Bridge Design - Loads and Load Combinations

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Division of Engineering Services

BRIDGE DESIGN PRACTICE

Caltrans

State of California
Department of Transportation

CHAPTER 3
LOADS AND LOAD COMBINATIONS

TABLE OF CONTENTS

CHAPTER 3
LOADS AND LOAD COMBINATIONS

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*, 6th Edition (AASHTO, 2012) and the *California Amendments to the AASHTO LRFD Bridge Design Specifications* (CA) (Caltrans, 2014). 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.

3.1.1 Load Path

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.

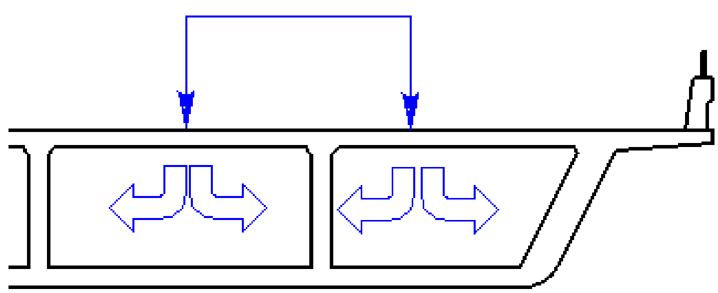


Figure 3.1-1 Truck Load Path from Deck Slab to Girders

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, see 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).

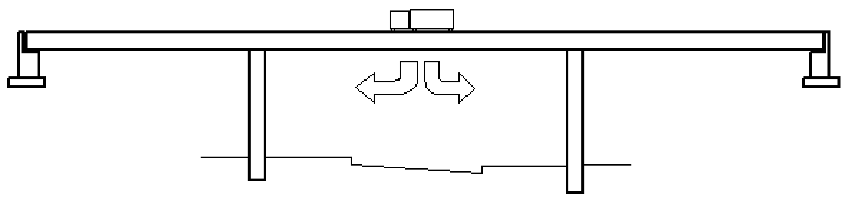


Figure 3.1-2 Truck Load Path from Girders to Bents

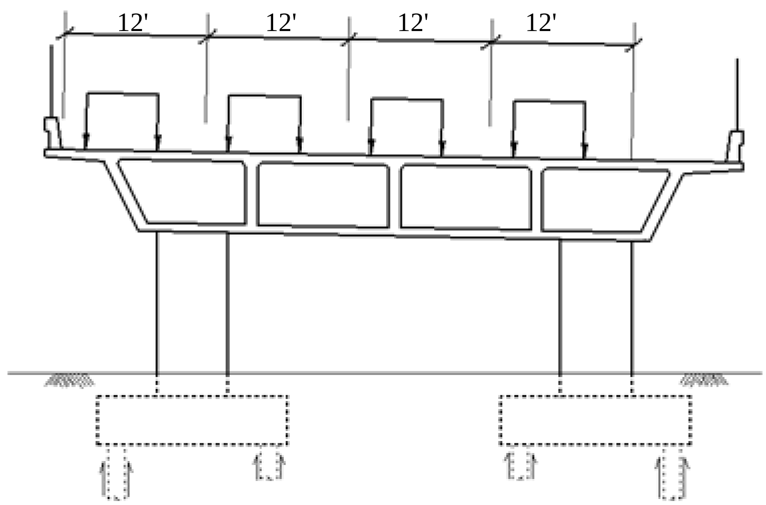


Figure 3.1-3 Truck Load on Bent Cap

When the girder load reaches the bent caps or abutments, it once again changes direction from longitudinal to transverse. 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 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 (*PS*), and 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 are varying over a short time interval. A transient load is any load that will not remain on the bridge indefinitely. This includes vehicular live loads (*LL*) and their secondary effects including dynamic load allowance (*IM*), braking force (*BR*), centrifugal force (*CE*), and live load surcharge (*LS*). Additionally, there are pedestrian live loads (*PL*), force effects due to uniform temperature (*TU*), and temperature gradient (*TG*), force effects due to settlement (*SE*), water loads and stream pressure (*WA*), wind loads on structure (*WS*), wind on live load (*WL*), friction forces (*FR*), ice loads (*IC*), vehicular collision forces (*CT*), vessel collision forces (*CV*), and 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.

