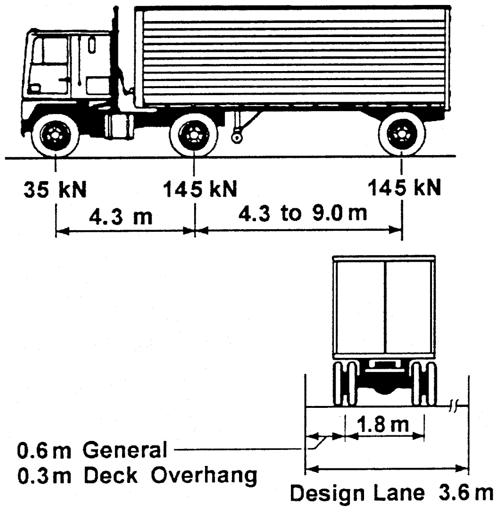

FIGURE 6.1 AASHTO-LRFD design truck. (AASHTO LRFD Bridge Design Specifications 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.)

When checking an extreme reaction at an interior pier or negative moment between points of contraflexure in the superstructure, two design trucks with a 4.3-m spacing between the 145-kN axles are to be placed on the bridge with a minimum of 15 m between the rear axle of the first truck and the lead axle of the second truck. Only 90% of the truck and lane load is used. This procedure differs from the Standard Specification which used shear and moment riders.

6.3.2 Permit Vehicles

Most U.S. states have developed their own "Permit Design Vehicle" to account for vehicles routinely granted permission to travel a given route, despite force effects greater than those due the design truck, i.e., the old HS20 loading. California uses anywhere from a 5- to 13-axle design vehicle as shown in Figure 6.2 [4]. Some states use an HS25 design truck, the configuration being identical to the HS20 but axle loads 25% greater.

The permit vehicular live load is combined with other loads in the Strength Limit State II as discussed in Chapter 5. Early editions of the AASHTO Specifications expect the design permit vehicle to be preceded and proceeded by a lane load. Furthermore, adjacent lanes may be loaded with the new HL93 load, unless restricted by escort vehicles.

6.3.3 Fatigue Loads

For fatigue loading, the LRFD Specification uses the design truck alone with a constant axle spacing of 9 m. The load is placed to produce extreme force effects. In lieu of more exact information, the frequency of the fatigue load for a single lane may be determined by multiplying the average daily truck traffic by p, where p is 1.00 in the case of one lane available to trucks, 0.85 in the case of two lanes available to trucks, and 0.80 in the case of three or more lanes available to trucks. If the average daily truck traffic is not known, 20% of the average daily traffic may be used on rural interstate bridges, 15% for other rural and urban interstate bridges, and 10% for bridges in urban areas.

6.3.4 Load Distribution for Superstructure Design

Figure 6.3 summarizes load distribution for design of longitudinal superstructure elements. Load distribution tables and the "lever rule" are approximate methods and intended for most designs.

