CHAPTER 3
LOADS AND LOAD COMBINATIONS

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3.1 INTRODUCTION

Properly identifying bridge loading is fundamental to the design of each component. Bridge design is iterative in the sense that member sizes are a function of loads and loads are a function of member sizes. It is, therefore, necessary to begin by proportioning members based on prior experience and then adjusting for actual loads and bridge geometry.

This chapter summarizes the loads to be applied to bridges specified in the *AASHTO LRFD Bridge Design Specifications*, 8th Edition (AASHTO, 2017) and the *California Amendments to the AASHTO LRFD Bridge Design Specifications* (CA) (Caltrans, 2019a) (AASHTO-CA BDS-8). It is important to realize that not every load listed will apply to every bridge. For example, a bridge located in Southern California may not need to consider ice loads. A pedestrian overcrossing structure may not have to be designed for vehicular live load.

The Engineer must provide a clear load path. The following illustrates the pathway of truck loading into the various elements of a box girder bridge.

The weight of the truck is distributed to each axle of the truck. One half of the axle load then goes to each wheel or wheel tandem. This load will be carried by the deck slab which spans between girders as shown in Figure 3.1-1.

Once the load has been transferred to the girders, the direction of the load path changes from transverse to longitudinal. The girders carry the load by spanning between bents and abutments (Figure 3.1-2).

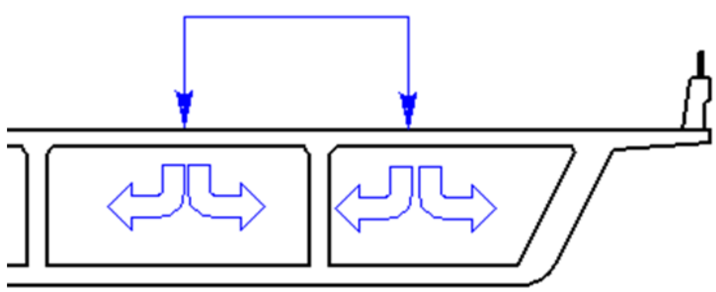


Figure 3.1-1 Truck Load Path from Deck Slab to Girders

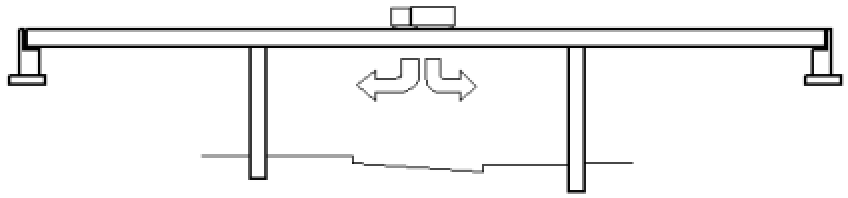


Figure 3.1-2 Truck Load Path from Girders to Bents

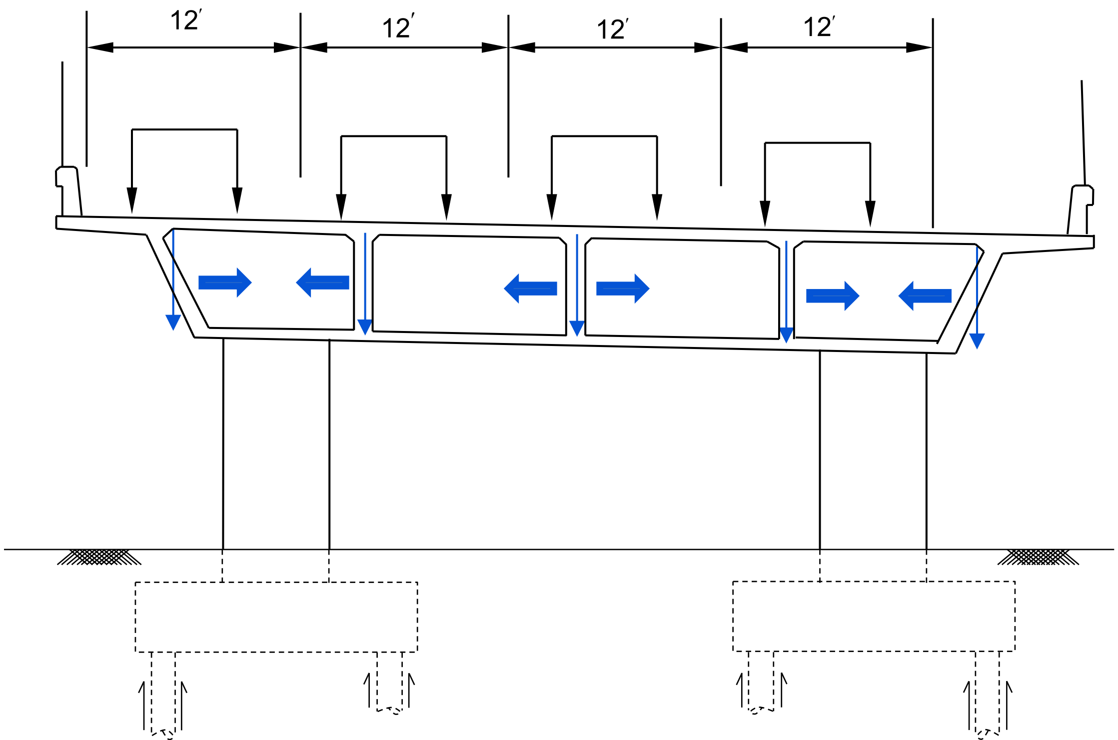


Figure 3.1-3 Truck Load Path from Bent Cap to Columns

When the girder load reaches the bent caps or abutments, it once again changes direction from longitudinal to transverse as shown in Figure 3.1-3. The bent cap beam transfers the load to the columns. Load distribution in the substructure is covered in Section 3.5.3. The columns are primarily axial load carrying members and carry the load to the footing and finally to the piles. The piles transfer the load to the soil where it is carried by the soil matrix.

Load distribution can be described in a more refined manner; however, the basic load path from the truck to the ground is as described above. Each load in AASHTO-CA BDS-

8 Table CA 3.4.1-1 has a unique load path. Some are concentrated loads, others are uniform line loads, while still others, such as wind load, are pressure forces on a surface.

3.2 LOAD DEFINITIONS

3.2.1 Permanent Loads

Permanent loads are defined as loads and forces that are either constant or varying over a long time interval upon completion of construction. They include:

- dead load of structural components and nonstructural attachments (*DC*)
- dead load of wearing surfaces and utilities (*DW*)
- downdrag forces (*DD*)
- horizontal earth pressure loads (*EH*)
- vertical pressure from dead load of earth fill (*EV*)
- earth surcharge load (*ES*)
- force effects due to creep (*CR*)
- force effects due to shrinkage (*SH*)
- secondary forces from post-tensioning for strength limit states or total prestress forces for service limit states (*PS*)
- miscellaneous locked-in force effects resulting from the construction process (*EL*).

3.2.2 Transient Loads

Transient loads are defined as loads and forces that can vary over a short time interval relative to the lifetime of the structure. A transient load is any load that will not remain on the bridge indefinitely. These loads are categorized and listed below:

Live loads such as vehicular live loads and their secondary effects and pedestrian live loads:

- vehicular live load (*LL*)
- vehicular dynamic load allowance (*IM*)
- vehicular braking force (*BR*)
- vehicular centrifugal force (*CE*)
- live load surcharge (*LS*)
- pedestrian live loads (*PL*)

Force effects due to superimposed deformations:

- uniform temperature (*TU*)
- temperature gradient (*TG*)
- settlement (*SE*)

Fluid forces:

- water loads and stream pressure (*WA*)
- wind loads on structure (WS)
- wind on live load (*WL*)

Friction Forces (*FR*)

Extreme event forces:

- ice loads (*IC*)
- vehicular collision forces (*CT*)
- vessel collision forces (*CV*)
- blast loading (*BL*)
- earthquake loads (*EQ*)

3.3 PERMANENT LOAD APPLICATION WITH EXAMPLES

The following structure, shown in Figures 3.3-1 to 3.3-3, is used as an example throughout this chapter, unless otherwise indicated, for use in determining individual loads.

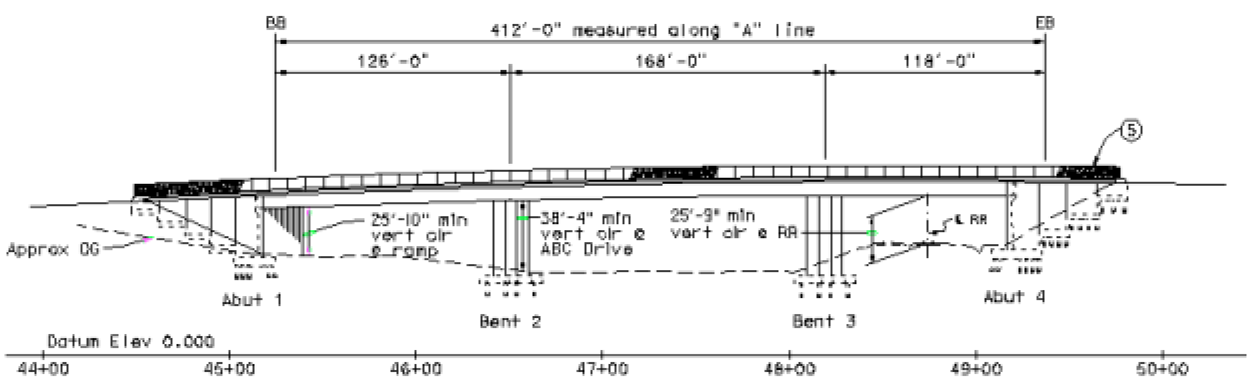


Figure 3.3-1 Elevation View of Example Bridge

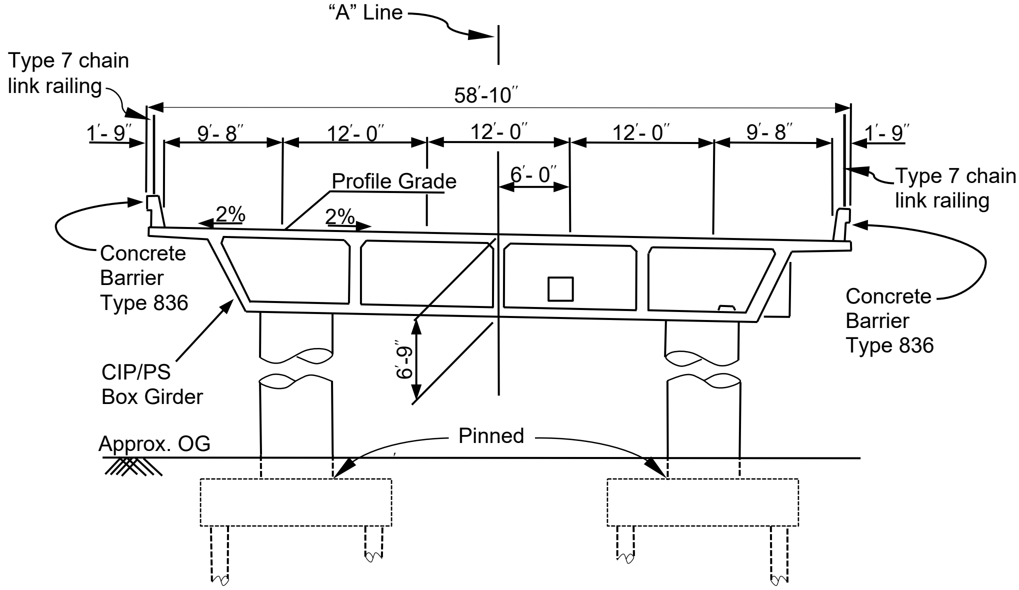


Figure 3.3-2 Typical Section View of Example Bridge

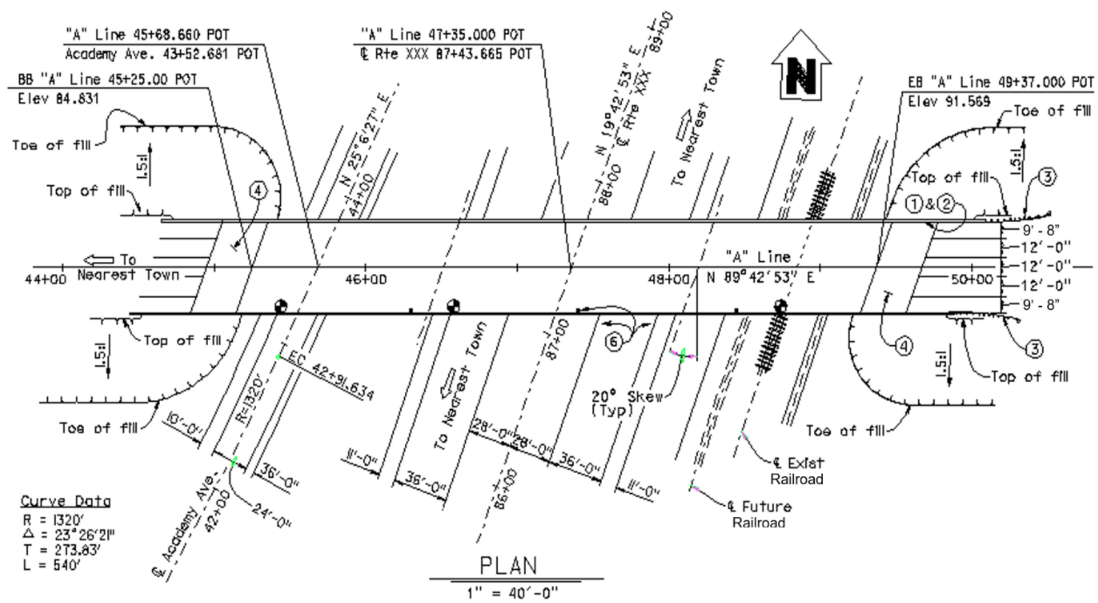


Figure 3.3-3 Plan View of Example Bridge

3.3.1 Dead Load of Components, *DC*

The dead load of the structure is a gravity load and is based on structural member geometry and material unit weight. It is generally calculated by modeling the structural section properties in a computer program such as CTBridge. Additional loads such as intermediate diaphragms, hinge diaphragms, and barriers must be applied separately.

When using the CTBridge program, the Engineer should be aware of possibly "double counting" *DC* loads. For example, when the weight of the bent cap is included in the longitudinal frame analysis, this weight must not be included again for the transverse analysis of the bent.

Normal weight reinforced concrete is assigned a density of 150 pcf which includes the weight of bar reinforcing steel (Article 3.5.1). Adjustments need not be made for the presence of prestressing tendons, soffit access openings, vents, and other small openings for utilities.