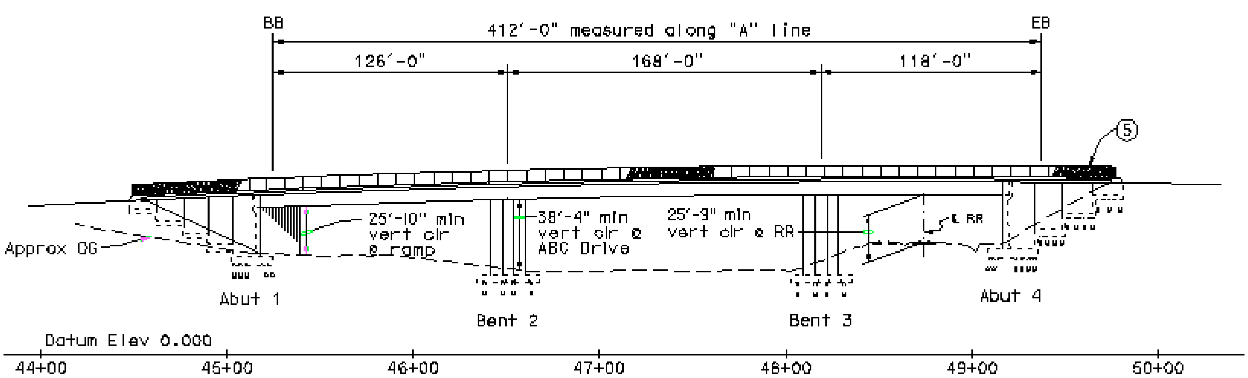


Figure 3.3-1 Elevation View of Example Bridge

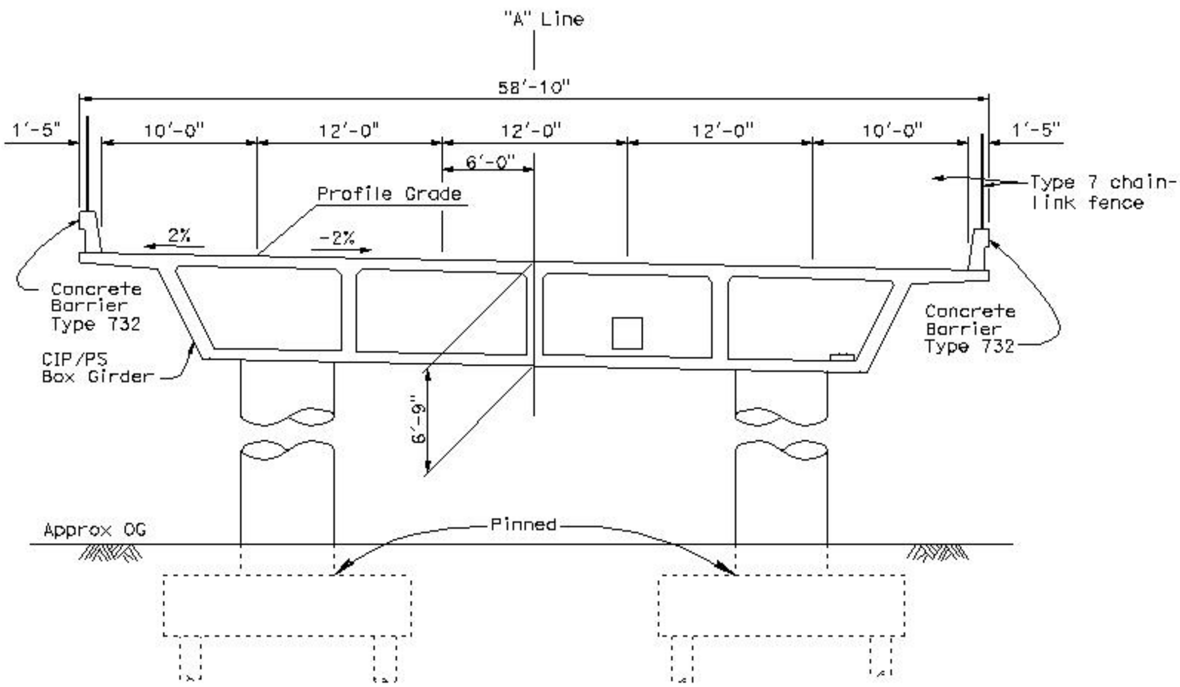


Figure 3.3-2 Typical Section View of Example Bridge

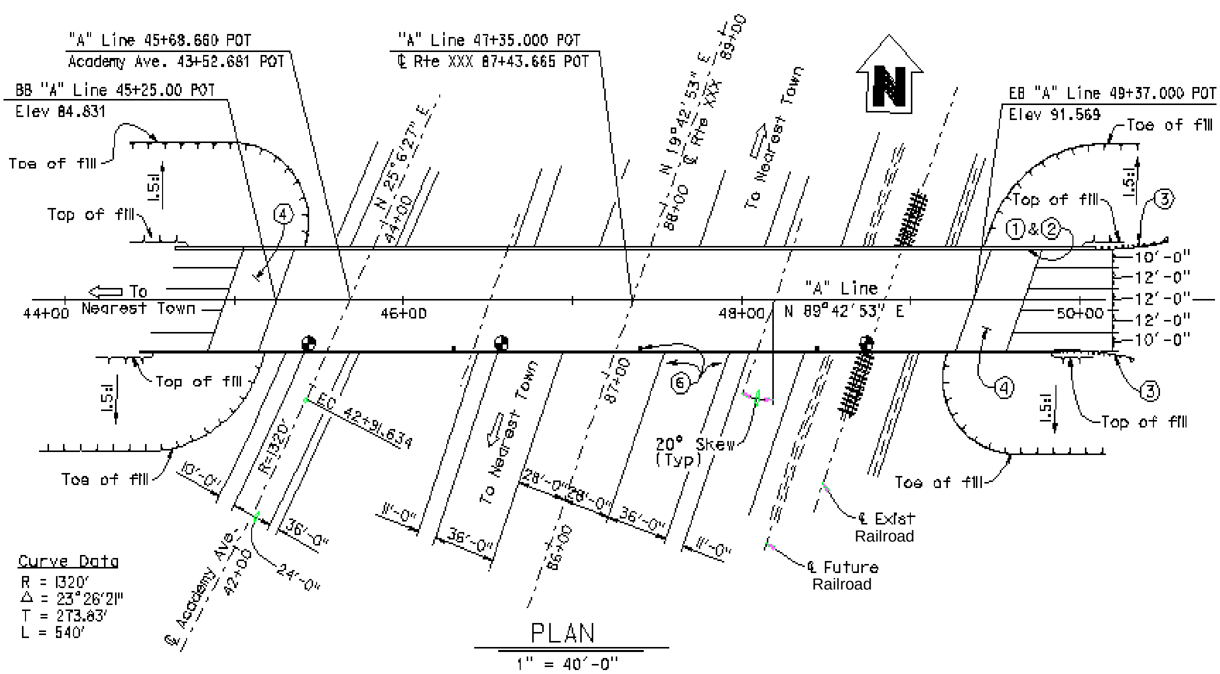


Figure 3.3-3 Plan View of Example Bridge

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.

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 shall not be included again in a transverse analysis of the bent.

Normal weight concrete is assigned a density of 150 pcf which includes the weight of bar reinforcing steel and lost formwork in cast-in-place (CIP) box girder superstructures. Adjustments need not be made for the presence of prestressing tendons, soffit access openings, vents and other small openings for utilities.

For this example bridge, the weight of a Type 732 barrier and Type 7 chain link fence is modeled as a line load in a longitudinal frame analysis as follows:

Type 732 barrier:

$A = 2.73\ \text{ft}^2$

$w_c = 0.15\ \text{kcf}$ (AASHTO C5.4.2.4)

$w_{barrier} = Aw_c = 2.73(0.15) = 0.41\text{kip/ft}$

Type 7 chain link fence:

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

Total weight of two barriers $w = (0.41 + 0.02)(2) = 0.86\ \text{kip/ft}$

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

Future wearing surfaces are generally asphalt concrete. New bridges require designing for a thickness of 3 in., which results in a load of 35 psf as specified in MTD 15-17 (Caltrans, 1988). Therefore, the weight of the wearing surface to be considered is:

Uniform weight: 35 psf

Width of bridge with AC: $58.83 - 2(1.42) = 55.99$ ft

Line Load: $w = 55.99(0.035) = 1.96\ \text{kip/ft}$

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.

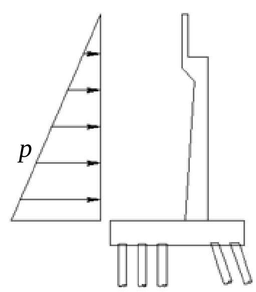
The geotechnical engineer is responsible for determining the additional load due to *DD* and incorporating that load with all other loads provided in the CA, Section 10 (Caltrans, 2014).

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: $k_a = 0.3$, $\gamma_s = 120$ pcf and abutment height, $H = 30$ ft.



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

Figure 3.3-4 Abutment 1 with *EH* Load

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

where z = depth below ground surface

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

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

Total Force = $16.2\,(62.6) = 1{,}014 \text{ kips}$

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

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

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 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.

$$\Delta_p = k_s q_s \qquad \text{(AASHTO 3.11.6.1-1)}$$

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

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

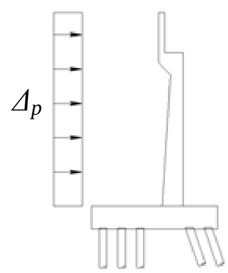


Figure 3.3-5 Abutment 1 with *ES* Load

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 7 and 8 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 6 to 9 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	-7.6	70.7	0.0	0.0	0.0	103.1
12.60	-6.7	69.4	0.0	0.0	0.0	819.4
25.20	-5.7	67.7	0.0	0.0	0.0	1519.0
37.80	-5.2	67.7	0.0	0.0	0.0	2246.6
50.40	-4.8	67.0	0.0	0.0	0.0	3007.0
63.00	-4.3	66.8	0.0	0.0	0.0	3638.7
75.60	-4.1	66.9	0.0	0.0	0.0	4461.8
88.20	-3.9	66.9	0.0	0.0	0.0	5157.2
100.80	2.8	66.4	0.0	0.0	0.0	6364.6
113.40	1.9	21.9	0.0	0.0	0.0	6842.7
123.00	10.0	-9.5	0.0	0.0	0.0	6895.4
***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	31.7	1.9	0.0	-0.0	0.0	0.0
11.00	31.7	1.9	0.0	-0.0	0.0	20.4
22.00	31.7	1.9	0.0	-0.0	0.0	40.8
33.00	31.7	1.9	0.0	-0.0	0.0	61.2
44.00	31.7	1.9	0.0	-0.0	0.0	81.7
***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	31.7	1.9	0.0	-0.0	0.0	0.0
11.00	31.7	1.9	0.0	-0.0	0.0	20.4
22.00	31.7	1.9	0.0	-0.0	0.0	40.8
33.00	31.7	1.9	0.0	-0.0	0.0	61.2
44.00	31.7	1.9	0.0	-0.0	0.0	81.7

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

There are instances when a bridge design requires force to be "locked" into the structure in order to be built. 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, *EL* forces do not need to be considered.