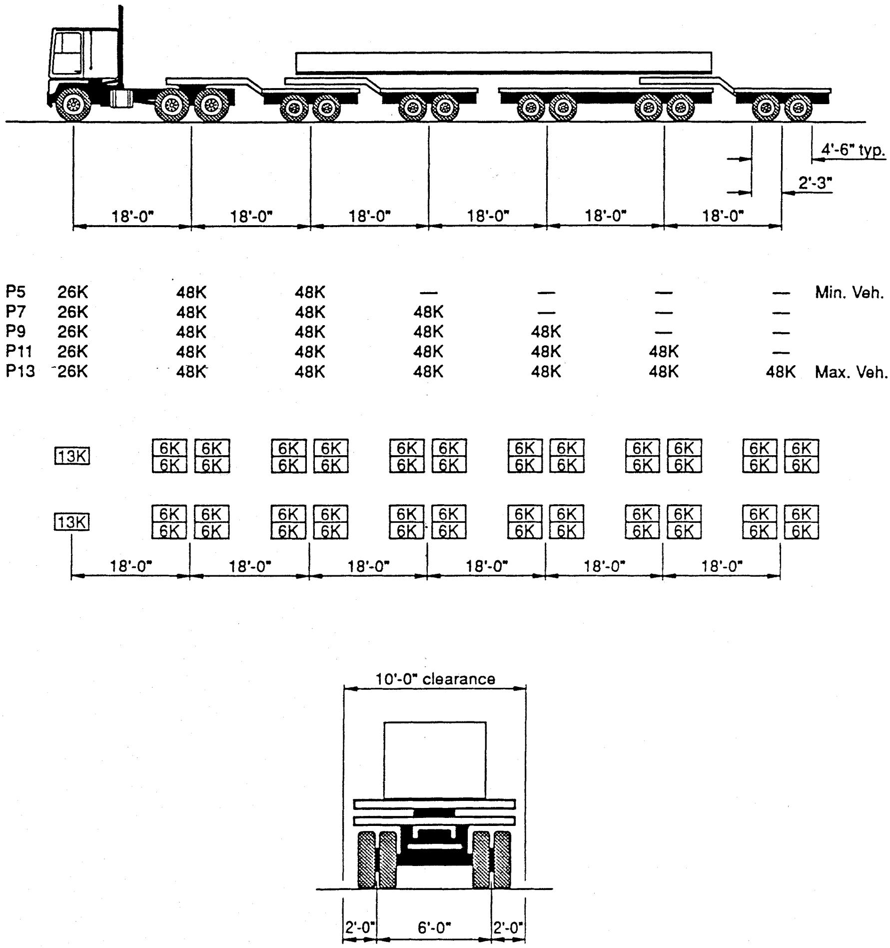

FIGURE 6.2 Caltrans permit truck. (AASHTO LRFD Bridge Design Specifications 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.)

The lever rule considers the slab between two girders to be simply supported. The reaction is determined by summing the reactions from the slabs on either side of the beam under consideration. "Refined analysis" refers to a three-dimensional consideration of the loads and is to be used on more complex structures. In other words, classical force and displacement, finite difference, finite element, folded plate, finite strip, grillage analogy, series/harmonic, or yield line methods are required to obtain load effects for superstructure design.

Note that, by definition of the vehicular design live load, no more than one truck can be in one lane simultaneously, except as previously described to generate maximum reactions or negative moments. After forces have been determined from the longitudinal load distribution and the longitudinal members have been designed, the designer may commence load distribution in the transverse direction for deck and substructure design.

6.3.4.1 Decks

Decks may be designed for vehicular live loads using empirical methods or by distributing loads on to "effective strip widths" and analyzing the strips as continuous or simply supported beams.