For the example bridge, the weight of a Type 836 barrier and Type 7 chain link railing is modeled as a line load in a longitudinal frame analysis as follows:

Type 836 barrier:

$A = 3.61\text{ ft}^2$

$w_c = 0.15$ kcf (AASHTO C5.4.2.4)

$w_{barrier} = Aw_c = 3.61\,(0.15) = 0.54$ kip/ft

Type 7 chain link railing:

$w_{chain} = 16$ lb/ft (this weight is essentially negligible)

Total weight of two barriers $w = (0.54 + 0.02)(2) = 1.12$ kip/ft

Note that for precast concrete and steel girders, CA Article 3.5.1 requires that *DC* of cast-in-place concrete decks shall be increased by 10 percent to account for the weight of stay-in-place metal forms.

3.3.2 Dead Load of Wearing Surfaces and Utilities, *DW*

Future added wearing surfaces are generally polyester concrete or "deck on deck" reinforced concrete. New bridges are designed for a load of 35 psf as required in CA Article 3.5.1. Future wearing surface is in addition to present day wearing surface if included in the design. Therefore, the weight of the future wearing surface to be considered is:

Uniform weight: 35 psf

Width of bridge with overlay: 58.83-2(1.75) = 55.33 ft

Line Load: $w = 55.33\,(0.035) = 1.94$ kip/ft

The bridge has a utility opening in one of the interior bays. It will be assumed that the weight of this utility is 0.100 kip/ft.

3.3.3 Downdrag, *DD*

Downdrag, or negative skin friction, can add to the permanent load on the piles. Therefore, if piles are located in an area where a significant amount of fill is to be placed over a compressible soil layer (such as at an abutment), this additional load on the piles needs to be considered.

The downdrag forces are determined by the geotechnical engineers. The additional load due to *DD* and incorporating that load with all other loads provided in the AASHTO-CA BDS-8, Section 10 (Caltrans, 2019a).

3.3.4 Horizontal Earth Pressure, *EH*

Horizontal earth pressure is a load that affects the design of the abutment including the footing, piles, and wing walls. Application follows standard soil mechanics principles.

As an example, the horizontal earth pressure resultant force acting on Abutment 1 of the example bridge is calculated below. This calculation is necessary to determine the total moment demand at the bottom of the abutment stem wall.

Assume: k_a = 0.3, γ_s = 120 pcf and abutment height, H = 30 ft.

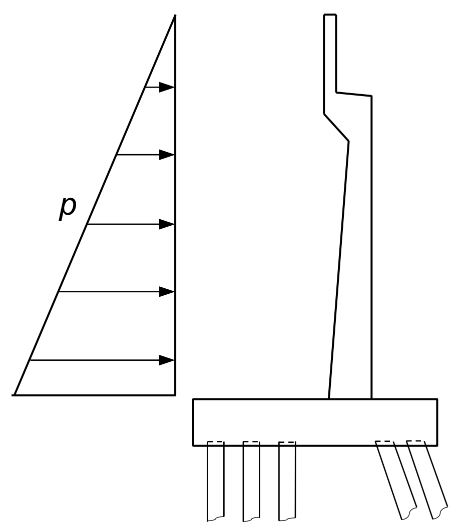


Figure 3.3-4 Abutment 1 with *EH* Load

Note: Refer to the geotechnical report for actual soil properties for a given bridge.

Pressure, $p = k_a \gamma_s z$ (AASHTO 3.11.5.1-1)

where:

z = depth below ground surface

Resultant line load = $\frac{1}{2}k_a\gamma_s z^2 = \frac{1}{2}(0.3)(0.12)(30)^2 = 16.2$ kip / ft

Abutment length = $\frac{58.83}{\cos 20^o} = 62.6$ ft

Total Force = 16.2 (62.6) = 1,014 kips

This force acts at a distance = $H/3$ from the top of footing.

Moment about base of stem wall = $1{,}014\left(\frac{30}{3}\right) = 10{,}140$ kip-ft

3.3.5 Vertical Pressure from Dead Load of Earth Fill, *EV*

Similar to horizontal earth pressure, vertical earth pressure can be calculated using basic mechanics principles. For the 30 ft tall abutment, the weight of earth on the heel at the Abutment 1 footing is obtained as:

Assume distance from heel to back of stem wall = 10.5 ft

$$EV = 10.5\left(\frac{58.83}{\cos 20^o}\right)(30)(0.12) = 2{,}366 \text{ kips}$$

3.3.6 Earth Surcharge, *ES*

This force effect is the result of a concentrated load or uniform load placed near the top of a retaining wall. For Abutment 1, the approach slab is considered an *ES* load.

$\Delta_p = k_s q_s$ (AASHTO 3.11.6.1-1)

$k_s = 0.3$; $q_s = (0.15)(1.0) = 0.150$ ksf; (approach slab thickness = 1 ft)

$\Delta_p = 0.3 \times 0.150 = 0.045$ ksf (*ES* Load)

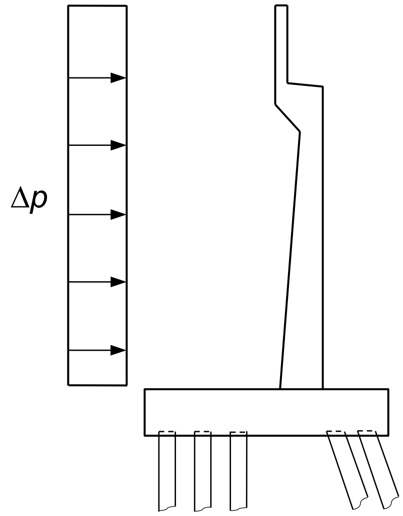


Figure 3.3-5 Abutment 1 with *ES* Load

3.3.7 Force Effect Due to Creep, *CR*

Creep is a time-dependent phenomenon of concrete structures due to sustained compression load. Generally, creep has little effect on the strength of structures, but it will cause prestress losses and leads to increased deflections for service loads (affecting camber calculations). Refer to Chapters 5.1 to 5.5 for more information.

3.3.8 Force Effect Due to Shrinkage, *SH*

Shrinkage of concrete structures occurs as they cure. Shrinkage, like creep, creates a loss in prestress force as the structure shortens beyond the initial elastic shortening due to the axial compressive stress of the prestressing. Refer to Chapters 5.1 to 5.5 for more information.

3.3.9 Forces from Post-Tensioning, *PS*

Post tensioning introduces axial compression into the superstructure. The primary post-tensioning forces counteract dead load forces.

Secondary *PS* forces introduce load into the members of a statically indeterminate structure as the structure shortens elastically toward the point of no movement. These forces can be calculated using the longitudinal frame analysis program, CTBridge. Table 3.3-1 shows the Span 1 and Bent 2 output due to these forces.

Table 3.3-1 PS Secondary Force Effects

PS Secondary Effects After Long Term Losses in Span 1 (All Frames)						
Location (ft)	AX (kips)	VY (kips)	VZ (kips)	TX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
1.5	-3.6	62.3	0	0	0	105.0
12.60	-3.0	61.2	0	0	0	835.2
25.20	-3.0	61.1	0	0	0	1546.1
37.80	-2.7	61.0	0	0	0	2284.9
50.40	-2.6	60.7	0	0	0	3056.0
63.00	-2.3	60.6	0	0	0	3696.1
75.60	-2.3	60.6	0	0	0	4529.9
88.20	-2.2	60.5	0	0	0	5233.8
100.80	-0.1	60.5	0	0	0	6456.1
113.40	-0.3	49.2	0	0	0	6944.2
123.00	0.9	47.2	0	0	0	7013.0
PS Secondary Effects After Long Term Losses in Bent 2, Column 1 (All Frames)						
Location (ft)	AX (kips)	VY (kips)	VZ (kips)	TX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
0.00	32.2	-1.9	0	0	0	0
11.00	32.2	-1.9	0	0	0	20.7
22.00	32.2	-1.9	0	0	0	41.5
33.00	32.2	-1.9	0	0	0	62.2
44.00	32.2	-1.9	0	0	0	83.0
PS Secondary Effects After Long Term Losses in Bent 2, Column 2 (All Frames)						
Location (ft)	AX (kips)	VY (kips)	VZ (kips)	TX (kip-ft)	MY (kip-ft)	MZ (kip-ft)
0.00	32.2	-1.9	0	0	0	0
11.00	32.2	-1.9	0	0	0	20.7
22.00	32.2	-1.9	0	0	0	41.5
33.00	32.2	-1.9	0	0	0	62.2
44.00	32.2	-1.9	0	0	0	83.0

Note: Location is shown from the left end of the span to the right. AX = axial force, VY = vertical shear, VZ = transverse shear, TX = torsion, MY = transverse bending, MZ = longitudinal bending

3.3.10 Locked-in Force Effects from the Construction Process, *EL*

There are instances when a bridge design requires force to be "locked" into the structure during construction. These forces are considered permanent loads and must be included in the analysis. Such an example might be found in a segmental bridge where the cantilever segments are jacked apart before the final closure pour is cast at the midspan. For the example bridge shown above, there are no *EL* forces.

3.4 TRANSIENT LOAD APPLICATION WITH EXAMPLES

For most ordinary bridges there are a few transient loads that should always be considered. Vehicular live loads (*LL*) and their secondary effects including braking force (*BR*), centrifugal force (*CE*), and dynamic load allowance (*IM*) are the most important to consider. These secondary effects are always combined with the gravity effects of live loads in an additive sense.

Uniform Temperature (*TU*) can be quite significant, especially for bridges with long frames and/or short columns. Wind load on structure (*WS*) and wind on live load (*WL*) are significant on structures with tall single column bents over 30 feet. Earthquake load (*EQ*) is specified by Caltrans *Seismic Design Criteria* (SDC) (Caltrans, 2019b) or *Caltrans Seismic Design Specifications for Steel Bridges* (Caltrans, 2016) and generally controls the majority of column designs in California.

3.4.1 Vehicular Live Load, *LL*

Vehicular live load consists of two types of vehicle groups. These are: design vehicular live load (HL-93) and permit vehicle live load (P loads). For both types of loads, axles that do not contribute to extreme force effects are neglected.

3.4.1.1 HL-93 Design Load

The AASHTO HL-93 (Highway Loading adopted in 1993) load includes variations and combinations of truck, tandem, and lane loading. The design truck is a 3-axle truck with variable rear axle spacing and a total weight of 72 kips (Figure 3.4-1). The design lane load is 640 plf (Figure 3.4-2). The design tandem is a two-axle vehicle, 25 kips per axle, spaced 4 ft apart (Figure 3.4-2).

When loading the superstructure with HL-93 loads, only one vehicle per lane is placed on the bridge at a time, except for Cases 3 and 4 (Figure 3.4-2). Trucks are to be placed transversely in as many lanes as practical. Multiple presence factors must be used to account for the probability of multiple fully loaded lanes side by side.

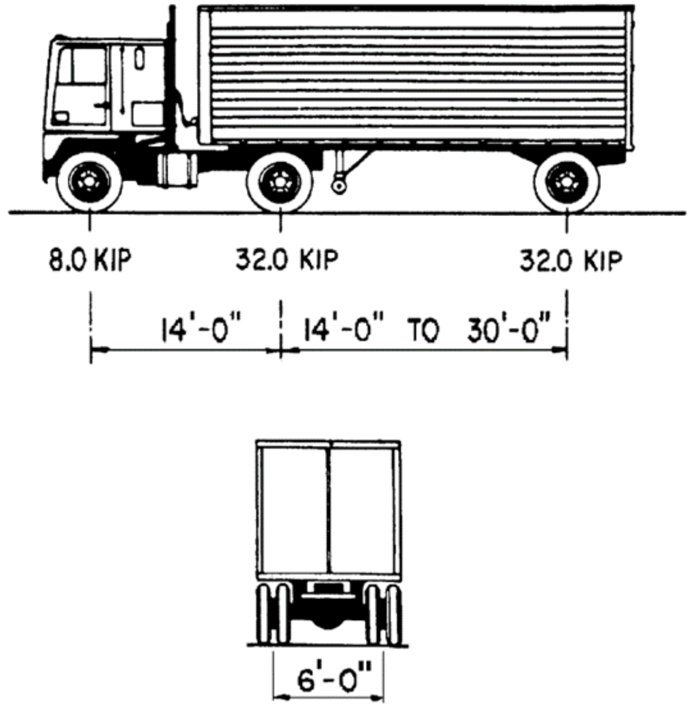


Figure 3.4-1 HL-93 Design Truck

The following 4 cases represent, in general, the requirements for HL-93 loads as shown in Figure 3.4-2. Cases 1 and 2 are for positive moments and associated shears. Cases 3 and 4 are for negative moments and associated shears and bent reactions only. For Case 3, 90 percent of two design trucks and 90 percent of the design lane load are included.