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 shall always be 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*) and generally controls the majority of column designs in California. Refer to Volume III of this practice manual for seismic design.

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 vehicles – 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 allowed on the bridge at a time, except for Cases 3 and 4 (Figure 3.4-2). Trucks shall be placed transversely in as many lanes as practical. Multiple presence factors shall be used to account for the improbability of multiple fully loaded lanes side by side.

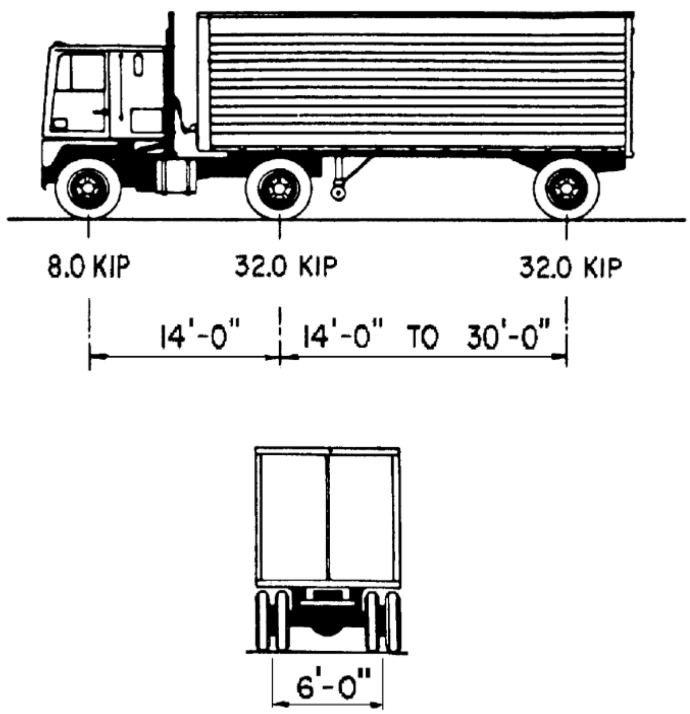
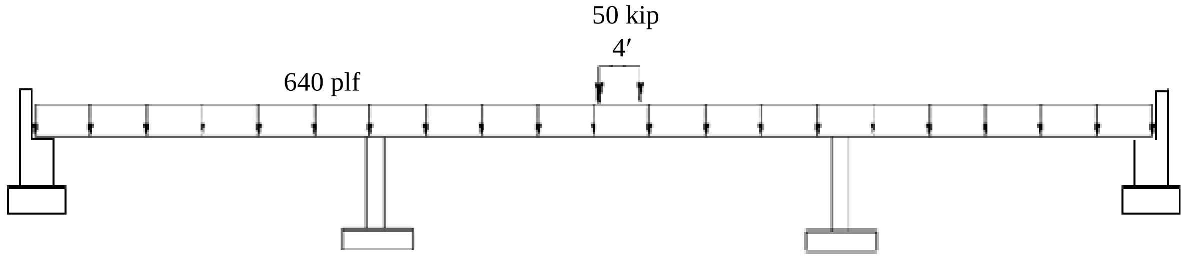
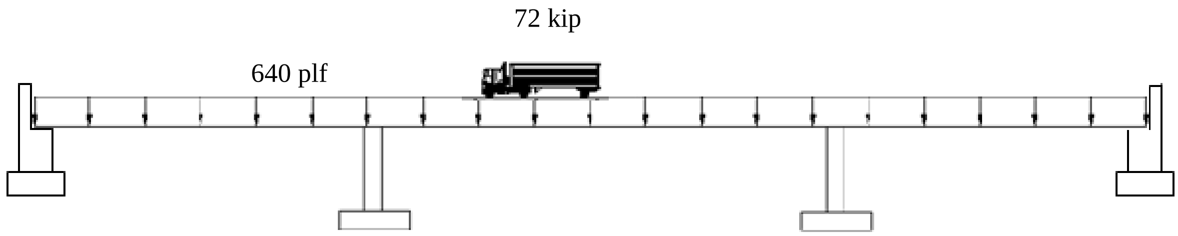


Figure 3.4-1 HL-93 Design Truck

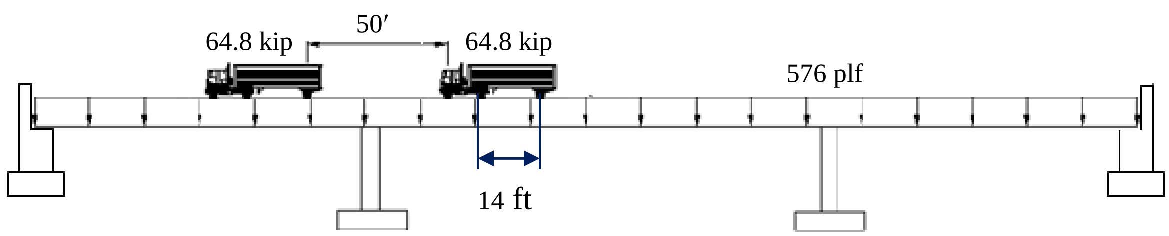
The following 4 cases represent, in general, the requirements for HL-93 loads as shown in Figure 3.4-2. Cases 1 and 2 are for positive moments and Cases 3 and 4 are for negative moments and bent reactions only.



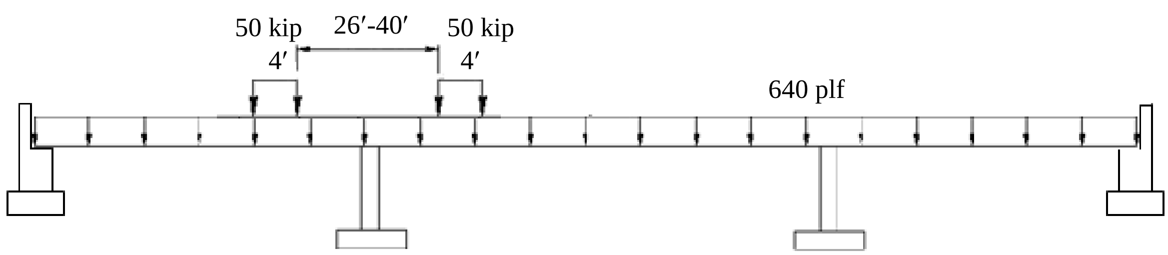
Case 1: tandem + lane



Case 2: design truck + lane



Case 3: two design trucks + lane



Case 4: two tandem trucks + lane

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 distribution 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 Design Truck Forces in Span 2 with *IM* = 1.33

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	1394.54	-41.07	5.671	4.231	-5950.39	321.25	5.671
16.80	4.092	1675.23	187.00	5.671	4.231	-3537.61	47.32	5.671
33.60	4.092	4546.13	135.41	5.671	4.231	-2944.20	47.32	5.671
50.40	4.092	6836.51	77.46	5.671	4.092	-2276.08	46.92	5.671
67.20	4.092	8272.60	14.47	5.671	4.092	-1707.10	46.92	5.671
84.00	4.092	8696.09	-194.92	5.671	4.092	-1138.12	46.78	5.671
100.80	4.092	8215.33	-259.64	5.671	4.092	-1523.62	-41.62	5.671
117.60	4.092	6730.25	-322.23	5.671	4.092	-2028.19	-41.62	5.671
134.40	4.092	4419.12	-379.54	5.671	4.092	-2535.09	-42.00	5.671
151.20	4.092	1570.81	-430.11	5.671	4.260	-3189.65	-252.06	5.671
165.00	4.092	1584.83	46.37	5.671	4.260	-6238.79	-329.02	5.671