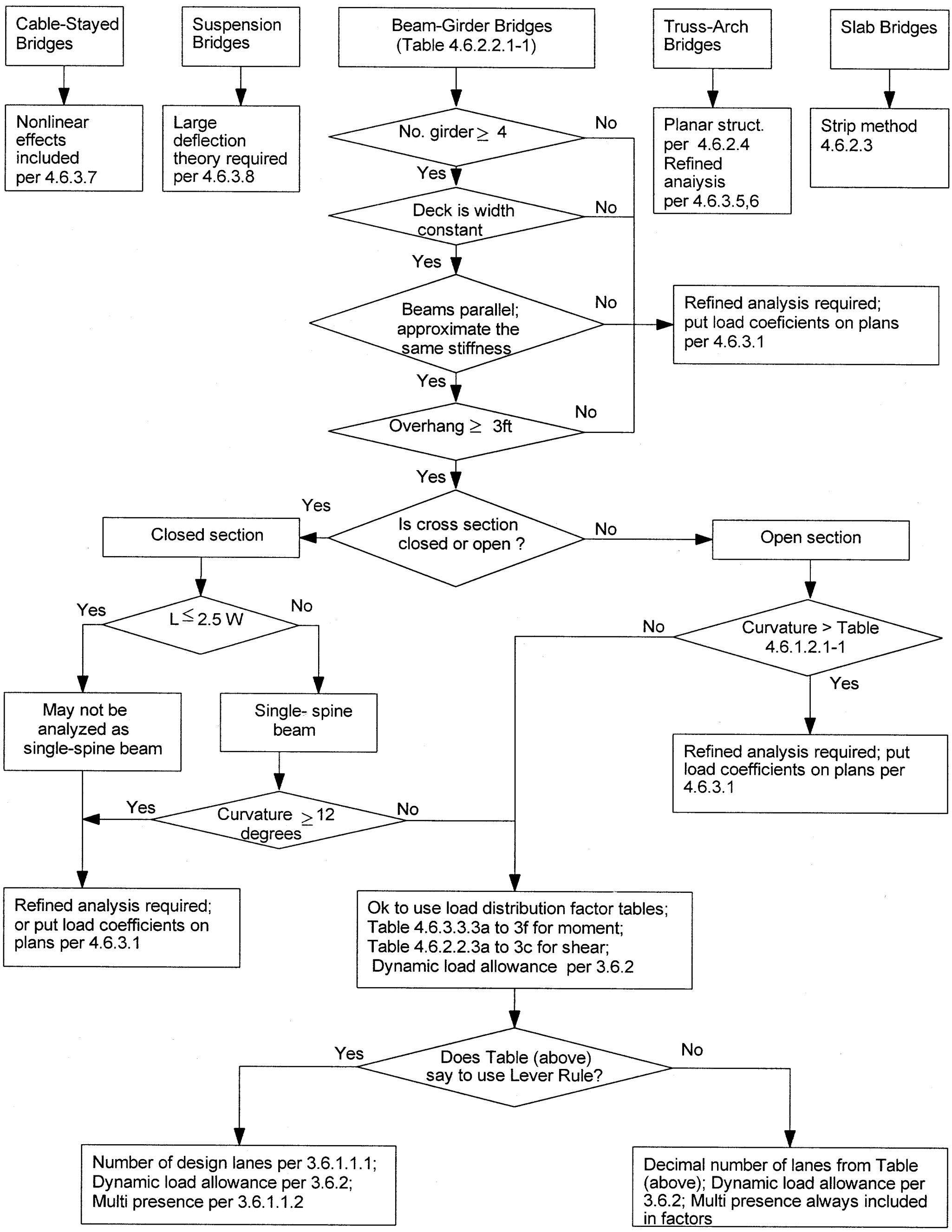

FIGURE 6.3 Live-load distribution for superstructure design.

Empirical methods rely on transfer of forces by arching of the concrete and shifting of the neutral axis. Loading is discussed in Chapter 24, Bridge Decks and Approach Slabs.

6.3.4.2 Beam–Slab Bridges

Approximate methods for load distribution on beam–slab bridges are appropriate for the types of cross sections shown in Table 4.6.2.2.1-1 of the AASHTO LRFD Specification. Load distribution

TABLE 6.1 Reduction of Load Distribution Factors for Moment in Longitudinal Beams on Skewed Supports

Type of Superstructure	Applicable Cross Section from Table 4.6.2.2.1-1	Any Number of Design Lanes Loaded	Range of Applicability
Concrete deck, filled grid, or partially filled grid on steel or concrete beams, concrete T-beams, or double T-sections	a, e, k i, j, if sufficiently connected to act as a unit	$1 - c_1(\tan\theta)^{1.5}$ $c_1 = 0.25\left(\frac{K_g}{Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.25}$ if $\theta < 30°$, then $c_1 = 0$ if $\theta > 60°$, use $\theta = 60°$	$30° \le \theta \le 60°$ $1100 \le S \le 4900$ $6000 \le L \le 73{,}000$ $N_b \ge 4$
Concrete deck on concrete spread box beams, concrete box beams, and double T-sections used in multibeam decks	b, c, f, g	$1.05 - 0.25 \tan\theta \le 1.0$ if $\theta > 60°$, use $\theta = 60°$	$0 \le \theta \le 60°$

Source: AASHTO LRFD Bridge Design Specifications, 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.

factors, generated from expressions found in AASHTO LRFD Tables 4.6.2.2.2a–f and 4.6.2.2.3a–c, result in a decimal number of lanes and are used for girder design. Three-dimensional effects are accounted for. These expressions are a function of beam area, beam width, beam depth, overhang width, polar moment of inertia, St. Venant's torsional constant, stiffness, beam span, number of beams, number of cells, beam spacing, depth of deck, and deck width. Verification was done using detailed bridge deck analysis, simpler grillage analyses, and a data set of approximately 200 bridges of varying type, geometry, and span length. Limitations on girder spacing, span length, and span depth reflect the limitations of this data set.