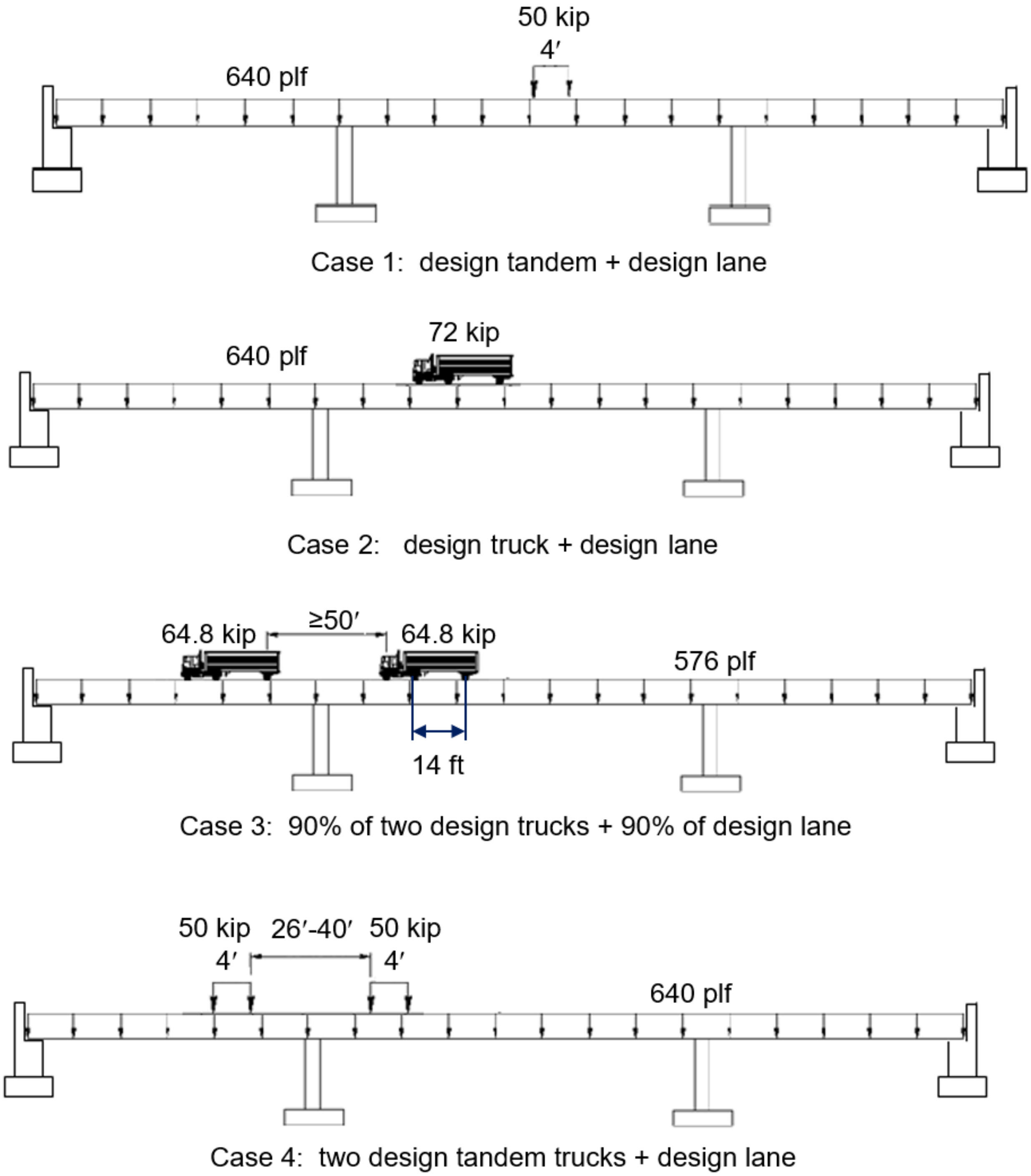


Figure 3.4-2 Four Load Cases for HL-93

Tables 3.4-1 to 3.4-4 list maximum positive moments in Span 2 obtained by the CTBridge program by applying HL-93 loads to the example bridge.

Looking at the Span 2 maximum positive moment only, Cases 1 and 2 apply. Case 1 moment is 6,761 + 4,510 = 11,271 kip-ft while Case 2 moment is 8,696 + 4,510 = 13,206 kip-ft. Case 2 controls (truck + lane). The example bridge has 4.092 live load lanes for maximum positive moment design. Live load distributions will be discussed in detail in Section 3.5. Dynamic load allowance (IM) is included in these tables. IM will be covered in Section 3.4.2.

Table 3.4-1 HL-93 Tandem Forces in Span 2 with IM = 1.33 (for Case 1)

Location (ft)	Positive Moment and Associate Shear				Negative Moment and Associate Shear			
	# Lanes	MZ+ (kip-ft)	Assoc VY (kips)	# Lanes	# Lanes	MZ- (kip-ft)	Assoc VY (kips)	# Lanes
3.00	4.092	995.28	-29.31	5.671	4.231	-4199.51	229.49	5.671
16.80	4.092	1812.60	156.97	5.671	4.231	-2515.05	33.64	5.671
33.60	4.092	3802.37	121.81	5.671	4.231	-2093.17	33.64	5.671
50.40	4.092	5408.64	81.97	5.671	4.092	-1618.17	33.36	5.671
67.20	4.092	6435.96	38.32	5.671	4.092	-1213.65	33.36	5.671
84.00	4.092	6760.64	-184.42	5.671	4.092	-809.13	33.26	5.671
100.80	4.092	6394.21	-229.54	5.671	4.092	-1087.36	-29.70	5.671
117.60	4.092	5333.85	-272.87	5.671	4.092	-1447.46	-29.70	5.671
134.40	4.092	3715.52	-312.24	5.671	4.092	-1809.22	-29.98	5.671
151.20	4.092	1744.94	-346.66	5.671	4.260	-2265.74	-178.53	5.671
165.00	4.092	1126.74	32.97	5.671	4.260	-4400.92	-229.07	5.671

Table 3.4-2 HL-93 Design Truck Forces in Span 2 with IM = 1.33 (for Case 2)

Location (ft)	Positive Moment and Associate Shear				Negative Moment and Associate Shear			
	# Lanes	MZ+ (kip-ft)	Assoc VY (kips)	# Lanes	# Lanes	MZ- (kip-ft)	Assoc VY (kips)	# Lanes
3.00	4.092	1394.51	-41.07	5.671	4.231	-5950.35	321.25	5.671
16.80	4.092	1675.25	187.00	5.671	4.231	-3537.59	47.32	5.671
33.60	4.092	4546.16	135.41	5.671	4.231	-2944.18	47.32	5.671
50.40	4.092	6836.54	77.46	5.671	4.092	-2276.07	46.92	5.671
67.20	4.092	8272.63	14.47	5.671	4.092	-1707.09	46.92	5.671
84.00	4.092	8696.12	-194.92	5.671	4.092	-1138.11	46.78	5.671
100.80	4.092	8215.35	-259.64	5.671	4.092	-1523.62	-41.61	5.671
117.60	4.092	6730.27	-322.23	5.671	4.092	-2028.18	-41.61	5.671
134.40	4.092	4419.14	-379.54	5.671	4.092	-2535.07	-42.00	5.671
151.20	4.092	1570.83	-430.11	5.671	4.260	-3189.63	-252.06	5.671
165.00	4.092	1584.82	46.37	5.671	4.260	-6238.76	-329.02	5.671

Table 3.4-3 HL-93 Lane Forces in Span 2 with IM = 1.0 (for Cases 1 & 2)

Location (ft)	Positive Moment and Associate Shear				Negative Moment and Associate Shear			
	# Lanes	MZ+ (kip-ft)	Assoc VY (kips)	# Lanes	# Lanes	MZ- (kip-ft)	Assoc VY (kips)	# Lanes
3.00	4.092	741.53	-12.38	5.671	4.231	-6369.30	308.97	5.671
16.80	4.092	852.84	37.25	5.671	4.231	-3686.97	209.28	5.671
33.60	4.092	1720.77	103.01	5.671	4.231	-1874.84	82.57	5.671
50.40	4.092	3069.59	120.20	5.671	4.092	-1280.05	4.59	5.671
67.20	4.092	4159.12	59.39	5.671	4.092	-1226.01	4.44	5.671
84.00	4.092	4509.62	-5.00	5.671	4.092	-1172.13	4.28	5.671
100.80	4.092	4123.39	-62.34	5.671	4.092	-1120.20	4.28	5.671
117.60	4.092	2998.15	-123.10	5.671	4.092	-1068.46	4.08	5.671
134.40	4.092	1709.11	-95.19	5.671	4.092	-1591.40	-84.70	5.671
151.20	4.092	942.33	-29.62	5.671	4.260	-3513.82	-211.24	5.671
165.00	4.092	894.23	17.29	5.671	4.260	-6220.60	-308.23	5.671

Table 3.4-4 HL-93 Design Vehicle Enveloped Forces in Span 2 with IM = 1.33 (Cases 3 & 4)

Location (ft)	Positive Moment and Associate Shear				Negative Moment and Associate Shear			
	# Lanes	MZ+ (kip-ft)	Assoc VY (kips)	# Lanes	# Lanes	MZ- (kip-ft)	Assoc VY (kips)	# Lanes
3.00	4.092	2136.04	-53.45	5.671	4.231	-14708.22	613.18	5.671
16.80	4.092	2665.45	194.23	5.671	4.231	-9177.29	454.12	5.671
33.60	4.092	6266.93	238.42	5.671	4.231	-4906.59	223.59	5.671
50.40	4.092	9906.14	197.65	5.671	4.092	-3556.12	51.52	5.671
67.20	4.092	12431.75	73.85	5.671	4.092	-2933.10	51.37	5.671
84.00	4.092	13205.74	-199.92	5.671	4.092	-2310.24	51.06	5.671
100.80	4.092	12338.74	-321.97	5.671	4.092	-2643.82	-37.33	5.671
117.60	4.092	9728.43	-445.33	5.671	4.092	-3096.64	-37.53	5.671
134.40	4.092	6128.26	-474.72	5.671	4.092	-4126.47	-126.71	5.671
151.20	4.092	2687.27	-376.27	5.671	4.260	-8884.22	-457.04	5.671
165.00	4.092	2479.04	63.66	5.671	4.260	-14454.96	-611.30	5.671

3.4.1.2 Permit Load

The California P15 permit (CA 3.6.1.8) vehicle is used in conjunction with the Strength II limit state. For superstructure design, if simplified distribution is used (Article 4.6.2.2), girder distribution factors are the same as the design vehicle distribution factors since the transverse tire spacing for the permit vehicle is the same as for the HL-93 design vehicle. If refined methods are used, either 1 or 2 permit trucks (without the HL-93 design vehicle), whichever controls, are placed on the bridge at a time, to the maximum effect. For bent cap design since lanes are located across the cap, either 1 or 2 permit trucks, which ever controls, are placed to the maximum effect. Table 3.4-5 shows the maximum positive moments in Span 2 obtained from CTBridge.

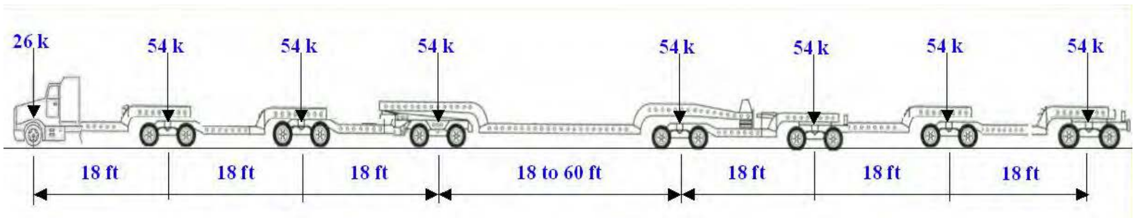


Figure 3.4-3 P15 Permit Truck

Table 3.4-5 Permit Moments in Span 2 with IM = 1.25

Location (ft)	Positive Moment and Associate Shear				Negative Moment and Associate Shear			
	# Lanes	MZ+ (kip-ft)	Assoc VY (kips)	# Lanes	# Lanes	MZ- (kip-ft)	Assoc VY (kips)	# Lanes
3.00	4.092	4301.37	-126.88	5.671	4.231	-24408.33	1094.85	5.671
16.80	4.092	3037.28	-126.88	5.671	4.231	-13982.37	920.76	5.671
33.60	4.092	8953.21	595.24	5.671	4.231	-9737.27	156.42	5.671
50.40	4.092	18103.51	500.79	5.671	4.092	-7528.58	155.11	5.671
67.20	4.092	24145.22	155.37	5.671	4.092	-5647.76	155.11	5.671
84.00	4.092	26029.15	-34.87	5.671	4.092	-3766.94	154.62	5.671
100.80	4.092	23859.78	-498.73	5.671	4.092	-4712.92	-128.55	5.671
117.60	4.092	17812.83	-498.73	5.671	4.092	-6271.56	-128.55	5.671
134.40	4.092	8607.82	-798.23	5.671	4.092	-7837.40	-129.75	5.671
151.20	4.092	3707.40	153.28	5.671	4.260	-13797.79	-947.21	5.671
165.00	4.092	5233.90	153.28	5.671	4.260	-24485.57	-1462.71	5.671

Notice that the maximum P15 moment of 26,029 kip-ft exceeds the HL-93 moment of 13,206 kip-ft. Although load factors have not yet been applied, Strength II will govern over Strength I in the majority of bridge superstructure design elements.

When determining the force effects on a section due to live load, the maximum moment and its associated shear, or the maximum shear and its associated moment should be considered. Combining maximum moments with maximum shears simultaneously for a section is too conservative.

3.4.1.3 Fatigue Load

There are two fatigue load limit states used to ensure the structure withstands cyclic loading. A single HL-93 design truck with rear axle spacing of 30 ft must be run across the bridge by itself for the first case. The second case is a P9 truck by itself. Dynamic load allowance is 15% for these cases.

3.4.1.4 Multiple Presence Factors (*m*)

To account for the improbability of fully loaded trucks crossing the structure side-by-side, multiple presence factors, *m*, are applied as follows:

Table 3.4-6 Multiple Presence Factors

Number of Loaded Lanes	Multiple Presence Factors, *m*
1	1.2
2	1.0
3	0.85
>3	0.65

3.4.2 Vehicular Dynamic Load Allowance, *IM*

To capture the “bouncing” effect and the resonant excitations due to moving trucks, the static truck live loads or their effects are increased by the percentage of the vehicular dynamic load allowance, *IM* as specified by CA Article 3.6.2.

For example, the maximum HL-93 static moment at the midspan of Span 2 due to the design truck is 6,538 kip-ft. The static moment due to the lane load is 4,510 kip-ft. The dynamic load allowance for the HL-93 load case is 33%. Therefore, $LL + IM = 1.33(6{,}538) + 4{,}510 = 13{,}206$ kip-ft. Note that *IM* does not apply to the lane load.

The Permit static moment at the midspan of Span 2 is 20,823 kip-ft. Dynamic load allowance for Permit is 25%. Therefore, $LL+IM = 1.25(20{,}823) = 26{,}029$ kip-ft.

3.4.3 Vehicular Braking Force, *BR*

This force accounts for traction (acceleration) and braking. It is a lateral force acting in the longitudinal direction and primarily affects the design of columns and bearings.

For the example bridge, *BR* is the greater of the following (Article 3.6.4):

1) 25% of the axle weight of the Design Truck or Design Tandem
2) 5% of (Design Truck + Lane Load) or 5% of (Design Tandem + Lane Load)

There are 4 cases to consider. Calculating *BR* force for one lane of traffic results in the following:

Case 1)	25% of Design Truck:	0.25(72) = 18.0 kips
Case 2)	25% of Design Tandem:	0.25(50) = 12.5 kips
Case 3)	5% of truck + lane:	0.05(72 + (412)(0.64)) = 16.8 kips
Case 4)	5% of tandem + lane:	0.05(50 + (412)(0.64)) = 15.7 kips

It is seen that Case 1 controls at 18.0 kips. For column design, this one lane result must be multiplied by as many lanes as practical considering the multiple presence factor, *m*. The maximum number of lanes that can fit on this structure is determined by using 12.0 ft traffic lanes:

Number of lanes: $\dfrac{58.83-2(1.75)}{12}=4.61$ lanes

Dropping the fractional portion, 4 lanes will fit.