Table 3.4-2 HL-93 Tandem Forces in Span 2 with *IM* = 1.33

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	995.30	-29.31	5.671	4.231	-4199.54	229.49	5.671
16.80	4.092	1812.59	156.97	5.671	4.231	-2515.07	33.64	5.671
33.60	4.092	3802.35	121.81	5.671	4.231	-2093.18	33.64	5.671
50.40	4.092	5408.62	81.97	5.671	4.092	-1618.18	33.36	5.671
67.20	4.092	6435.94	38.32	5.671	4.092	-1213.66	33.36	5.671
84.00	4.092	6760.62	-184.42	5.671	4.092	-809.14	33.26	5.671
100.80	4.092	6394.19	-229.54	5.671	4.092	-1087.36	-29.70	5.671
117.60	4.092	5333.83	-272.87	5.671	4.092	-1447.47	-29.70	5.671
134.40	4.092	3715.51	-312.24	5.671	4.092	-1809.23	-29.98	5.671
151.20	4.092	1744.93	-346.66	5.671	4.260	-2265.75	-178.53	5.671
165.00	4.092	1126.76	32.97	5.671	4.260	-4400.94	-229.07	5.671

Table 3.4-3 HL-93 Lane Forces in Span 2 with *IM* = 1.0

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	741.54	-12.38	5.671	4.231	-6369.34	308.97	5.671
16.80	4.092	852.85	37.25	5.671	4.231	-3687.00	209.28	5.671
33.60	4.092	1720.76	103.01	5.671	4.231	-1874.86	82.58	5.671
50.40	4.092	3069.57	120.20	5.671	4.092	-1280.05	4.59	5.671
67.20	4.092	4159.10	59.39	5.671	4.092	-1226.01	4.44	5.671
84.00	4.092	4509.60	-5.00	5.671	4.092	-1172.13	4.28	5.671
100.80	4.092	4123.37	-62.34	5.671	4.092	-1120.20	4.28	5.671
117.60	4.092	2998.13	-123.10	5.671	4.092	-1068.46	4.08	5.671
134.40	4.092	1709.11	-95.18	5.671	4.092	-1591.42	-84.70	5.671
151.20	4.092	942.33	-29.62	5.671	4.260	-3513.85	-211.24	5.671
165.00	4.092	894.24	17.29	5.671	4.260	-6220.63	-308.23	5.671

Table 3.4-4 HL-93 Design Vehicle Enveloped Forces in Span 2 with *IM* = 1.33

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	2136.08	-53.45	5.671	4.231	-14708.31	613.18	5.671
16.80	4.092	2665.43	194.22	5.671	4.231	-9177.36	454.12	5.671
33.60	4.092	6266.89	238.42	5.671	4.231	-5787.08	145.45	5.671
50.40	4.092	9906.08	197.65	5.671	4.092	-3556.13	51.52	5.671
67.20	4.092	12431.70	73.85	5.671	4.092	-2933.11	51.37	5.671
84.00	4.092	13205.69	-199.92	5.671	4.092	-2310.25	51.06	5.671
100.80	4.092	12338.69	-321.97	5.671	4.092	-2643.82	-37.33	5.671
117.60	4.092	9728.38	-445.33	5.671	4.092	-3096.65	-37.53	5.671
134.40	4.092	6128.23	-474.72	5.671	4.092	-4126.50	-126.71	5.671
151.20	4.092	2687.26	-376.27	5.671	4.260	-8884.28	-457.04	5.671
165.00	4.092	2479.07	63.66	5.671	4.260	-14643.57	-755.57	5.671

3.4.1.2 Permit Load

The California P-15 permit (CA 3.6.1.8) vehicle is used in conjunction with the Strength II limit state. For superstructure design, if refined methods are used, either 1 or 2 permit trucks shall be placed on the bridge at a time, whichever controls. If simplified distribution is used (AASHTO 4.6.2.2), girder distribution factors shall be the same as the design vehicle distribution factors.

Table 3.4-5 shows the maximum positive moments in Span 2 obtained by the CTBRIDGE program.

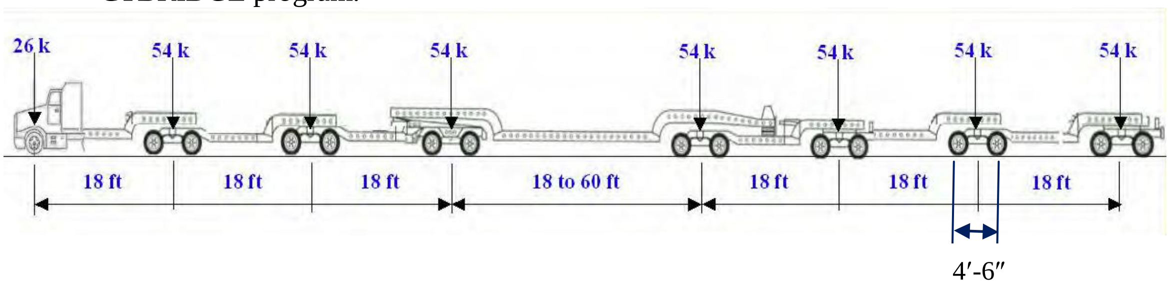


Figure 3.4-3 P-15 Truck

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	4301.45	-126.88	5.671	4.231	-24408.50	1094.85	5.671
16.80	4.092	3037.35	-126.88	5.671	4.231	-13982.51	920.77	5.671
33.60	4.092	8953.10	595.24	5.671	4.231	-9737.33	156.42	5.671
50.40	4.092	18103.38	500.79	5.671	4.092	-7528.62	155.11	5.671
67.20	4.092	24145.10	155.37	5.671	4.092	-5647.79	155.11	5.671
84.00	4.092	26029.03	-34.87	5.671	4.092	-3766.96	154.62	5.671
100.80	4.092	23859.67	-498.73	5.671	4.092	-4712.93	-128.55	5.671
117.60	4.092	17812.72	-498.73	5.671	4.092	-6271.59	-128.55	5.671
134.40	4.092	8607.76	-798.23	5.671	4.092	-7837.45	-129.75	5.671
151.20	4.092	3707.44	153.29	5.671	4.260	-13797.91	-947.21	5.671
165.00	4.092	5233.96	153.29	5.671	4.260	-24485.67	-1462.71	5.671

Notice that the maximum P-15 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 insure the structure withstands cyclic loading. A single HL-93 design truck with rear axle spacing of 30 ft shall be run across the bridge by itself for the first case. The second case is a P-9 truck by itself. Dynamic load allowance shall be 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, MPFs 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 shall be increased by the percentage of the vehicular dynamic load allowance, *IM* as specified by CA 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 (AASHTO 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: $\frac{58.83 - 2(1.42)}{12} = 4.66$ 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, as in CTBRIDGE. Apply a user load and input the load factors to a superstructure member in the longitudinal direction.