The load distribution factors for moment and shear at the obtuse corner are multiplied by skew factors as shown in Tables 6.1 and 6.2, respectively.

6.3.4.3 Slab-Type Bridges

Cast-in-place concrete slabs or voided slabs, stressed wood decks, and glued/spiked wood panels with spreader beams are designed for an equivalent width of longitudinal strip per lane for both shear and moment. That width, E (mm), is determined from the formula:

$$E = 250 + 0.42\sqrt{L_1 W_1} \tag{6.1}$$

when one lane is loaded, and

$$E = 2100 + 0.12\sqrt{L_1 W_1} \le W/N_L \tag{6.2}$$

when more than one lane is loaded. L_1 is the lesser of the actual span or 18,000 mm, W_1 is the lesser of the edge-to-edge width of bridge and 18,000 mm in the case of single-lane loading, and 18,000 mm in the case of multilane loading, and N_L is the numbr of design lanes.

6.3.5 Load Distribution for Substructure Design

Bridge substructure includes bent caps, columns, pier walls, pile caps, spread footings, caissons, and piles. These components are designed by placing one or more design vehicular live loads on the traveled way as previously described for maximum reaction and negative bending moment, not exceeding the maximum number of vehicular lanes permitted on the bridge. This maximum may be determined by dividing the width of the traveled way by the standard lane width (3.6 m), and "rounding down," i.e., disregarding any fractional lanes. Note that (1) the traveled way need not be

TABLE 6.2 Correction Factors for Load Distribution Factors for Support Shear of the Obtuse Corner

Type of Superstructue	Applicable Cross Section from Table 4.6.2.2.1-1	Correction Factor	Range of Applicability
Concrete deck, filled grid, or partially filled grid on steel or concrete beams, concrete T-beams or double T-sections	a, e, k i, j, if sufficiently connected to act as a unit	— $1.0+2.0\left(\frac{Lt_s^3}{K_g}\right)^{0.3}\tan\theta$	$0° \le \theta \le 60°$ $1100 \le S \le 4900$ $6000 \le L \le 73{,}000$ $N_b \ge 4$
Multicell concrete box beams, box sections	d	$1.0+\left[0.25+\frac{L}{70d}\right]\tan\theta$	$0° \le \theta \le 60°$ $1800 \le S \le 4000$ $6000 \le L \le 73000$ $900 \le d \le 2700$ $N_b \ge 3$
Concrete deck on spread concrete box beams	b, c	$1.0+\frac{\sqrt{Ld}}{6S}\tan\theta$	$0° \le \theta \le 60°$ $1800 \le S \le 3500$ $6000 \le L \le 43{,}000$ $450 \le d \le 1700$ $N_b \ge 3$
Concrete box beams used in multibeam decks	f, g	$1.0+\frac{L\sqrt{\tan\theta}}{90d}$	$0° \le \theta \le 60°$ $6000 \le L \le 37{,}000$ $430 \le d \le 1500$ $900 \le b \le 1500$ $5 \le N_b \le 20$

Source: AASHTO LRFD Bridge Design Specifications, 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.

measured from the edge of deck if curbs or traffic barriers will restrict the traveled way for the life of the structure and (2) the fractional number of lanes determined using the previously mentioned load distribution charts for girder design is not used for substructure design.

Figure 6.4 shows selected load configurations for substructure elements. A critical load configuration may result from not using the maximum number of lanes permissible. For example, Figure 6.4a shows a load configuration that may generate the critical loads for bent cap design and Figure 6.4b shows a load configuration that may generate the critical bending moment for column design. Figure 6.4c shows a load configuration that may generate the critical compressive load for design of the piles. Other load configurations will be needed to complete design of a bridge footing. Note that girder locations are often ignored in determination of substructure design moments and shears: loads are assumed to be transferred directly to the structural support, disregarding load transfer through girders in the case of beam–slab bridges. Adjustments are made to account for the likelihood of fully loaded vehicles occurring side-by-side simultaneously. This "multiple presence factor" is discussed in the next section.