The controlling BR force is therefore the maximum of:

1)	One lane only:	(18.0)(1.2)(1)	= 21.6 kips
2)	Two lanes:	(18.0)(1.0)(2)	= 36.0 kips
3)	Three lanes:	(18.0)(0.85)(3)	= 45.9 kips
4)	Four lanes:	(18.0)(0.65)(4)	= 46.8 kips

Four lanes control at 46.8 kips. This force is a horizontal force to be applied at deck level in the longitudinal direction resulting in shear and bending moments in the columns. In order to determine these column forces, a longitudinal frame model can be used, such as in the CTBridge program. Apply a user load to a superstructure member in the longitudinal direction and input the load factors.

When a percentage of the truck weight is used to determine *BR*, only that portion of the truck that fits on the bridge is utilized. For example, if the bridge total length is 25 ft, then only the two 32 kip axles that fit are used for *BR* calculations.

3.4.4 Vehicular Centrifugal Force, *CE*

Horizontally curved bridges are subject to *CE* forces. These forces primarily affect substructure design. The sharper the curve is, the higher these forces will be. These forces act in a direction that is perpendicular to the alignment and toward the outside of the curve. Centrifugal forces apply to both HL-93 live load (truck and tandem only) and Permit live load. Dynamic load allowance does not apply to these calculations.

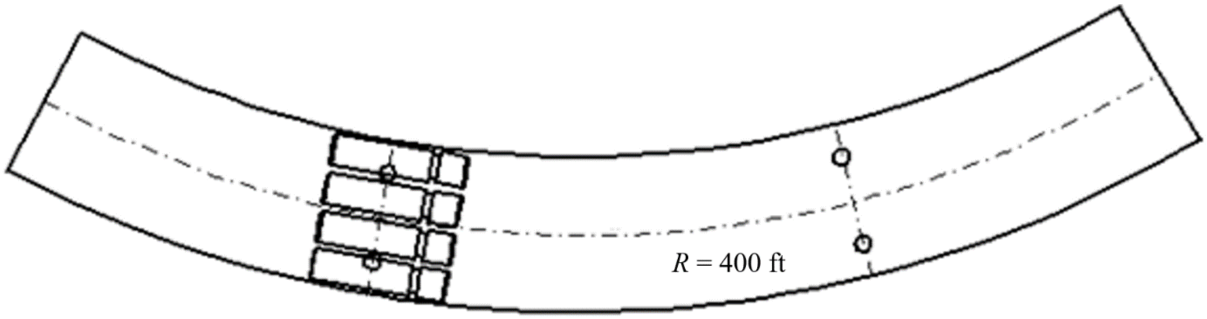


Figure 3.4-4 Centrifugal Force Example

$$C = f\left(\frac{v^2}{gR}\right)$$ (AASHTO 3.6.3-1)

Example

Assume: v = 70 mph (Highway Design Speed)

f = 4/3 (Strength I Load combination)

Reaction of one lane of HL-93 truck at Bent 2 = 71.6 kips

Reaction of one lane of HL-93 tandem at Bent 2 = 50.0 kips

R = 400 ft

Convert v to feet per second:

$$v = \left(70\frac{\text{miles}}{\text{hr}}\right)\left(\frac{1\text{ hr}}{3600\text{ sec}}\right)\left(\frac{5280\text{ ft}}{1\text{ mile}}\right) = 102.7\text{ ft/sec}$$

$$C = \frac{4}{3}\left(\frac{102.7^2}{(32.2)(400)}\right) = 1.092$$

Total shear for 4 lanes over Bent 2 simultaneously:

Shear = 1.092(71.6)(4)(0.65) = 203.3 kips

3.4.5 Live Load Surcharge, *LS*

This load shall be applied when trucks can come within one half of the wall height at the top of the wall on the side of the wall where earth is being retained.

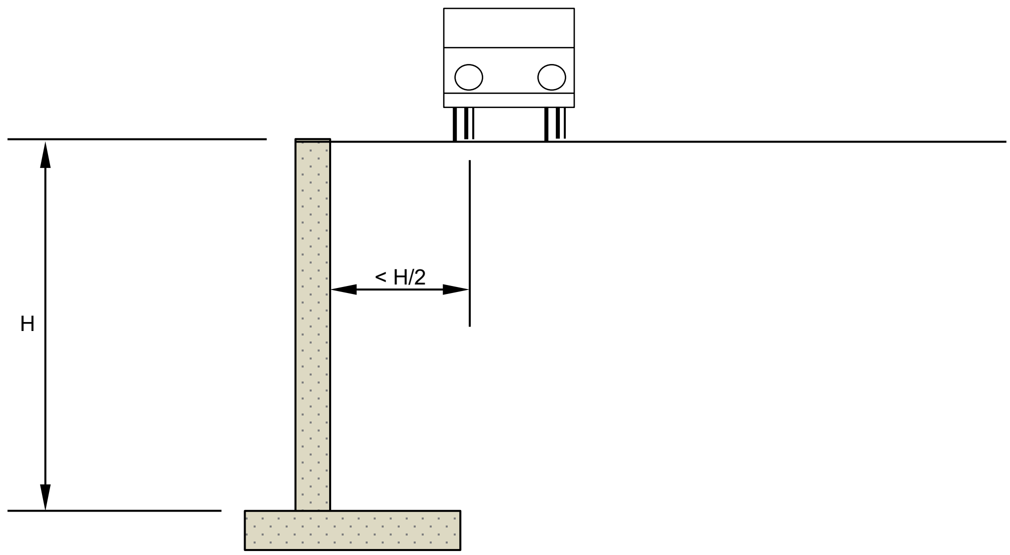


Figure 3.4-5 Applicability of Live Load Surcharge

When the condition of Figure 3.4-5 is met, then the following constant horizontal earth pressure may be estimated as:

$$\Delta_p = k\gamma_s h_{eq} \quad \text{(AASHTO 3.11.6.4-1)}$$

An equivalent height of soil is used to approximate the effect of live load acting on the fill. Refer to AASHTO Table 3.11.6.4-1. For the example bridge, the live load surcharge for Abutment 1 is calculated as follows:

Abutment Height = 30 ft

h_{eq} = 2.0 ft

Δ_p = 0.3(0.12)(2.0) = 0.072 ksf

Loading is similar to *ES* as shown in Figure 3.3-5.

3.4.6 Pedestrian Live Load, *PL*

Pedestrian live loads (*PL*) are assumed to be a uniform load accounting for the presence of large crowds, parades, and regular use of the bridge by pedestrians. Pedestrian live load can act alone or in combination with vehicular loads if the bridge is designed for mixed use.

This load is investigated when pedestrians have access to the bridge. Either the bridge will be designed as a pedestrian overcrossing or will have a sidewalk where both vehicles and pedestrians utilize the same structure.

The *PL* load is 75 psf vertical pressure on sidewalks wider than 2 ft. For pedestrian overcrossings (POCs) the vertical pressure is 90 psf as specified in the AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges (AASHTO, 2009b).

The example bridge does not have a sidewalk and would therefore not need to be designed for pedestrian live load.

3.4.7 Uniform Temperature, *TU*

Superstructures will either expand or contract due to changes in temperature. This movement will introduce additional forces in statically indeterminate structures and results in displacements at the bridge joints and bearings that need to be considered. These effects can be rather large in some instances.

The design thermal range for which a structure must be designed is shown in AASHTO Table 3.12.2.1-1.

AASHTO Table 3.12.2.1-1 Procedure A Temperature Ranges

Climate	Steel or Aluminum	Concrete	Wood
Moderate	0° to 120°F	10° to 80°F	10° to 75°F
Cold	-30° to 120°F	0° to 80°F	0° to 75°F

For the example bridge, column movements due to a uniform temperature change are calculated below. This can be accomplished using a frame analysis program such as CSiBridge or CTBridge. A hand method is shown below. To start, calculate the point of no movement. The following relative stiffness method can be used to accomplish this.

Table 3.4-7 Center of Stiffness Calculation

	Abut 1	Bent 2	Bent 3	Abut 4	SUM
P@1″ (kip/in.)	0	206	169	0	375
D (ft)	0	126	294	412	-
PD/100	0	260	497	0	757

Force to deflect the top of column by 1 in. (P@1 in.) can be determined from:

$$P = \frac{3EI_{col}\Delta}{L^3} \text{ (for fixed-pinned columns)}$$

where

D = accumulated distance from reference point, Abutment 1, to a support.

Δ = 1 in.; E = 3,834 ksi; $I_{col} = \frac{\pi r^4}{4}$; L = 44 ft at Bent 2, 47 ft at Bent 3;

r = 3.0 ft

$$\text{The point of no movement} = \frac{\sum \frac{PD}{100}}{\sum P}(100) = \frac{757}{375}(100) = 201.9 \text{ ft}$$

The factor of 100 is used to keep the numbers small and can be factored out if preferred. This point of no movement is the location from Abutment 1 where no movement is expected due to uniform temperature change.

Next determine the rise or fall in temperature change. From AASHTO Table 3.12.2.1-1, assuming a moderate climate, the temperature range is 10 to 80°F. Design thermal movement range for force effects is determined by the following formula:

$$\Delta_T = \alpha L\left(T_{MaxDesign} - T_{MinDesign}\right)/2 \qquad \text{(AASHTO-CA 3.12.2.1-1)}$$

Using a temperature change of +/-70°F, we can now determine a movement factor using concrete properties.

α = coefficient of thermal expansion for a given material

= 0.000006 / °F (for concrete)

$$\text{Thermal Movement} = \left(0.000006/{}^\circ\text{F}\right)\left(70^\circ\text{F}\right)\left(1200\frac{\text{in}}{100\text{ ft}}\right)/2 = 0.25 \text{ in / 100 ft}$$

The movement at each bent is then calculated (movement at abutments is determined in a similar fashion):

$$\text{Bent 2} = (0.25)\frac{(201.9-126)}{100} = 0.19 \text{ in.}$$

$$\text{Bent 3} = (0.25)\frac{(294 - 201.9)}{100} = 0.23 \text{ in.}$$

The factored load is calculated using γ_{TU} = 0.5 for force effects.

Design thermal movement range for joints and bearings is determined by the following formula:

$$\Delta T = \alpha L(T_{MaxDesign} - T_{MinDesign}) \qquad \text{(AASHTO-CA 3.12.2.3-1)}$$

For joint displacements and bearing design the larger factor γ_{TU} = 1.2 is used. Refer to Chapter 14.2 for expansion joint calculations.

3.4.8 Temperature Gradient, *TG*

Bridge decks are exposed to the sunlight thereby causing them to heat up much faster than the bottom of the superstructure. This thermal gradient can induce additional stresses in the statically indeterminate structure. For simply-supported or well-balanced framed bridge types with span lengths less than 200 ft this effect can be safely ignored. If, however, your superstructure is built using very thick concrete members, or for structures where mass concrete is used, thermal gradients should be investigated especially in an environment where air temperature fluctuations are extreme.

3.4.9 Settlement, *SE*

Differential settlement of supports causes force effects in statically indeterminate structures. A predefined maximum settlement of 1 in. for continuous spans or simple spans with diaphragm abutments or 2 in. for simple spans with seat abutments is generally assumed for foundation design. At this level of settlement, ordinary bridges will not be significantly affected if the actual differential settlement is not expected to exceed ½ inch. When the differential settlement is likely to exceed ½ inch, force effects due to predefined settlements shall be included in the design of the superstructure. If, however, this criterion makes the foundation cost unacceptable, larger settlements may be allowed. In that case, settlement analysis will be required.

For example, if an actual differential settlement of one inch for the example bridge is assumed, one would have to consider loads generated by *SE* and check the superstructure under Strength load combinations. To perform this analysis, assume Bent 2 doesn't settle. Then allow Bent 3 to settle one inch. Force effects that result from this scenario become *SE* loads.

3.4.10 Water Load and Stream Pressure, *WA*

The example bridge can be modified by assuming Bent 2 is a pier in a stream as shown in Figure 3.4-6. See the figure below for the pier configuration.

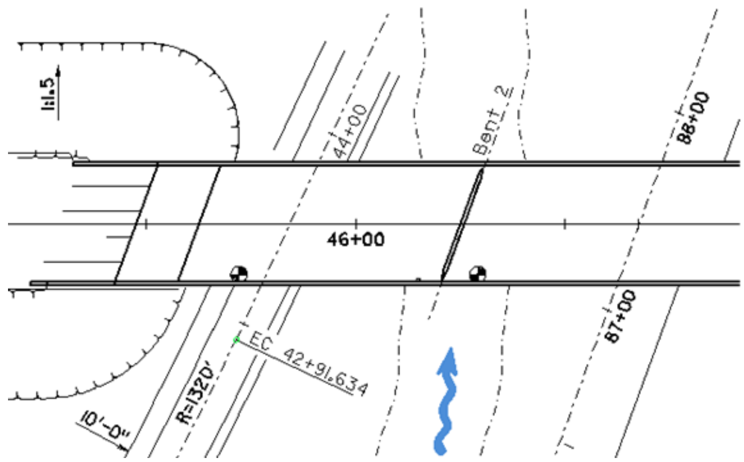


Figure 3.4-6 Stream Flow Example

Assume the angle between stream flow and the pier is 10 degrees and the stream flow velocity is 6.0 fps. The pressure on the pier in the direction of the longitudinal axis of the pier is calculated by:

$$p = \frac{C_D V^2}{1000} \qquad \text{(AASHTO 3.7.3.1-1)}$$

Assume: Pier is 56 feet long

Skew = 10°

C_D = 0.7 (semicircular-nosed pier)

C_L = 0.7 (lateral drag for 10 degree flow direction)

V = 6.0 fps

$$p = \frac{0.7 \times 6^2}{1000} = 0.0252 \text{ ksf}$$

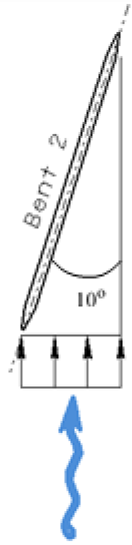


Figure 3.4-7 Longitudinal to Pier Forces due to Stream Flow

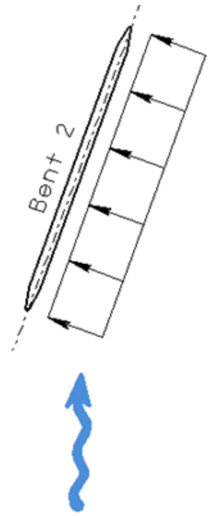


Figure 3.4-8 Transverse to Pier Forces due to Stream Flow

This pressure is applied to the pier's projected area. Assume here the distance from the river bottom to the high-water elevation is 12 ft.