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

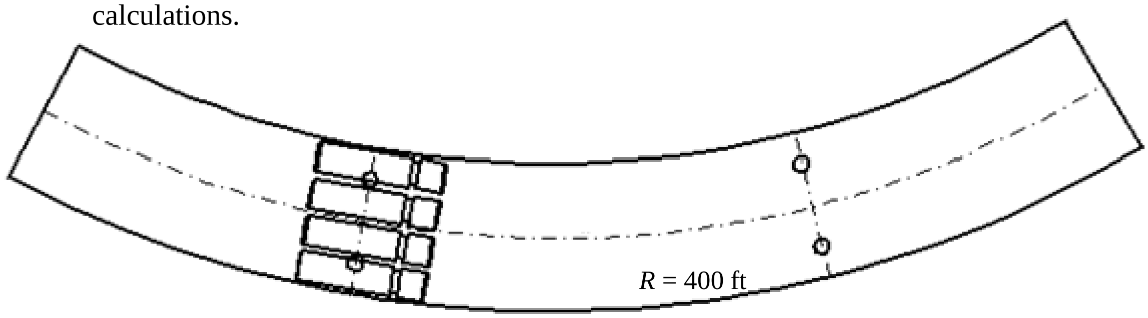


Figure 3.4-4 Centrifugal Force Example

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

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.

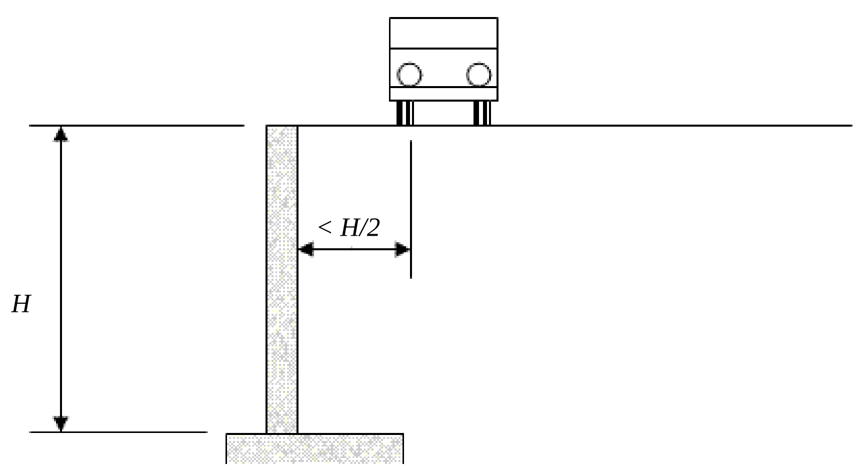


Figure 3.4-5 Applicability of Live Load Surcharge

When the condition of Figure 3.4-5 is met, then the following constant horizontal earth pressure shall be applied to the wall:

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

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:

$$\text{Abutment Height} = 30\,\text{ft}$$
$$h_{eq} = 2.0\,\text{ft}$$
$$\Delta_p = 0.3(0.12)(2.0) = 0.072\ \text{ksf}$$

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.

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 taken into account. 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:

$P = \dfrac{3EI_{col}\Delta}{L^3}$ (for pinned columns)

Where

Δ = 1 in.; E = 3834 ksi; $I_{col} = \dfrac{\pi r^4}{4}$; L = 44 ft at Bent 2, 47 ft at Bent 3;

r = 3.0 ft

The point of no movement = $\dfrac{\Sigma \dfrac{PD}{100}}{\Sigma P}(100) = \dfrac{757}{375}(100) = 201.8\,\text{ft}$

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 is determined by the following formula:

$$\Delta_T = \alpha L(T_{MaxDesign} - T_{MinDesign})/2$$

Using a temperature change of +/-40°F, we can now determine a movement factor using concrete properties.

$$\text{Movement Factor} = \alpha \times \Delta T$$

α = coefficient of thermal expansion for a given material

Movement Factor = (0.000006/°F)(40°F)($1200 \frac{\text{in.}}{100\text{ ft})}$ = 0.29 in./100ft.

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

$$\text{Bent 2} = (0.29)\frac{(201.8 - 126)}{100} = 0.220\text{ in.}$$

$\text{Bent 3} = (0.29)\frac{294-201.8}{100} = 0.267\text{ in.}$

The factored load is calculated using γ_{TU} = 0.5. For joint displacements the larger factor γ_{TU} = 1.2 is used. Refer to Chapter 14 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 structure. 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. or 2 in. at Service-I Limit State 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. 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 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.

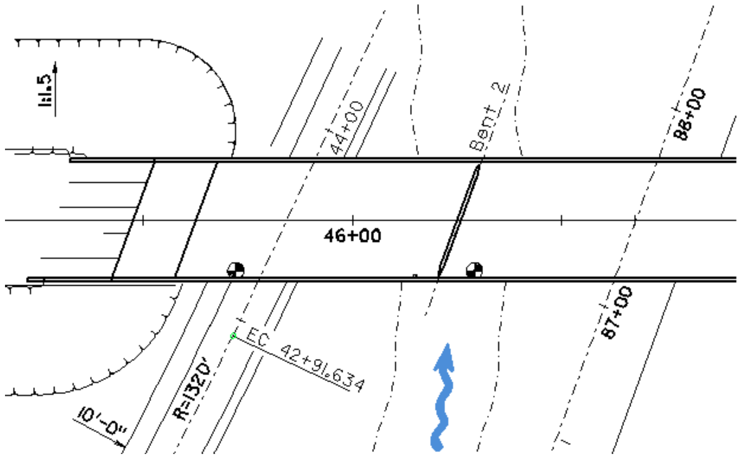


Figure 3.4-6 Stream Flow Example

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:

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

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

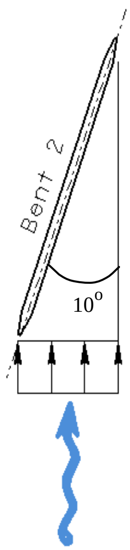


Figure 3.4-7 Longitudinal to Pier Forces due to Stream Flow

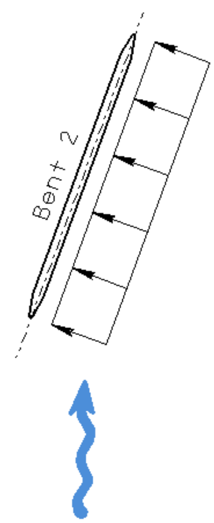


Figure 3.4-8 Transverse to Pier Forces due to Stream Flow

This pressure is applied to the pier's projected area, assuming 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 30 ft from ground to top of barrier. Wind load primarily affects the substructure design.

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

First calculate the design wind velocity:

$$V_{DZ} = 2.5 V_0 \left(\frac{V_{30}}{V_B}\right) \ln\left(\frac{Z}{Z_0}\right) \qquad \text{(AASHTO 3.8.1.1-1)}$$

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

$$V_{DZ} = 2.5 \times 8.2 \left(\frac{100}{100}\right) \ln\left(\frac{50.25}{0.23}\right) = 110.4\text{ mph}$$

Next, a design wind pressure, P_D is calculated.

$$P_D = P_B \left(\frac{V_{DZ}}{V_B}\right)^2 \qquad \text{(AASHTO 3.8.1.2.1-1)}$$