In the case of rigid frame structures, bending moments in the longitudinal direction will also be needed to complete column (or pier wall) as well as foundation designs. Load configurations which generate these three cases must be checked:

1. Maximum/minimum axial load with associated transverse and longitudinal moments;
2. Maximum/minimum transverse moment with associated axial load and longitudinal moment;
3. Maximum/minimum longitudinal moment with associated axial load and transverse moment.

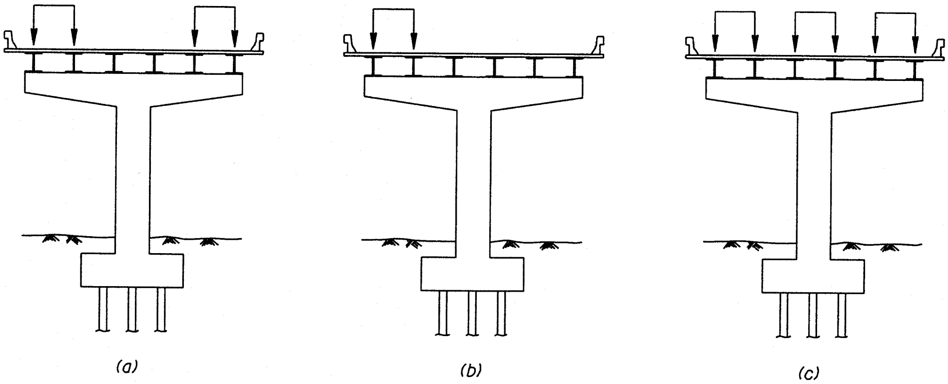

FIGURE 6.4 Various load configurations for substructure design.

TABLE 6.3 Multiple Presence Factors

Number of Loaded Lanes	Multiple Presence Factors m
1	1.20
2	1.00
3	0.85
>3	0.65

Source: AASHTO LRFD Bridge Design Specifications, 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.

If a permit vehicle is also being designed for, then these three cases must also be checked for the load combination associated with Strength Limit State II (discussed in Chapter 5).

6.3.6 Multiple Presence of Live-Load Lanes

Multiple presence factors modify the vehicular live loads for the probability that vehicular live loads occur together in a fully loaded state. The factors are shown in Table 6.3.

These factors should be applied prior to analysis or design only when using the lever rule or doing three-dimensional modeling or working with substructures. Sidewalks greater than 600 mm can be treated as a fully loaded lane. If a two-dimensional girder line analysis is being done and distribution factors are being used for a beam-and-slab type of bridge, multiple presence factors are not used because the load distribution factors already consider three-dimensional effects. For the fatigue limit state, the multiple presence factors are also not used.

6.3.7 Dynamic Load Allowance

Vehicular live loads are assigned a "dynamic load allowance" load factor of 1.75 at deck joints, 1.15 for all other components in the fatigue and fracture limit state, and 1.33 for all other components and limit states. This factor accounts for hammering when riding surface discontinuities exist, and long undulations when settlement or resonant excitation occurs. If a component such as a footing is completely below grade or a component such as a retaining wall is not subject to vertical reactions from the superstructure, this increase is not taken. Wood bridges or any wood component is factored at a lower level, i.e., 1.375 for deck joints, 1.075 for fatigue, and 1.165 typical, because of the energy-absorbing characteristic of wood. Likewise, buried structures such as culverts are subject to the dynamic load allowance but are a function of depth of cover, D_E (mm):

$$IM = 40(1.0 - 4.1 \times 10^{-4} D_E) \geq 0\% \qquad (6.3)$$

6.3.8 Horizontal Loads Due to Vehicular Traffic

Substructure design of vertical elements requires that horizontal effects of vehicular live loads be designed for. Centrifugal forces and braking effects are applied horizontally at a distance 1.80 m above the roadway surface. The centrifugal force is determined by multiplying the design truck or design tandem — alone — by the following factor:

$$C = \frac{4v^2}{3gR} \qquad (6.4)$$

Highway design speed, v, is in m/s; gravitational acceleration, g, is 9.807 m/s^2; and radius of curvature in traffic lane, R, is in m. Likewise, the braking force is determined by multiplying the design truck or design tandem from all lanes likely to be unidirectional in the future, by 0.25. In this case, the lane load is not used because braking effects would be damped out on a fully loaded lane.