Total pier force = 0.0252 (56) sin (10°)(12) = 2.94 kips

Then, pressure on the pier in the direction perpendicular to the axis of the pier is calculated using the following:

$$p = \frac{C_L V^2}{1000} \qquad \text{(AASHTO 3.7.3.2-1)}$$

$$p = \frac{0.7 \times 6^2}{1000} = 0.0252 \text{ ksf}$$

Total pressure on the pier in the lateral direction is therefore:

Total pier force = 0.0252(56)(12) = 16.93 kips

3.4.11 Wind Load on Structure, *WS*

Wind load is based on a base wind velocity that is increased for bridges taller than 33 ft from ground to top of area exposed to wind which is usually the barrier but may be a sound wall on top of the bridge. Wind load primarily affects the substructure design.

Using the example bridge, calculate wind load on the structure as shown below.

First find the design wind velocities from AASHTO Table 3.8.1.1.2-1. Strength III wind speed is taken from Figure 3.8.1.1.2-1, Strength V is 80 mph, Service I is 70 mph and Service IV is 0.75 of the speed used in Strength III. The design wind speed is 110 mph for all of California except special wind regions. The example bridge is not located in a special zone so we'll use 110 mph.

Assume the bridge is in 'open country' with an average height from ground to top of barrier equal to 50.25 ft.

Next, a design wind pressure, *P* is calculated.

$$P = 2.56x10^{-6} V^2 K_Z G C_D \qquad \text{(AASHTO 3.8.1.2.1-1)}$$

With wind acting normal to the structure (skew = 0 degree),

V = 110 mph (AASHTO Figure 3.8.1.1.2-1)

Z = 50.25

Open terrain corresponds to the ground surface roughness category C and the wind exposure category C so the pressure exposure and elevation coefficient $K_Z(C)$ will be used. The value for K_Z(C) can be calculated from AASHTO equation 3.8.1.2.1-3 or the value may be selected from AASHTO Table C3.8.1.2.1-1.

$$K_Z(C) = \frac{\left[2.5\ln\left(\frac{Z}{0.0984}\right) + 7.35\right]^2}{478.4}$$

$$= \frac{\left[2.5\ln\left(\frac{50.25}{0.0984}\right) + 7.35\right]^2}{478.4} = 1.10 \qquad \text{(AASHTO 3.8.2.1-3)}$$

G = 1.00 (for structures other than sound barriers) (AASHTO Table 3.8.1.2.1-1)

C_D = 1.3 (for box-girder bridge superstructures) (AASHTO Table 3.8.1.2.1-2)

$$P = 2.56x10^{-6} V^2 K_Z G C_D = 2.56x10^{-6}(110)^2(1.10)(1.00)(1.3) = 0.044 \text{ ksf}$$

For the wind pressure on the columns, the drag coefficient for bridge substructures must be used. Since the bridge substructure does not extend above the elevation of the superstructure, the same *Z* as the superstructure may be used.

C_D = 1.6 (for bridge substructures) (AASHTO Table 3.8.1.2.1-2)

$$P = 2.56x10^{-6} V^2 K_Z G C_D = 2.56x10^{-6}(110)^2(1.10)(1.00)(1.6) = 0.055 \text{ ksf}$$

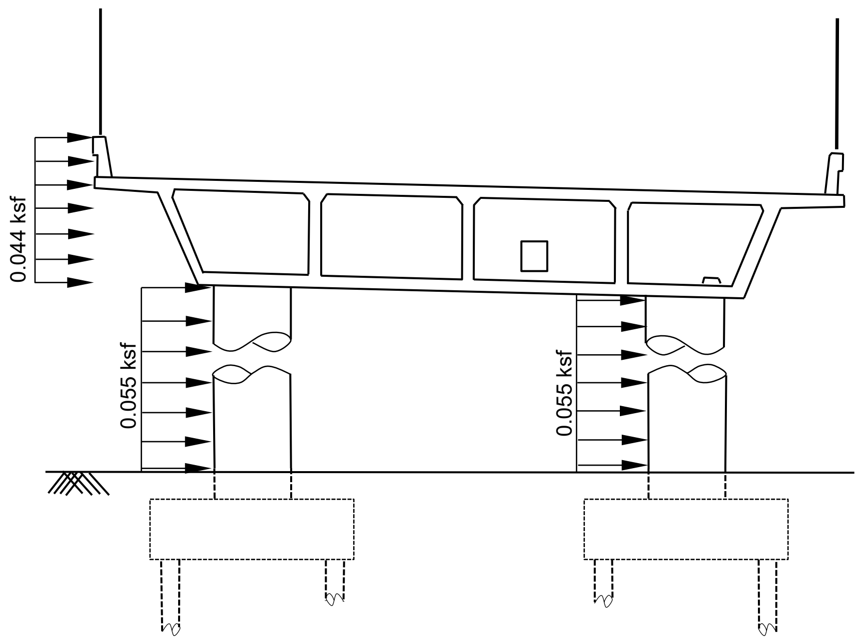


Figure 3.4-9 WS Application

It is convenient to turn these pressures into line loads for application to a frame analysis model.

Load on the spans = (6.75 + 3.00) 0.044 = 0.429 klf

Load on the columns = (6.00) 0.055 = 0.330 klf

WS load application within a statically indeterminate frame model is shown in Figure 3.4-10.

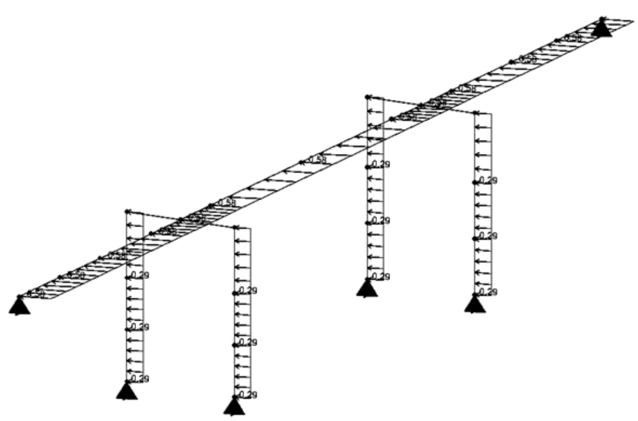


Figure 3.4-10 Wind on Structure

For the superstructure use AASHTO Table 3.8.1.2.3a-1 to calculate the pressure from various angles skewed from the perpendicular to the longitudinal axis. Results are shown below in Table 3.4-8. The “Trusses, Columns, and Arches” heading in the AASHTO table refers to superstructure elements. The table refers to spandrel columns in a superstructure not pier/substructure columns. Transverse and longitudinal pressures should be applied simultaneously.

Table 3.4-8 Wind Load at Various Angles of Attack

	Superstructure			
Skew Angle of Attack	*Transverse Skew Coefficient*	*Pressure (ksf)*	*Longitudinal Skew Coefficient*	*Pressure (ksf)*
0	1.000	0.044	0.000	0.000
15	0.880	0.039	0.120	0.005
30	0.820	0.036	0.240	0.011
45	0.660	0.029	0.320	0.014
60	0.340	0.015	0.380	0.017

For application to the substructure, the transverse and longitudinal superstructure wind forces are resolved into components aligned relative to the pier axes.

Load perpendicular to the plane of the pier:

$$F_L = F_{L,super} \cos(20°) + F_{T,super} \sin(20°)$$

At 0 degrees:

$$F_L = 0(6.75 + 3.00)\cos(20°) + 0.044(6.75 + 3.00) \sin(20°) = 0.147 \text{ klf}$$

At 60 degrees

$$F_L = 0.017(6.75 + 3.00) \cos(20°) + 0.015(6.75 + 3.00) \sin(20°)$$
$$= 0.156 \text{ klf} + 0.050 \text{ klf} = 0.206 \text{ klf}$$

And load in the plane of the pier (parallel to the columns):

$$F_T = F_{L,super} \sin(20°) + F_{T,super} \cos(20°)$$

At 0 degrees:

$$F_T = 0(9.75) \sin(20°) + 0.044(9.75)\cos(20°) = 0.403 \text{ klf}$$

At 60 degrees:

$$F_T = 0.017(9.75) \sin(20°) + 0.015(9.75) \cos(20°)$$
$$= 0.057 \text{ klf} + 0.137 \text{ klf} = 0.194 \text{ klf}$$

The wind pressure applied directly to the substructure is resolved into components perpendicular to the end and front elevations of the substructure. The pressure perpendicular to the end elevation of the pier is applied simultaneously with the wind load from the superstructure in the same direction.

3.4.12 Wind on Live Load, *WL*

This load is applied to vehicles traveling on the bridge during periods of a moderately high wind of 80 mph. The 80 mph 3-second gust wind speed with a load factor of 1.0 is approximately equivalent to past editions 55-mph fastest-mile wind speed with a load factor of 1.4. This load is to be 0.1 klf applied transverse to the bridge deck as shown in Figure 3.4-11.

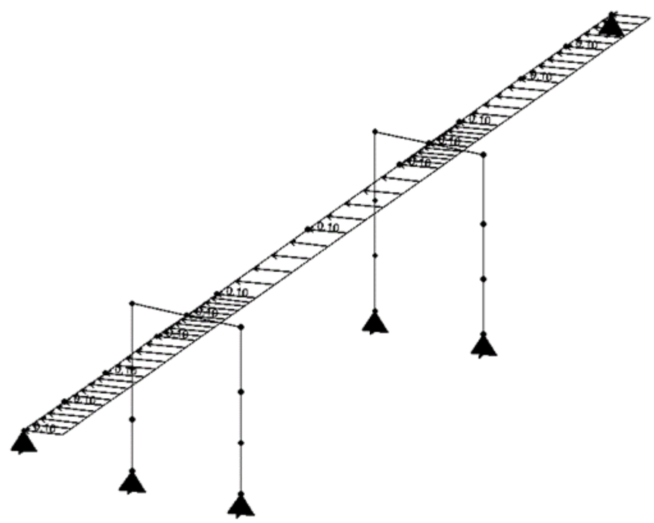


Figure 3.4-11 Wind on Live Load

3.4.13 Friction, *FR*

Friction loading can be any loading that is transmitted to an element through a frictional interface. There are no *FR* forces for the example bridge.

3.4.14 Ice Load, *IC*

The presence of ice floes in rivers and streams can result in extreme event forces on the pier. These forces are a function of the ice crushing strength, thickness of ice floe, and width of pier. For equations and commentary on ice load, see AASHTO Article 3.9. Snow load/accumulation on a bridge need not be considered in general.

3.4.15 Vehicular Collision Force, *CT*

Vehicle collision refers to collisions that occur with the barrier rail or at unprotected columns (Article 3.6.5).

Referring to AASHTO Section 13, the design loads for *CT* forces on barrier rails are as shown in AASHTO Table A13.2-1. Test Level Four (TL-4) will apply most of the time.

These forces are applied to our Type 836 barrier rail from our example bridge as follows:

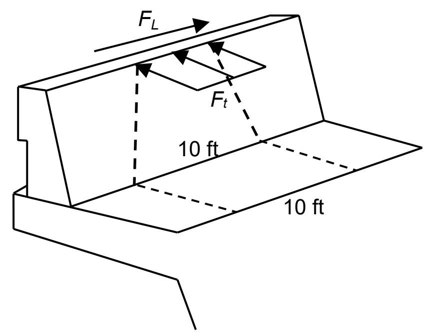


Figure 3.4-12 *CT* Force on Barrier

F_t = 54 kips

F_L = 18 kips

Load from this collision force spreads out over a width calculated based on detailing of the barrier bar reinforcement and yield line theory. Caltrans policy is to assume this distance to be 10 ft at the base of the barrier for Standard Plan barriers that are solid. Given that the barrier height is 3'-0", we can calculate the moment per foot as follows:

$$M_{CT} = \frac{54 \times 3.0}{10} = 16.2 \text{ kip-ft/ft}$$

Applying a 20% factor of safety (CA A13.4.2) results in:

$1.2 \times 16.2 = 19.4$ kip-ft/ft

Standard plan barriers have already been designed for these *CT* forces. However, these forces must be carried into the overhang and deck. Overhang design shall include *CT* forces. For a bridge with a long overhang or an unusual typical section configuration, for which the deck design charts do not apply, calculations for *CT* force should be performed.

Post-type (see-through) barriers require special analysis for various failure modes and are not covered here.

3.4.16 Vessel Collision Force, *CV*

Generally, California bridges over navigable waterways are protected by a fender system. In these instances, the fender system is then subject to the requirements of AASHTO 3.14 and/or the AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges (AASHTO, 2009a). If the bridge piers are not protected by a fender system, they must be designed to withstand the *CV* force. Due to the infrequent occurrence of these bridges, an example of *CV* force calculations will not be made here.

3.4.17 Blast Loading, *BL*

The importance of the bridge is used as the basis for determining whether a bridge should be designed for blast forces. More information on blast loading design can be found in the AASHTO Bridge Security Guidelines, 2nd Edition (AASHTO, 2022). Blast loading is not included for the example bridge.

3.4.18 Earthquake, *EQ*

In California, a high percentage of bridges are close enough to a major fault to be controlled by *EQ* forces. *EQ* loads are a function of structural mass, structural period, and the Acceleration Response Spectrum (ARS). The ARS curve is determined from a Caltrans online mapping tool or supplied by the Office of Geotechnical Services. These requirements will be covered in detail in Chapter 20.1. It is recommended that *EQ* forces be considered early in the design process in order to properly size members.

3.5 LOAD DISTRIBUTION FOR GIRDER BRIDGES

3.5.1 Permanent Loads

Load distribution for permanent loads follows standard structure mechanics methods. There are, however, a few occasions where assumptions are made to simplify the design process, rather than follow an exact load distribution pathway.

3.5.1.1 Barriers

Barrier loads are generally distributed equally to all girders in the superstructure section (Figure 3.5-1). The weight of the barrier is light enough that a more detailed method of distribution is not warranted.

For the example bridge, *DC* load for barriers is 1.12 klf for two barriers. The barrier load to each girder is simply 1.12/5 = 0.224 klf (Figure 3.5-1).

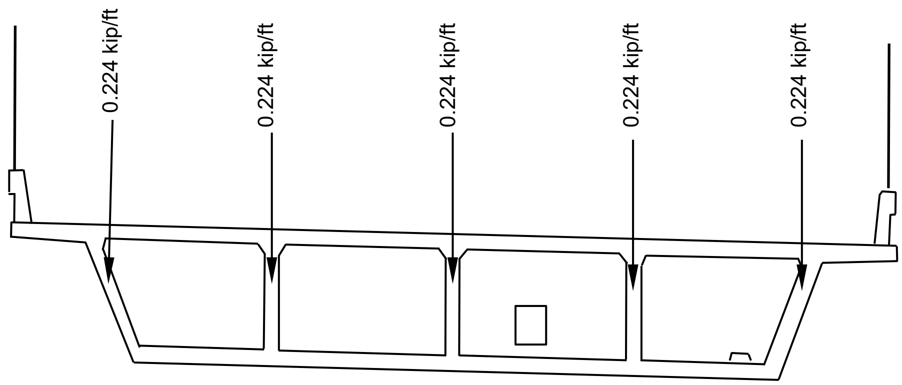


Figure 3.5-1 Barrier Distribution

3.5.1.2 Sound Walls

Since a sound wall has a much higher load per lineal length than a barrier, a more refined analysis may be performed to obtain more accurate distribution. The following procedure is specified in Caltrans STP 4.3 (Caltrans, 2021) for sound wall dead load distribution on box girder bridges. A separate procedure is also specified in STP 4.3 for beam-slab bridges.