For the superstructure with wind acting normal to the structure (skew = 0 degree),

$$P_D = 0.05 \left(\frac{110.4}{100}\right)^2 = 0.061\text{ ksf}$$

For the columns

$$P_D = 0.04 \left(\frac{110.4}{100}\right)^2 = 0.049\text{ ksf}$$

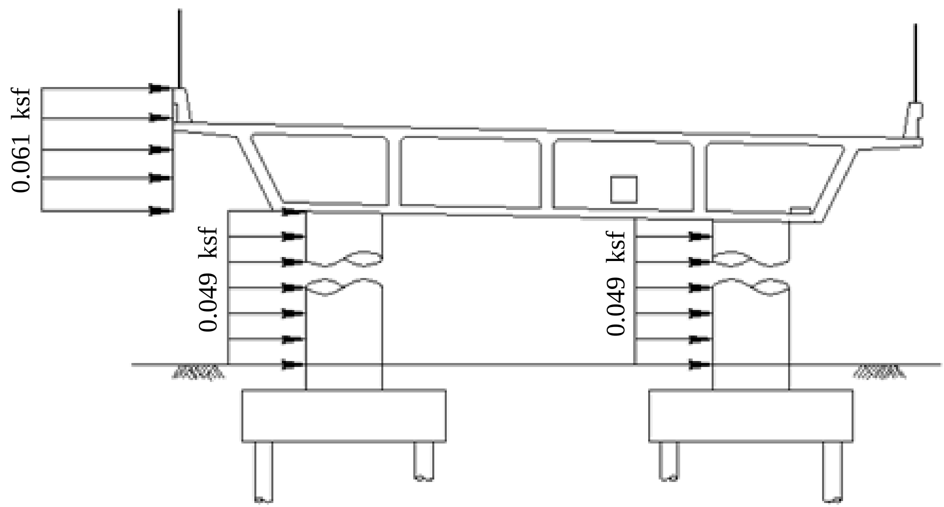


Figure 3.4-9 *WS* Application

Table 3.4-8 Wind Load at Various Angles of Attack

	Superstructure			
Skew	P_D lat	P_D long		
0	0.061	0.000		
15	0.054	0.007		
30	0.050	0.015		
45	0.040	0.020		
60	0.021	0.023		

In order to use these pressures, it is convenient to turn these into line loads for application to a frame analysis model.

Load on the spans = $(6.75+2.67)\times 0.061=0.575\text{ klf} > 0.30\text{ klf (min)}$

Load on columns = $6.0\times 0.049=0.294\text{klf}$

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

For the superstructure use table 3.8.1.2.2-1 to calculate the pressure from various angles skewed from the perpendicular to the longitudinal axis. Results are shown above 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.

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:

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

At 0 degrees:

$$F_L = (0)\cos(20^o) + 0.061(6.75+2.67)\sin(20^o) = 0.196 \text{ klf}$$

At 60 degrees:

$$F_L = 0.023(6.75+2.67)\cos(20^o) + 0.021(6.75+2.67)\sin(20^o)$$
$$= 0.204 \text{ klf} + 0.068 \text{ klf} = 0.272 \text{ klf}$$

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

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

At 0 degrees:

$$F_T = (0)\sin(20^o) + 0.061(9.42)\cos(20^o) = 0.540 \text{ klf}$$

At 60 degrees:

$$F_T = 0.023(9.42)\sin(20^o) + 0.021(9.42)\cos(20^o)$$
$$= 0.074 \text{ klf} + 0.186 \text{ klf} = 0.260 \text{ klf}$$

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.

3.4.12 Wind on Live Load, *WL*

This load is applied directly to vehicles traveling on the bridge during periods of a moderately high wind of 55 mph. This load is to be 0.1 klf applied transverse to the bridge deck. *WL* load application is shown in Figure 3.4-11.

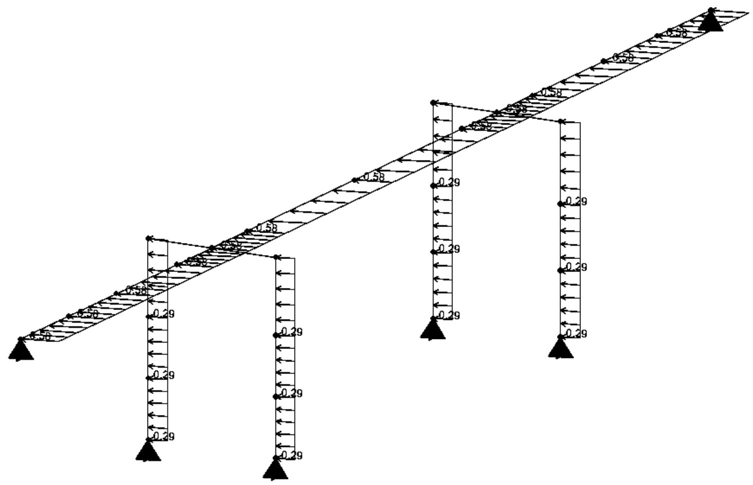


Figure 3.4-10 Wind on Structure

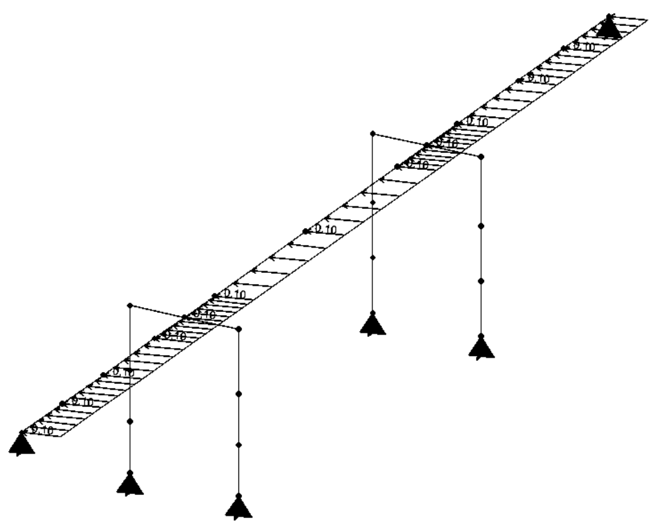


Figure 3.4-11 Wind on Live Load

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 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 (AASHTO 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 732 barrier rail from our example bridge as follows:

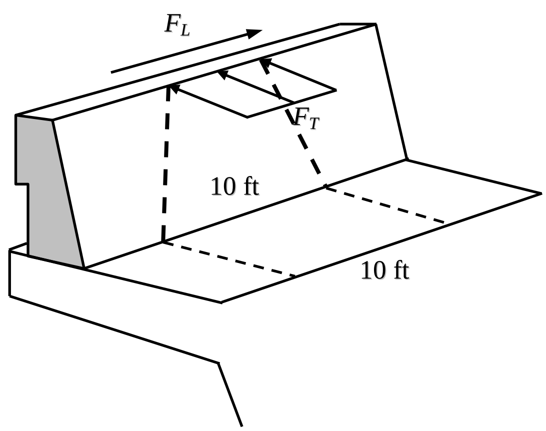


Figure 3.4-12 *CT* Force on Barrier

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 2′-8″, we can calculate the moment per foot as follows:

$$M_{CT} = \frac{54 \times 2.67}{10} = 14.4 \text{ kip-ft/ft}$$

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

$1.2 \times 14.4 = 17.28$ kip-ft/ft

Standard plan barriershave already been designed for these *CT* forces. However, these forces must be carried into the overhang and deck. Caltrans deck design charts in MTD 10-20 (Caltrans, 2008) were developed to include these *CT* forces in the overhang. 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, 2010). Due to the infrequent occurrence of these bridges, an example of *CV* force calculations will not be made here.

3.4.17 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 Volume III of this practice manual. It is recommended that *EQ* forces be considered early in the design process in order to properly size members.

3.5 LOAD DISTRIBUTION FOR BEAM-SLAB 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 0.86 klf for two barriers. The barrier load to each girder is simply 0.86/5 = 0.172 klf (Figure 3.5-1).