6.4 Pedestrian Loads

Live loads also include pedestrians and bicycles. The LRFD Specification calls for a 3.6×10^{-3} MPa load simultaneous with highway loads on sidewalks wider than 0.6 m. "Pedestrian- or bicycle-only" bridges are to be designed for 4.1×10^{-3} MPa. If the pedestrian- or bicycle-only bridge is required to carry maintenance or emergency vehicles, these vehicles are designed for, omitting the dynamic load allowance. Loads due to these vehicles are infrequent and factoring up for dynamic loads is inappropriate.

TABLE 6.4 Base Wind Pressures, P_B, corresponding to V_B = 160 km/h

Structural Component	Windward Load, MPa	Leeward Load, MPa
Trusses, columns, and arches	0.0024	0.0012
Beams	0.0024	NA
Large flat surfaces	0.0019	NA

Source: AASHTO LRFD Bridge Design Specifications, 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.

6.5 Wind Loads

The LRFD Specification provides wind loads as a function of base design wind velocity, V_B equal to 100 mph; and base pressures, P_B, corresponding to wind speed V_B. Values for P_B are listed in Table 6.4. The design wind pressure, P_D, is then calculated as

$$P_D = P_B \left(\frac{V_{DZ}}{V_B} \right)^2 = P_B \frac{V_{DZ}^2}{25{,}600} \qquad (6.5)$$

where V_{DZ} is the design wind velocity at design elevation Z in km/h. V_{DZ} is a function of the friction velocity, V_0 (km/h), multiplied by the ratio of the actual wind velocity to the base wind velocity both at 10 m above grade, and the natural logarithm of the ratio of height to a meteorological constant length for given surface conditions:

TABLE 6.5 Values of V_o and Z_o for Various Upstream Surface Conditions

Condition	Open Country	Suburban	City
V_o (km/h)	13.2	15.2	19.4
Z_o (mm)	70	300	800

Source: AASHTO LRFD Bridge Design Specifications, 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.

TABLE 6.6 Temperature Ranges, °C

Climate	Steel or Aluminum	Concrete	Wood
Moderate	–18 to 50	–12 to 27	–12 to 24
Cold	–35 to 50	–18 to 27	–18 to 24

Source: AASHTO LRFD Bridge Design Specifications, 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.

$$V_{DZ} = 2.5V_0\left(\frac{V_{10}}{V_B}\right)\ln\left(\frac{Z}{Z_0}\right) \tag{6.6}$$

Values for V_0 and Z_0 are shown in Table 6.5.

The resultant design pressure is then applied to the surface area of the superstructure as seen in elevation. Solid-type traffic barriers and sound walls are considered as part of the loading surface. If the product of the resultant design pressure and applicable loading surface depth is less than a lineal load of 4.4 N/mm on the windward chord, or 2.2 N/mm on the leeward chord, minimum loads of 4.4 and 2.2 N/mm, respectively, are designed for.

Wind loads are combined with other loads in Strength Limit States III and V, and Service Limit State I, as defined in Chapter 5. Wind forces due to the additional surface area from trucks is accounted for by applying a 1.46 N/mm load 1800 mm above the bridge deck.

Wind loads for substructure design are of two types: loads applied to the substructure and those applied to the superstructure and transmitted to the substructure. Loads applied to the superstructure are as previously described. A base wind pressure of 1.9×10^{-3} MPa force is applied directly to the substructure, and is resolved into components (perpendicular to the front and end elevations) when the structure is skewed.