Sound wall distribution is simplified by applying a percentage of the sound wall dead load to the girders.

For shear, 100% of the sound wall dead load demand is applied to the exterior girder closest to the sound wall. 1/*n* of the sound wall dead load demand is applied to the first interior girder, where *n* is the number of girders. The shear demand in the remaining girders is assumed to be unaffected by the presence of the sound wall so the sound wall dead load demand need not be included for these girders.

For moment, 25% + 1/*n* of the sound wall dead load demand is applied to the exterior girder closet to the sound wall, where *n* is the number of girders. 25% + 1/*n* of the sound wall dead load demand is also applied to the first interior girder, and 1/*n* of the sound wall dead load demand is applied to the remaining girders.

For the example bridge, assume a sound wall 10 ft tall using 8-inch blocks on the north side of the bridge instead of the Type 7 chain link railing. The approximate weight per foot assuming solid grouting is 88 psf × 10 ft = 880 plf. Applying this load in a 2-D frame program such as CTBridge, the results are shown in Table 3.5-1.

Table 3.5-1 Unfactored Sound Wall Forces

Location	Whole Bridge		Apply to Adjacent Exterior Girder		Apply to First Interior Girder		Apply to Remining Girders	
	VY (kip)	MZ (kip-ft)	VY (kip)	MZ (kip-ft)	VY (kip)	MZ (kip-ft)	VY (kip)	MZ (kip-ft)
Span 1								
1.5	38.3	58.5	38.3	26.3	7.7	26.3	0.0	11.7
12.6	28.5	429.8	28.5	193.4	5.7	193.4	0.0	86.0
25.2	17.5	719.8	17.5	323.9	3.5	323.9	0.0	144.0
37.8	6.4	870.0	6.4	391.5	1.3	391.5	0.0	174.0
50.4	-4.7	880.4	-4.7	396.2	-0.9	396.2	0.0	176.1
63.0	-15.8	751.1	-15.8	338.0	-3.2	338.0	0.0	150.2
75.6	-26.9	482.1	-26.9	216.9	-5.4	216.9	0.0	96.4
88.2	-38.0	73.3	-38.0	33.0	-7.6	33.0	0.0	14.7
100.8	-49.0	-475.1	-49.0	-213.8	-9.8	-213.8	0.0	-95.0
113.4	-60.1	-1162.9	-60.1	-523.3	-12.0	-523.3	0.0	-232.6
123.0	-68.6	-1780.8	-68.6	-801.4	-13.7	-801.4	0.0	-356.2
Span 2								
3.0	70.9	-1734.7	70.9	-780.6	14.2	-780.6	0.0	-346.9
16.8	58.7	-839.8	58.7	-377.9	11.7	-377.9	0.0	-168.0
33.6	44.0	23.4	44.0	10.5	8.8	10.5	0.0	4.7
50.4	29.2	638.4	29.2	287.3	5.8	287.3	0.0	127.7
67.2	14.4	1005.4	14.4	452.4	2.9	452.4	0.0	201.1
84.0	-0.3	1123.9	-0.3	505.8	-0.1	505.8	0.0	224.8
100.8	-15.1	994.3	-15.1	447.4	-3.0	447.4	0.0	198.9
117.6	-29.9	616.4	-29.9	277.4	-6.0	277.4	0.0	123.3
134.4	-44.6	-9.5	-44.6	-4.3	-8.9	-4.3	0.0	-1.9
151.2	-59.4	-883.3	-59.4	-397.5	-11.9	-397.5	0.0	-176.7
165.0	-71.5	-1786.8	-71.5	-804.1	-14.3	-804.1	0.0	-357.4
Span 3								
3.0	64.1	-1551.4	64.1	-698.1	12.8	-698.1	0.0	-310.3
11.8	56.3	-1021.5	56.3	-459.7	11.3	-459.7	0.0	-204.3
23.6	46.0	-417.8	46.0	-188.0	9.2	-188.0	0.0	-83.6
35.4	35.6	63.2	35.6	28.4	7.1	28.4	0.0	12.6
47.2	25.2	421.8	25.2	189.8	5.0	189.8	0.0	84.4
59.0	14.8	657.8	14.8	296.0	3.0	296.0	0.0	131.6
70.8	4.4	771.3	4.4	347.1	0.9	347.1	0.0	154.3
82.6	-6.0	762.3	-6.0	343.0	-1.2	343.0	0.0	152.5
94.4	-16.3	630.7	-16.3	283.8	-3.3	283.8	0.0	126.1
106.2	-26.7	376.6	-26.7	169.5	-5.3	169.5	0.0	75.3
116.5	-35.8	54.7	-35.8	24.6	-7.2	24.6	0.0	10.9

3.5.2 Live Loads on Superstructure

3.5.2.1 Cantilever Overhang Loads

Live load distribution on the overhang is determined using an equivalent strip width method. The overhang is designed for Strength I and Extreme Event II only (AASHTO A13.4)

Consider the case of the maximum overhang moment due to the HL-93 design truck (Strength I). Since the overhang is designed on a lineal length basis, it is necessary to determine how much of the overhang is effective at resisting this load. Wheel loads can be placed up to 1 ft from the face of the barrier. The 32-kip axle weight of the HL-93 truck is divided by two to get a 16-kip point load, 1 ft from the barrier. See Figure 3.5-2.

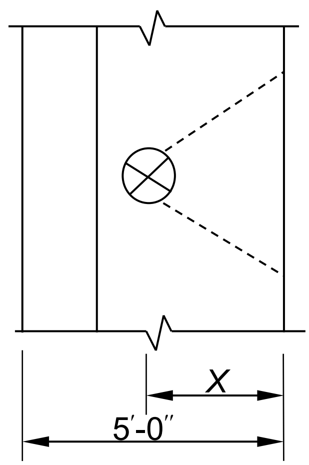


Figure 3.5-2 Overhang Wheel Load

The moment arm for this load is:

X = 5.0 - 1.75 - 1.0 = 2.25 ft

The strip width is therefore:

Strips = 45.0 + 10X = 45 + 10 (2.25) = 67.5 in. (AASHTO Table 4.6.2.1.3-1)

Overhang moment for design is therefore:

$$M_{LL} = \frac{(16)(2.25)}{67.5/12} = 6.4 \text{ kip-ft/ft}$$

Include dynamic load allowance:

M_{LL} = 6.4 (1.33) = 8.51 kip-ft/ft

Include the Strength I load factor of 1.75:

M_{LL} = 8.51 (1.75) = 14.9 kip-ft/ft

3.5.2.2 CIP Box Girder

Live load distribution to each girder in a box girder superstructure is accomplished using empirical formulas to determine the fractional number of live load lanes each girder must be designed to carry. The fractional number of lanes are represented by the girder live load distribution factor, *g*. Empirical formulas are used because a bridge is generally modeled in 2D. Refined methods can be used in lieu of empirical methods whereby a 3D model is used to develop individual girder live load distribution.

These expressions were developed by exponential curve-fitting of force effects from a large bridge database and comparing to results from more refined analyses. Because flexural behavior differs from shear behavior, and force effects in exterior girders differ from those in interior girders, different formulae are provided for each. Due to the torsional rigidity and load sharing capability of a box girder, the box is often considered as a single girder so the formula for interior girders then applies to all girders.

1. **Live Load Distribution for Interior Girder Moment**

Span 1

$S \approx 12$ ft, $L = 126$ ft, $N_c = 4$
(falls within the range of applicability of AASHTO Table 4.6.2.2.2b-1)

One lane loaded case:

$$g_M = \left(1.75 + \frac{S}{3.6}\right)\left(\frac{1}{L}\right)^{0.35}\left(\frac{1}{N_c}\right)^{0.45} = \left(1.75 + \frac{12}{3.6}\right)\left(\frac{1}{126}\right)^{0.35}\left(\frac{1}{4}\right)^{0.45} = 0.501$$

Fatigue limit state:
(multiple presence factor of 1.2 must be removed from distribution factor)

$$g_M = \frac{0.501}{1.2} = 0.418$$

Two or more lanes loaded case:

$$g_M = \left(\frac{13}{N_c}\right)^{0.3}\left(\frac{S}{5.8}\right)\left(\frac{1}{L}\right)^{0.25} = \left(\frac{13}{4}\right)^{0.3}\left(\frac{12}{5.8}\right)\left(\frac{1}{126}\right)^{0.25} = 0.879$$

The distribution factors for all spans are listed in Table 3.5-2.

Table 3.5-2 Girder Live Load Distribution for Moment, g_M

Span	Fatigue Limit State*	All other Limit States
1	0.418	0.879
2	0.378	0.818
3	0.428	0.894

**m* of 1.2 has been divided out for the Fatigue Limit State

For a whole bridge design method (such as is used in CTBridge), multiply by the number of girders. For span 1, $(g_M)_{total}$ = 4.395.

2. **Live Load Distribution for Interior Girder Shear**

Span 1

Depth of beam, d = 81 in.
(falls within the range of applicability of AASHTO Table 4.6.2.2.3a-1)

One lane loaded case:

$$g_S = \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{d}{12.0L}\right)^{0.1} = \left(\frac{12.0}{9.5}\right)^{0.6}\left(\frac{81}{12\times 126.0}\right)^{0.1} = 0.859$$

Fatigue limit state:
(multiple presence factor of 1.2 must be removed from distribution factor)

$$g_S = \frac{0.859}{1.2} = 0.716$$

Two or more lanes loaded case:

$$g_S = \left(\frac{S}{7.3}\right)^{0.9}\left(\frac{d}{12.0L}\right)^{0.1} = \left(\frac{12.0}{7.3}\right)^{0.9}\left(\frac{81}{12\times 126.0}\right)^{0.1} = 1.167$$

The distribution factors for all spans are listed in Table 3.5-3.

Table 3.5-3 Girder Live Load Distribution for Shear

Span	Fatigue Limit State*	All other Limit States
1	0.716	1.167
2	0.695	1.134
3	0.720	1.175

*m of 1.2 has been divided out for the Fatigue Limit State

The total for the whole bridge for span 1 would be: $(g_S)_{total} = 5.835$

3.5.2.3 Precast I, Bulb-Tee, or Steel Plate Girder

In general, the live load distribution at the exterior girder is not the same as that for the interior girder. However, in no instance should the exterior girder be designed for fewer live load lanes than the interior girder, in case of future widening.

A precast I-girder bridge is shown in Figure 3.5-3. Calculations for live load distribution factors for interior and exterior girders follow.

Given:

$S = 9.67$ ft; $L = 110$ ft; $t_s = 8$ in.;
K_g = longitudinal stiffness parameter (in.^{4}); $N_b = 6$

Calculation of the longitudinal stiffness parameter, K_g:

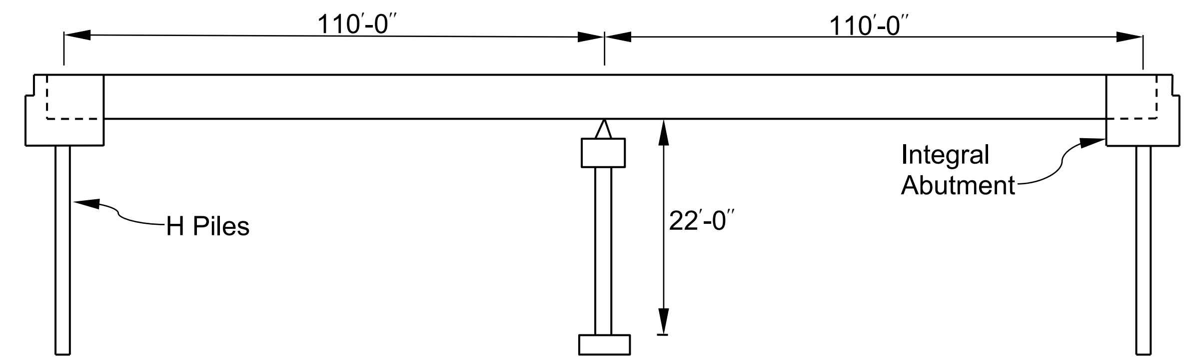
$K_g = n\,(I + Ae_g^2)$ (AASHTO 4.6.2.2.1-1)

$$n = \frac{E_B}{E_D} = \frac{4696}{3834} = 1.225$$ (AASHTO 4.6.2.2.1-2)

I = 733,320 in.^{4}; A = 1,085 in.^{2} beam only

e_g = vertical distance from c.g. beam to c.g. deck = 39.62 in.

$K_g = 1.225\,(733{,}320 + 1{,}085 \times 39.62^2) = 2{,}984{,}704$ in.^{4}



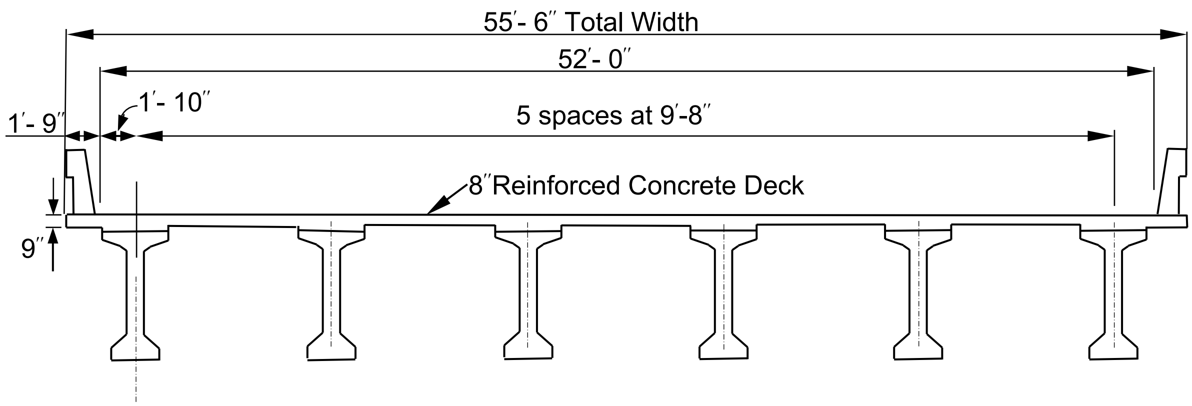


Figure 3.5-3 Precast Bulb-Tee Bridge to be Used for Distribution Calculations

1. **Live Load Distribution for Interior Girder Moment**

One lane loaded case:

$$g_M = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

$$= 0.06 + \left(\frac{9.67}{14}\right)^{0.4}\left(\frac{9.67}{110}\right)^{0.3}\left(\frac{2{,}984{,}704}{(12)(110)(8)^3}\right)^{0.1} = 0.542$$

Note: The term $\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ could have been taken as 1.09 for preliminary design (Table 4.6.2.2.1-3) but was not used here.