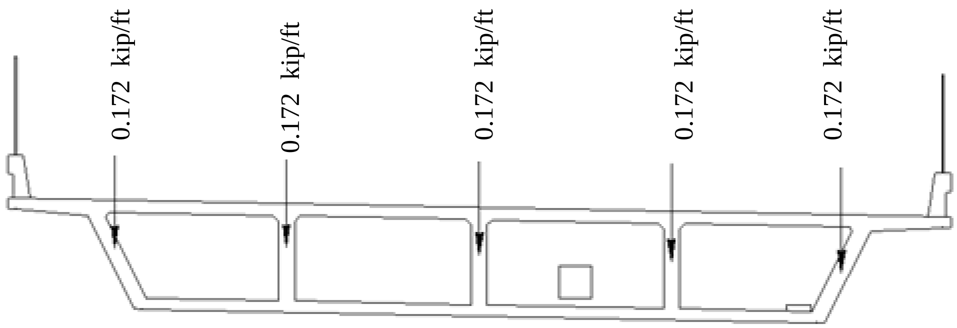


Figure 3.5-1 Barrier Distribution

3.5.1.2 Soundwalls

Since a soundwall has a much higher load per lineal length than a barrier, a more refined analysis should be performed to obtain more accurate distribution. The following procedure can be found in MTD 22-2 (Caltrans, 2004) for non-seismic design.

Soundwall distribution is simplified by applying 100% of the soundwall shear demand on the exterior girder. Secondly, apply $1/n$ to the first interior girder; where n = number of girders. For moment, apply 60% to the exterior girder and $1/n$ to the first interior girder. It is assumed that other girders in the bridge are unaffected by the presence of the soundwall.

For the example bridge, assume a soundwall 10 ft tall using 8-inch blocks on the north side of the bridge. 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.

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 maximum overhang moment due to the HL-93 design truck (Strength I). Since the overhang is designed on a lineal length basis it is, therefore, 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.

Table 3.5-1 Soundwall Forces

Location	Whole Bridge		Apply to Exterior Girder		Apply to First Interior Girder	
	VY (kips)	MZ (kip-ft)	VY (kips)	MZ (kip-ft)	VY (kips)	MZ (kip-ft)
Span 1						
1.50	38.3	58.5	38.3	35.1	7.66	11.7
12.60	28.5	429.8	28.5	258.0	5.7	86.0
25.20	17.5	719.8	17.5	432.0	3.5	144.0
37.80	6.4	870.0	6.4	522.0	1.28	174.0
50.40	-4.7	880.4	-4.7	528.0	-0.94	176.0
63.00	-15.8	751.1	-15.8	451.0	-3.16	150.0
75.60	-26.9	482.1	-26.9	289.0	-5.38	96.4
88.20	-38.0	73.3	-38	44.0	-7.6	14.7
100.80	-49.0	-475.1	-49	-285.0	-9.8	-95.0
113.40	-60.1	-1162.9	-60.1	-698.0	-12.0	-233.0
123.00	-68.6	-1780.8	-68.6	-1068.0	-13.7	-356.2
Span 2						
3.00	70.9	-1734.7	70.9	-1041.0	14.2	-347.0
16.80	58.7	-839.8	58.7	-504.0	11.7	-168.0
33.60	44.0	23.4	44.0	14.0	8.8	4.68.0
50.40	29.2	638.4	29.2	383.0	5.84	128.0
67.20	14.4	1005.3	14.4	603.0	2.88	201.0
84.00	-0.3	1123.9	-0.3	674.0	-0.06	225.0
100.80	-15.1	994.3	-15.1	597.0	-3.02	199.0
117.60	-29.9	616.4	-29.9	370.0	-5.98	123.0
134.40	-44.6	-9.5	-44.6	-5.7	-8.92	-1.9
151.20	-59.4	-883.3	-59.4	-530.0	-11.9	-177.0
165.00	-71.5	-1786.9	-71.5	-1072.0	-14.3	-357.0
Span 3						
3.00	64.1	-1551.0	64.1	-931.0	12.8	-310.0
11.80	56.3	-1021.0	56.3	-613.0	11.3	-204.0
23.60	46.0	-418.0	46.0	-251.0	9.20	-83.6
35.40	35.6	63.2	35.6	37.9	7.12	12.6
47.20	25.2	422.0	25.2	253.0	5.04	84.4
59.00	14.8	658.0	14.8	395.0	2.96	132.0
70.80	4.4	771.0	4.4	463.0	0.88	154.0
82.60	-6.0	762.0	-6.0	457.0	-1.20	152.0
94.40	-16.3	631.0	-16.3	378.0	-3.26	126.0
106.20	-26.7	377.0	-26.7	226.0	-5.34	75.3
116.50	-35.8	54.7	-35.8	32.8	-7.16	10.9

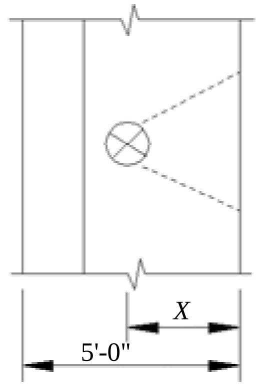


Figure 3.5-2 Overhang Wheel Load

The moment arm for this load is:

$X = 5.0–1.42–1.0 = 2.58$ ft

The strip width is therefore:

Strips = 45.0+10*X* = 45+10(2.58) = 70.8 in. (AASHTO Table 4.6.2.1.3-1)

Overhang moment for design is therefore:

$$M_{LL} = \frac{(16)(2.58)}{70.8/12} = 7.0 \text{ kip-ft/ft}$$

Include dynamic load allowance:

$$M_{LL} = 7.0(1.33) = 9.31\ kip - ft/ft$$

Include the Strength I load factor of 1.75:

$$M_{LL} = 9.31(1.75) = 16.3\ kip - ft/ft$$

3.5.2.2 CIP Box Girder

Live load distribution to each girder in a box girder bridge is accomplished using empirical formulas to determine how many live load lanes each girder must be designed to carry. 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. 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:

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

Two or more lanes loaded case:

$$g_M = \left(\frac{13}{N_c}\right)^{0.3}\left(\frac{S}{5.8}\right)\left(\frac{1}{L}\right)^{0.25} = \left(\frac{13}{4}\right)^{0.3}\left(\frac{12}{5.8}\right)\left(\frac{1}{126}\right)^{0.25} = 0.880$$

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

Table 3.5-2 Girder Live Load Distribution for Moment

Span	Fatigue Limit State*	All other Limit States
1	0.418	0.880
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, $(g_M)_{total} = 4.400$.

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

Span 1

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

One lane loaded case:

$$g_S = \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{d}{12.0L}\right)^{0.1} = \left(\frac{12.0}{9.5}\right)^{0.6}\left(\frac{81}{12\times126.0}\right)^{0.1} = 0.859$$

Fatigue limit state:

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

Two or more lanes loaded case:

$$g_S = \left(\frac{S}{7.3}\right)^{0.9}\left(\frac{d}{12.0L}\right)^{0.1} = \left(\frac{12.0}{7.3}\right)^{0.9}\left(\frac{81}{12\times126.0}\right)^{0.1} = 1.167$$

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:

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

$$n = \frac{E_B}{E_D} = \frac{4696}{3834} = 1.225$$

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.