In absence of live loads, an upward load of 9.6×10^{-4} MPa is multiplied by the width of the superstructure and applied at the windward quarter point simultaneously with the horizontal wind loads applied perpendicular to the length of the bridge. This uplift load may create a worst condition for substructure design when seismic loads are not of concern.

6.6 Effects Due to Superimposed Deformations

Elements of a structure may change size or position due to settlement, shrinkage, creep, or temperature. Changes in geometry cause additional stresses which are of particular concern at connections. Determining effects from foundation settlement are a matter of structural analysis. Effects due to shrinkage and creep are material dependent and the reader is referred to design chapters elsewhere

TABLE 6.7 Basis of Temperature Gradients

	Concrete		50 mm Asphalt		100 mm Asphalt	
Zone	T_1 (°C)	T_2 (°C)	T_1 (°C)	T_2 (°C)	T_1 (°C)	T_2 (°C)
1	30	7.8	24	7.8	17	5
2	25	6.7	20	6.7	14	5.5
3	23	6	18	6	13	6
4	21	5	16	5	12	6

Source: AASHTO LRFD Bridge Design Specifications, 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.

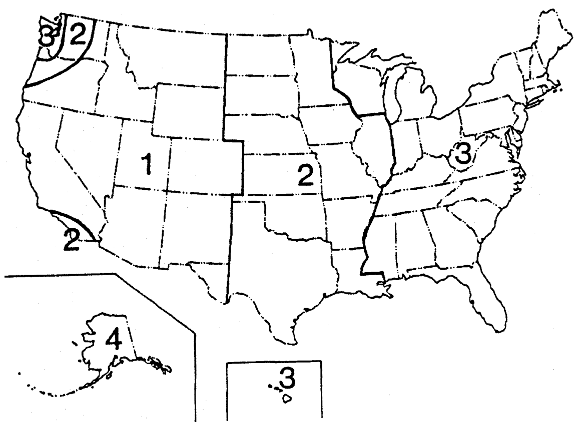

FIGURE 6.5 Solar radiation zones for the United States. (AASHTO LRFD Bridge Design Specifications 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.)

in this book. Temperature effects are dependent on the maximum potential temperature differential from the temperature at time of erection. Upper and lower bounds are shown in Table 6.6, where "moderate" and "cold" climates are defined as having fewer or more than 14 days with an average temperature below 0°C, respectively.

By using appropriate coefficients of thermal expansion, effects from temperature changes are calculated using basic structural analysis. More-refined analysis will consider the time lag between the surface and internal structure temperatures. The LRFD Specification identifies four zones in the United States and provides a linear relationship for the temperature gradient in steel and concrete. See Table 6.7 and Figures 6.5 and 6.6.

6.7 Exceptions to Code-Specified Design Loads

The designer is responsible not only for providing plans that accommodate design loads per the referenced Design Specifications, but also for any loads unique to the structure and bridge site. It is also the designer's responsibility to indicate all loading conditions designed for in the contract documents — preferably the construction plans. History seems to indicate that the next generation of bridge engineers will indeed be given the task of "improving" today's new structure. Therefore, the safety of future generations depends on today's designers doing a good job of documentation.

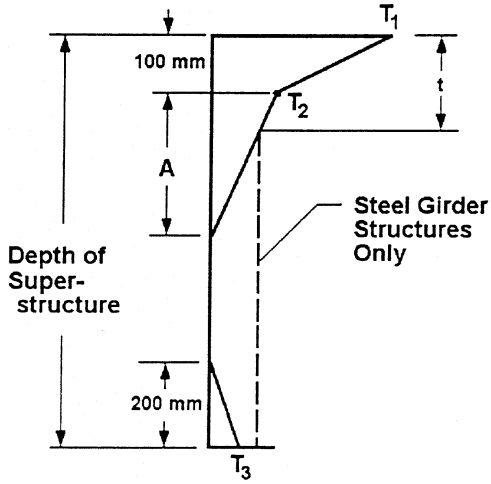

FIGURE 6.6 Positive vertical temperature gradient in concrete and steel superstructures. (AASHTO LRFD Bridge Design Specifications 2nd. ed., American Association of State Highway and Transportation Officials. Washington, D.C., 1998. With permission.)

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