Fatigue limit state:
(multiple presence factor of 1.2 must be removed from distribution factor)

$$g_M = \frac{0.542}{1.2} = 0.452$$

Two or more lanes loaded case:

$$g_M = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

$$= 0.075 + \left(\frac{9.67}{9.5}\right)^{0.6}\left(\frac{9.67}{110}\right)^{0.2}\left(\frac{2{,}984{,}704}{(12)(110)(8)^3}\right)^{0.1} = 0.796$$

2. **Live Load Distribution for Exterior Girder Moment**

One lane loaded case:

Use the lever rule. The lever rule assumes the deck is a simply supported member between girders. Live loads must be placed to maximize the reaction of one lane of live load (Figure 3.5-4).

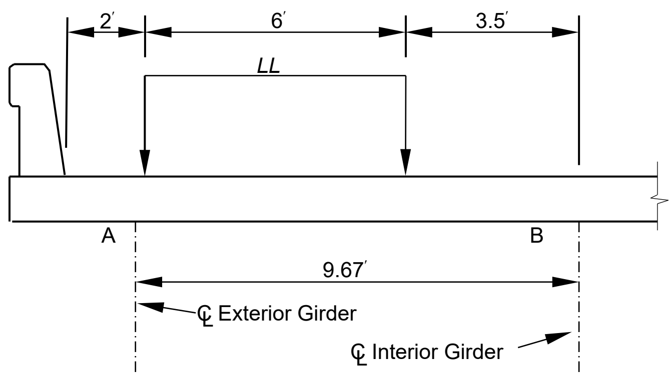


Figure 3.5-4 Lever Rule Example for Exterior Girder Distribution Factor

$$\sum M_B = 0$$

$$\frac{LL}{2}(3.5 + 9.5) = R_A \times 9.67$$

$$R_A = 0.672 \text{ lanes}$$

Therefore, for exterior girder moment, g_M = 0.672 lanes. Use for the Fatigue Limit State. For other limit states, g_M = 1.2 (0.672) = 0.806 lanes.

Two or more lanes loaded case:

$$g_M = e(g_M)_{interior}$$

$$e = 0.77 + \frac{d_e}{9.1}$$

$$d_e = 1.83 \text{ ft}$$

$$e = 0.77 + \frac{1.83}{9.1} = 0.971$$

$$g_M = 0.971(0.796) = 0.773 \text{ lanes}$$

It is seen that the one lane loaded case controls for all limit states.

3. **Live Load Distribution for Interior Girder Shear**

One lane loaded case:

$$g_S = 0.36 + \frac{S}{25.0} = 0.36 + \frac{9.67}{25.0} = 0.747$$

Fatigue limit state:
(multiple presence factor of 1.2 must be removed from distribution factor)

$$g_S = \frac{0.747}{1.2} = 0.623$$

Two or more lanes loaded case:

$$g_S = 0.2 + \frac{S}{12.0} - \left(\frac{S}{35}\right)^2 = 0.2 + \frac{9.67}{12.0} - \left(\frac{9.67}{35}\right)^2 = 0.929$$

4. **Live Load Distribution for Exterior Girder Shear**

One lane loaded case:

This case requires the lever rule once again. The result is exactly the same for moment as for shear. Therefore (g_S)exterior = 0.672 for the Fatigue Limit State and (g_S)exterior = 0.806 for all other limit states.

Two or more lanes loaded case:

$$(g_S)_{exterior} = e(g_s)_{interior}$$
$$e = 0.6 + \frac{d_e}{10} = 0.6 + \frac{1.83}{10} = 0.783$$
$$g_S = 0.783(0.929) = 0.727$$

However, because the exterior girder cannot be designed for fewer live load lanes than the interior girders, use (g_S)exterior = 0.929 for all other limit states.

The complete list of distribution factors for this bridge is shown in Tables 3.5-4 and 3.5-5.

Table 3.5-4 Girder Live Load Distribution for Moment

Girder	Fatigue Limit State	All other Limit States
Interior	0.452	0.796
Exterior	0.672	0.806

Table 3.5-5 Girder Live Load Distribution for Shear

Girder	Fatigue Limit State	All other Limit States
Interior	0.623	0.929
Exterior	0.672	0.929

3.5.3 Live Loads on Substructure

Substructure elements include the bent cap beam, columns, footings, and piles. To calculate the force effects on these elements a “transverse” analysis is performed.

In order to properly load the bent with live load, results from the longitudinal frame analysis are used. In this section, live load forces affecting column design are discussed.

For column design there are 3 cases to consider:

1) $(M_T)_{max} + (M_L)_{assoc} + P_{assoc}$
2) $(M_L)_{max} + (M_T)_{assoc} + P_{assoc}$
3) $P_{max} + (M_L)_{assoc} + (M_T)_{assoc}$

Each of these three cases applies to both the Design Vehicle live load and the Permit load. In the Permit load case, up to two permit trucks are placed in order to produce maximum force effects. These loads are then used in a column design program such as Caltrans’ WINYIELD (2007).

Example

Consider the following bridge with a single column bent as shown in Figure 3.5-5 and 3.5-6 to calculate the force effects at the top of the column:

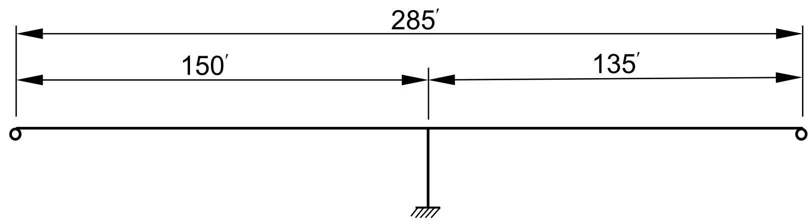


Figure 3.5-5 Example Bridge Elevation for Substructure Calculations

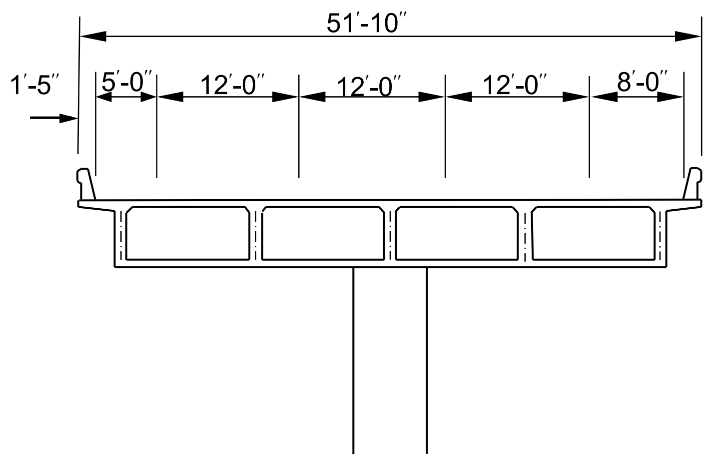


Figure 3.5-6 Example Bridge Typical Section for Substructure Calculations

Live load effects from a longitudinal frame analysis are tabulated:

Table 3.5-6 CTBridge Live Load Effects

Top of Column Live Load Forces (one lane + *IM*)			
Vehicle class	Case	P (kips)	M_L(kip-ft)
Design Truck+*IM*	P_{max}	154	66
	$(M_L)_{max}$	100	465
Design Lane	P_{max}	103	39
	$(M_L)_{max}$	61	228
Permit Truck+*IM*	P_{max}	455	240
	$(M_L)_{max}$	333	1,319

1. **HL-93 Design Vehicle**

Maximum Transverse Moment $(M_T)_{max}$ Case

To obtain the top of column moments in the transverse direction, the axial forces due to one lane of live load listed above are placed on the bent to produce maximum effects.

By inspection, placing two design vehicle lanes on one side of the bent will produce maximum transverse moments in the column (Figure 3.5-7). When not obvious, cases with one, two, three, and four vehicles should be evaluated. Note that wheel lines must be placed 2 ft from the face of the barrier. If the bridge has a sidewalk that may be removed in the future, the vehicle live loads shall be applied 2 ft from the face of the future barrier. Longitudinally, the vehicles are located over the bent thus maximizing M_T.

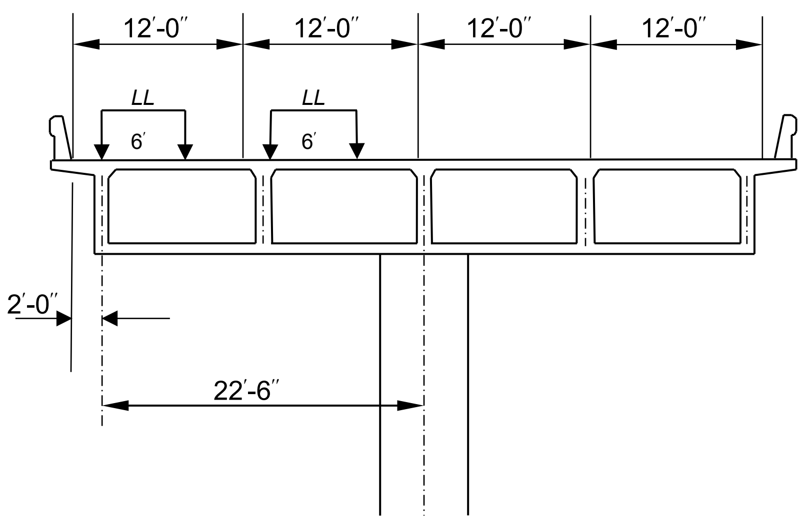


Figure 3.5-7 Vehicle Position for $(M_T)_{max}$

LL = 154 + 103 = 257 kips

Multiple presence factor, m = 1.0 for two lanes.

$$(M_T)_{\max} = (1.0)\frac{257}{2}(22.5 + 16.5 + 10.5 + 4.5) = 6{,}939 \text{ kip-ft}$$

$(M_L)_{associated}$ =(1.0)(66 + 39) × 2 = 210 kip-ft

$P_{associated}$ = (1.0) 257× 2 = 514 kips

Maximum Axial Force P_{max} Case

To maximize axial forces on the column, place as many lanes as can fit on the bridge. In this case four lanes are required:

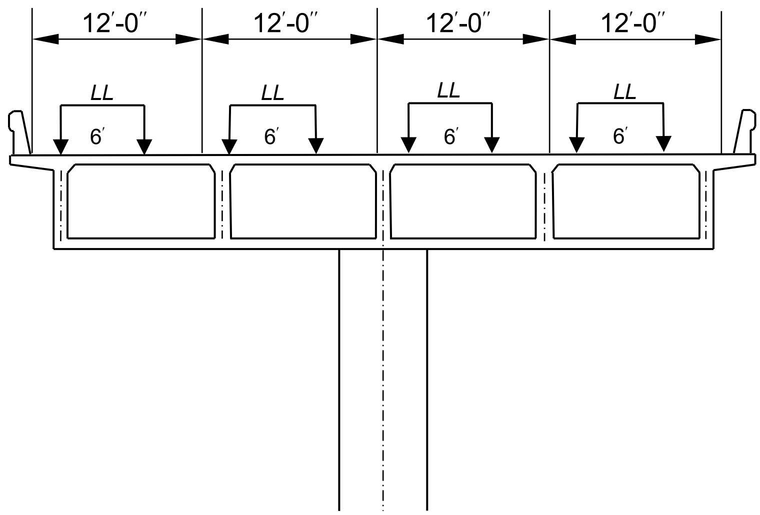


Figure 3.5-8 Vehicle Position for $(P)_{max}$

Multiple presence, m = 0.65 for four-lanes loaded.

$$(M_T)_{associated} = (0.65)\left(\frac{257}{2}\right)(22.5 + 16.5 + 10.5 + 4.5 - 1.5 - 7.5 - 13.5 - 19.5)$$
$$= 1{,}002 \text{ kip-ft}$$

$(M_L)_{associated}$ = (0.65)(66+39)(4) = 273 kip-ft

P_{max} = (0.65)(257)(4) = 668 kips

Maximum Longitudinal Moment $(M_L)_{max}$ Case

Load the bridge with as many lanes as possible but this time, the vehicles are located longitudinally somewhere within the span:

$$(M_T)_{associated} = (0.65)\left(\frac{100 + 61}{2}\right)(22.5 + 16.5 + 10.5 + 4.5 - 1.5 - 7.5 - 13.5 - 19.5)$$
$$= 628 \text{ kip-ft}$$

$(M_L)_{max}$ = (0.65)(465 + 228)(4) = 1,802 kip-ft

$P_{associated}$ = (0.65)(100 + 61)(4) = 419 kips

2. **Permit Vehicle**

Next calculate the live load forces at the top of the column due to the Permit vehicle. Note: Multiple presence, m = 1.0 when using either one or two lanes (Article CA 3.6.1.8.2).

$(M_T)_{max}$ Case

Two lanes of Permit load are placed on one side of the bent cap as shown in Figure 3.5-7.

$$\left(M_T\right)_{\max} = \frac{455}{2}\left(22.5 + 16.5 + 10.5 + 4.5\right) = 12{,}285 \text{ kip-ft}$$

$(M_L)_{associated}$ = 240 (2) = 480 kip-ft

$P_{associated}$ = 455(2) = 910 kips

P_{max} Case

Again, to maximize the axial force, the trucks are located right over the bent and a maximum of 2 lanes of Permit vehicles are placed on the bridge. This results in the same configuration as in the (M_T)max case. Therefore, the results are the same.

$(M_L)_{max}$ Case

$$\left(M_T\right)_{associated} = \frac{333}{2}\left(22.5 + 16.5 + 10.5 + 4.5\right) = 8{,}991 \text{ kip-ft}$$

$(M_L)_{max}$ = 1,319(2) = 2,638 kip-ft

$P_{associated}$ = 333(2) = 666 kips

Summary of the live load forces at the top of column for all live load cases are shown in Tables 3.5-7 and 3.5-8.

Table 3.5-7 Summary of Design Vehicle Forces for Column Design

Load	$(M_T)_{max}$ Case (kip-ft)	$(M_L)_{max}$ Case (kip-ft)	P_{max} Case (kips)
M_T	6,939	628	1,002
M_L	210	1,802	273
P	514	419	668

Table 3.5-8 Summary of Permit Vehicle Forces for Column Design

Load	$(M_T)_{max}$ Case (kip-ft)	$(M_L)_{max}$ Case (kip-ft)	P_{max} Case (kips)
M_T	12,285	8,991	12,285
M_L	480	2,638	480
P	910	666	910

3.5.4 Skew Modification of Shear Force in Superstructures

To illustrate the effect of skew modification, the example bridge shown in Figure 3.5-9 is used. Because load takes the shortest pathway to a support, the girders at the obtuse corners of the bridge will carry more load. A 2-D model cannot capture the effects of skewed supports. Therefore, shear forces must be amplified according to Table 3.5-9 and Table 3.5-10.

Table 3.5-9 Skew Correction of Shear Forces for Live Load Distribution Factors

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Correction Factor	Range of Applicability
Cast-in-place Concrete Multicell Box	d	For exterior girder: $1 + \theta/50 \leq 1.6$ For first interior girder: $1 + \theta/300 \leq 1.1$	$0 < \theta \leq 60^{o}$ $6.0 < S \leq 13.0$ $20 \leq L \leq 240$ $35 \leq d \leq 110$ $N_c \geq 3$

Table 3.5-10—Skew Correction Factors of Shear Forces for *DC*, *DW*, *PS*, and *TU*

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Correction Factor	Range of Applicability
Cast-in-place Concrete Multicell Box	d	For exterior girder: $1 + \theta/25 \leq 2.2$ For first interior girder: $1 + \theta/150 \leq 1.2$	$0^{o} < \theta < 60^{o}$ $6.0 < S < 13.0$ $20 < L < 240$ $35 < d < 110$ $N_c > 3$

The example bridge has a 20 degree skew. Skew correction Factors for live load are as follows:

Exterior Girder: $1.0+\frac{20}{50}=1.4\leq 1.6$

First Interior Girder: $1.0+\frac{20}{300}=1.067\leq 1.1$

Permanent and *TU* correction factors are as follows:

Exterior Girder: $1.0+\frac{\theta}{25}=1.8\leq 2.2$

First Interior Girder: $1.0+\frac{\theta}{150}=1.133\leq 1.2$

To illustrate the application of these correction factors, apply them to live load (*LL*) shear forces on the northern most exterior girder. Different correction factors are to be applied to *DC*, *DW*, *PS* and *TU*. Figure 3.5-9 shows the girder layout and Table 3.5-11 lists *LL* and *DC* correction factors for the example bridge.

Similar factors will apply to the south side girders but will not include the sound wall in the DC loads.

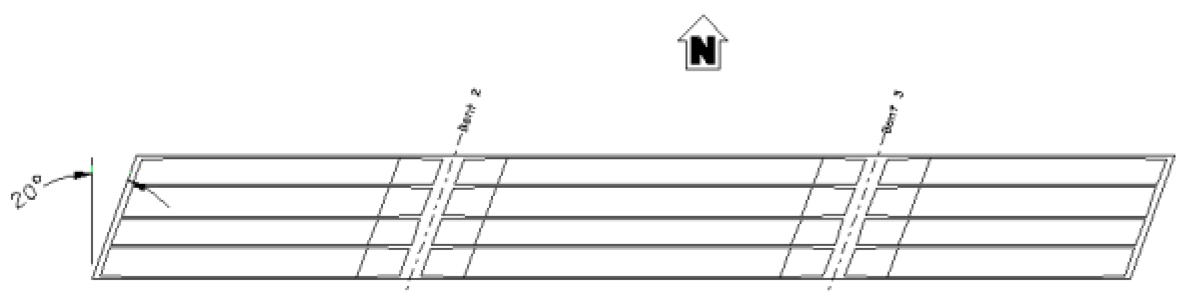


Figure 3.5-9 Girder Layout for Skew Modification Example

Table 3.5-11 Example Bridge DC Skew Correction (Northern Most Exterior Girder)

Span	Tenth Point	V_{DC} (kips)	(V_{DC}) per girder (kips)	Correction Factor	Corrected (V_{DC}) (kips)	V_{LL} (kips)	(V_{LL}) per girder (kips)	Correction Factor	Corrected (V_{LL}) (kips)
1	0.0	735	147	1.80	265	696	139	1.40	195
	0.1	550	110	1.64	180	596	119	1.32	157
	0.2	340	68	1.48	101	489	98	1.24	122
	0.3	130	26	1.32	34	391	78	1.16	90
	0.4	-79	-16	1.16	-19	301	60	1.08	65
	0.5	-289	-58	1.00	-58	211	42	1.00	42
	0.6	-499	-100	1.00	-100	153	31	1.00	31
	0.7	-708	-142	1.00	-142	95	19	1.00	19
	0.8	-923	-185	1.00	-185	50	10	1.00	10
	0.9	-1141	-228	1.00	-228	21	4	1.00	4
	1.0	-1308	-262	1.00	-262	6	1	1.00	1
2	0.0	1381	276	1.80	497	831	166	1.40	232
	0.1	1144	229	1.64	376	745	149	1.32	197
	0.2	856	171	1.48	253	638	128	1.24	159
	0.3	572	114	1.32	150	531	106	1.16	123
	0.4	293	59	1.16	68	426	85	1.08	92
	0.5	13	3	1.00	3	317	63	1.00	63
	0.6	-266	-53	1.00	-53	238	48	1.00	48
	0.7	-546	-109	1.00	-109	160	32	1.00	32
	0.8	-829	-166	1.00	-166	95	19	1.00	19
	0.9	-1118	-224	1.00	-224	74	15	1.00	15
	1.0	-1355	-271	1.00	-271	72	14	1.00	14
3	0.0	1241	248	1.80	446	806	161	1.40	225
	0.1	1088	218	1.64	358	747	149	1.32	197
	0.2	882	176	1.48	260	663	133	1.24	165
	0.3	682	136	1.32	180	575	115	1.16	133
	0.4	485	97	1.16	113	486	97	1.08	105
	0.5	289	58	1.00	58	382	76	1.00	76
	0.6	92	18	1.00	18	307	61	1.00	61
	0.7	-104	-21	1.00	-21	219	44	1.00	44
	0.8	-301	-60	1.00	-60	140	28	1.00	28
	0.9	-497	-99	1.00	-99	97	19	1.00	19
	1.0	-668	-134	1.00	-134	94	19	1.00	19

3.6 LOAD FACTORS AND COMBINATIONS

The Limit States of AASHTO-CA BDS-8 Section 3 require combining the individual loads with specific load factors to achieve design objectives. The example bridge shown in Figure 3.3-1 is used to determine the maximum positive moments in the superstructure by factoring all relevant load effects in the appropriate limit states.

Tables 3.6-1, 3.6-2, and 3.6-3 summarize load factors used for the example bridge Span 2.

For Span 2, unfactored midspan positive moments are as follows:

M_{DC} = 21,318 kip-ft

M_{DW} = 2,467 kip-ft

$M_{HL\text{-}93}$ = 13,206 kip-ft

M_{PERMIT} = 26,029 kip-ft

M_{PS} = 7,118 kip-ft

Factored positive moments are calculated as follows:

Strength I:

$$M = 1.25(21{,}318) + 1.5(2{,}467) + 1.0(7{,}118) + 1.75(13{,}206) = 60{,}577 \text{ kip-ft}$$

Strength II:

$$M = 1.25(21{,}318) + 1.5(2{,}467) + 1.0(7{,}118) + 1.35(26{,}029) = 72{,}605 \text{ kip-ft}$$

Therefore, the Strength II Limit State controls for positive moment at this location.

Table 3.6-1 Load Combinations for Span 2 +*M*

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	$LL_{HL\text{-}93}$ *IM* *CE* *BR* *PL* *LS*	$\underline{LL_{Permit}}$ *<u>IM</u>* *<u>CE</u>*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	*EQ* *BL* *IC* *CT* *CV* (use only one)
STRENGTH I	γ_p	1.75		1.0	-	-	1.0	0.50/1.20	γ_{TG}	γ_{SE}	-
STRENGTH II	γ_p	-	<u>1.35</u>	1.0	-	-	1.0	0.50/1.20	γ_{TG}	γ_{SE}	-

Table 3.6-2 Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments *DC*: Strength IV, only		1.25 1.50	0.90 0.90
DD: Downdrag	Piles, α Tomlison Method Piles, λ Method Drilled Shafts, O'Neill and Reese (1999) Method	1.4 1.05 1.25	0.25 0.30 0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure • Active • At-Rest • *AEP* for Anchored Walls		 1.50 1.35 1.35	 0.90 0.90 N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure • Overall Stability • Retaining Walls and Abutments • Rigid Buried Structure • Rigid Frames • Flexible Buried Structures ○ Metal Box Culverts and Structural Plate Culverts with Deep Corrugations ○ Thermoplastic Culverts ○ All Others		 1.00 1.35 1.30 1.35 1.5 1.3 1.95	 N/A 1.00 0.90 0.90 0.9 0.9 0.9
ES: Earth Surcharge		1.50	0.75

Table 3.6-3 Load Factors for Permanent Loads Due to Superimposed Deformations, γ_p

Bridge Component	*PS*	*CR,SH*
Superstructures–Segmental Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)	1.0	See γ_p for *DC*, Table 3.6-2
Concrete Superstructures–non-segmental	1.0	1.0
Substructures supporting non-segmental Superstructures • using I_g • using $I_{effective}$	 0.5 1.0	 0.5 1.0
Steel Substructures	1.0	1.0

NOTATION

Load Designations

BL	=	blast loading
BR	=	vehicular braking force
CE	=	vehicular centrifugal force
CR	=	force effects due to creep
CT	=	vehicular collision force
CV	=	vessel collision force
DC	=	dead load of components
DD	=	downdrag
DW	=	dead load of wearing surfaces and utilities
EH	=	horizontal earth pressure load
EL	=	miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
EQ	=	earthquake
ES	=	earth surcharge load
EV	=	vertical pressure from dead load of earth fill
FR	=	friction
IC	=	ice load
IM	=	vehicular dynamic load allowance
LL	=	vehicular live load
LS	=	live load surcharge
PL	=	pedestrian live load
PS	=	secondary forces from post-tensioning for strength limit states total prestress forces for service limit states
SE	=	force effects due to settlement
SH	=	force effects due to shrinkage
TG	=	force effects due to temperature gradient
TU	=	force effects due to uniform temperature
WA	=	water load and stream pressure
WL	=	wind on live load
WS	=	wind load on structure

General Symbols

A	=	area of section (ft^2)
AX	=	axial force (kip)
C	=	centrifugal force factor
C_D	=	drag coefficient
C_L	=	lateral drag coefficient
d	=	structure depth (in.); depth of beam (in.)
d_e	=	distance from centerline of exterior girder web to face of barrier (ft)
D	=	distance from first member of frame (ft)
e	=	girder *LL* distribution factor multiplier for exterior girders
e_g	=	vertical distance from c.g. beam to c.g. deck (in.)
E	=	modulus of elasticity (ksi)
E_B	=	modulus of elasticity of beam material (ksi)
E_D	=	modulus of elasticity of deck material (ksi)
f	=	*CE* fatigue factor
F_t	=	transverse barrier collision force (kip)
F_L	=	longitudinal barrier collision force (kip)
g	=	gravitational acceleration (32.2 ft/sec); live load distribution factor representing the number of design lanes
g_M	=	girder LL distribution factor for moment
g_S	=	girder LL distribution factor for shear
G	=	gust effect factor
h_{eq}	=	equivalent height of soil for vehicular load (ft)
H	=	height of element (ft)
I	=	moment of inertia (ft^4)
$I_{effective}$	=	effective moment of inertia (ft^4)
I_g	=	gross moment of inertia (ft^4)
k	=	coefficient of lateral earth pressure
k_a	=	active earth pressure coefficient
k_s	=	earth pressure coefficient due to surcharge
K_g	=	longitudinal stiffness parameter (in.^{4})
K_z	=	pressure exposure and elevation coefficient

L	=	span length (ft)
m	=	multiple presence factor
M_B	=	moment about point B (k-ft)
M_{CT}	=	vehicular collision moment on barrier (kip-ft)
M_L	=	longitudinal moment on column (kip-ft)
M_{LL}	=	moment due to live load (kip-ft)
M_{DC}	=	moment due to dead load (kip-ft)
M_{DW}	=	moment due to dead load wearing surface (kip-ft)
$M_{HL\text{-}93}$	=	moment due to design vehicle (kip-ft)
M_{PERMIT}	=	moment due to permit vehicle (kip-ft)
M_{PS}	=	moment due to secondary pre-stress forces (kip-ft)
M_T	=	transverse moment on column (kip-ft)
MY	=	transverse bending (kip-ft)
MZ	=	longitudinal bending (kip-ft)
n	=	modular ratio; number of girders
N_b	=	number of beams
N_c	=	number of cells in the box girder section
p	=	stream force pressure (ksf), pressure against wall (ksf)
P	=	axial load on column (kip), design wind pressure (ksf)
q_s	=	uniform surcharge applied to upper surface of the active earth wedge (ksf)
r	=	column radius (ft)
R	=	radius of curvature of traffic lane (ft)
R_A	=	reaction at point A (kip)
S	=	center to center girder spacing (ft)
t_s	=	top slab thickness (in.)
$T_{MaxDesign}$	=	maximum design temperature used for thermal movement effects (°F)
$T_{MinDesign}$	=	minimum design temperature used for thermal movement effects (°F)
TX	=	torsion (kip-ft)
v	=	highway design speed (ft/sec)
V	=	design velocity of water (ft/sec); design wind speed (mph)

V_{DC}	=	shear due to dead load (kip)
V_{LL}	=	shear due to live load (kip)
VY	=	transverse shear (kip)
VZ	=	longitudinal shear (kip)
w	=	uniform load (kip/ft)
$w_{barrier}$	=	weight of barrier (kip/ft)
w_c	=	unit weight of concrete (kcf)
w_{chain}	=	weight of chain link railing (kip/ft)
X	=	moment arm for overhang load (ft)
z	=	depth to point below ground surface (ft)
Z	=	height of structure at which wind loads are being calculated (ft)
α	=	coefficient of thermal expansion
Δ	=	column deflection
Δ_{p}	=	earth surcharge load
Δ_T	=	design thermal movement range (in)
γ_{p}	=	load factor for permanent loading
γ_{s}	=	density of soil (pcf)
γ_{SE}	=	load factor for settlement
γ_{TG}	=	load factor for temperature gradient
θ	=	skew angle (degrees)

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