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

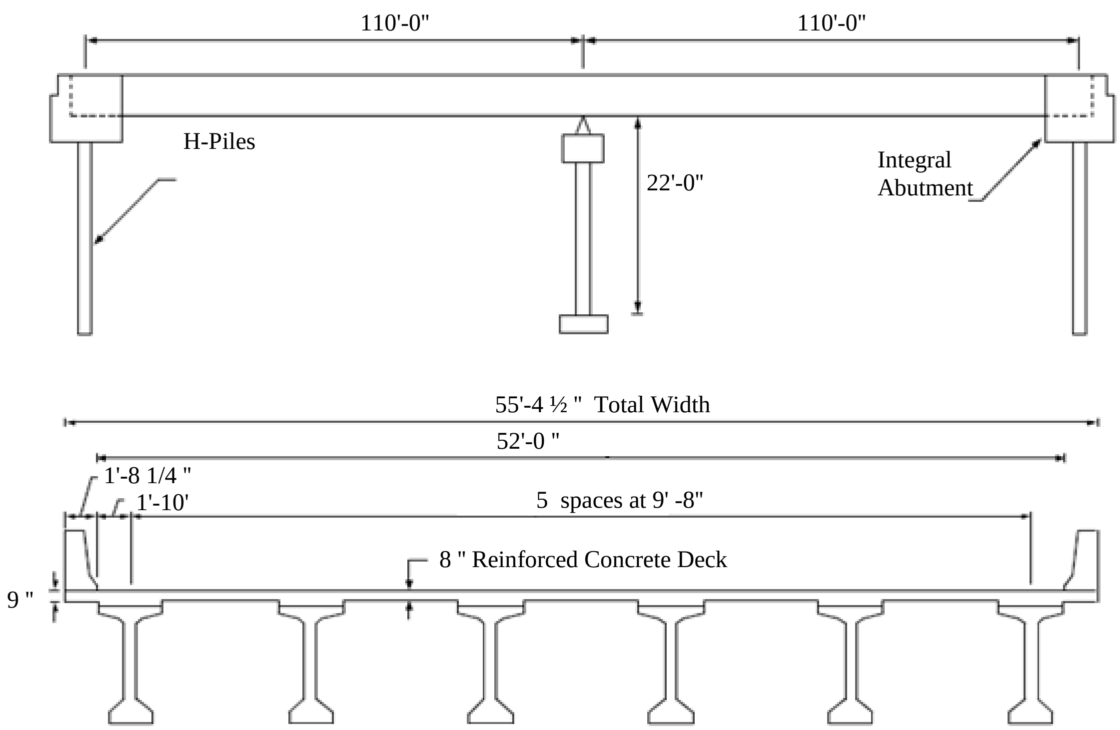


Figure 3.5-3 Precast Bulb-Tee Bridge to be Used for Distribution Calculations

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(AASHTO 4.6.2.2.1-2), but was not used here.

Fatigue limit state: $g_M = \dfrac{0.542}{1.2} = 0.452$

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 shall be placed to maximize the reaction of one lane of live load (Figure 3.5-4).

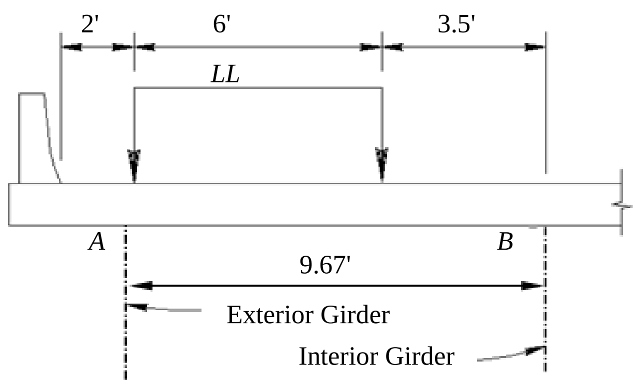


Figure 3.5-4 Lever Rule Example for Exterior Girder Distribution Factor

$$\Sigma M_B = 0$$

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

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

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: $g_M = \frac{0.747}{1.2} = 0.623$

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 shall be 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) $(M_T)_{\text{max}} + (M_L)_{\text{assoc}} + P_{assoc}$
2) $(M_L)_{\text{max}} + (M_T)_{\text{assoc}} + P_{assoc}$
3) $P_{max} + (M_L)_{\text{assoc}} + (M_T)_{assoc}$

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).

3.5.3.1 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 bottom of the column:

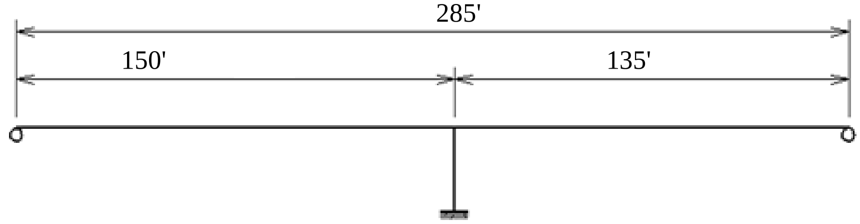


Figure 3.5-5 Example Bridge Elevation for Substructure Calculations

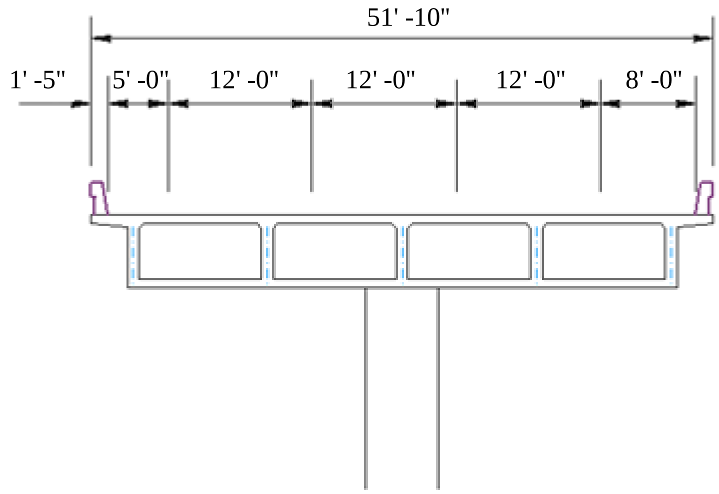


Figure 3.5-6 Example Bridge Typical Section for Substructure Calculations

Live load effects from a longitudinal frame analysis are tabulated:

Table 3.5-6 CTBRIDGE Live Load Effects

Bottom 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	1319

1. Design Vehicle

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

To obtain the 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. The edge of deck to edge of deck case should also be checked. Longitudinally, the vehicles are located over the bent thus maximizing M_T.

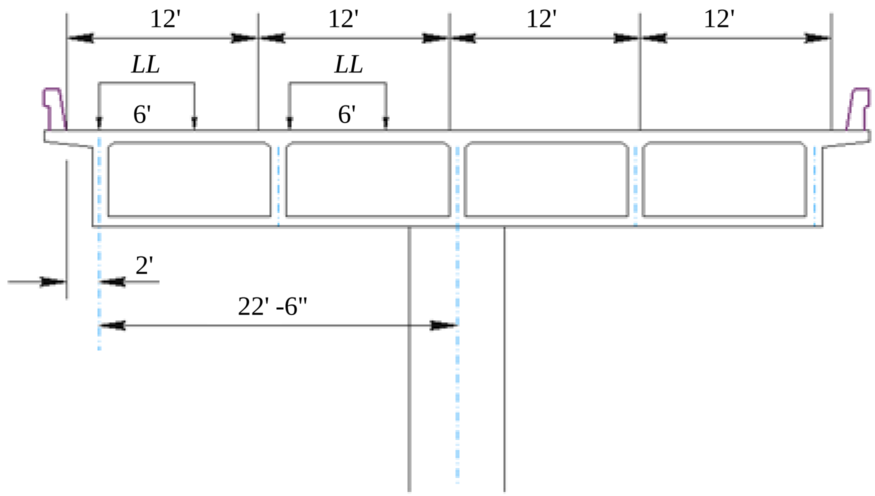


Figure 3.5-7 Vehicle Position for $(M_T)_{max}$

$$LL = 154 + 103 = 257 \text{ kips}$$

Multiple presence factor, m = 1.0 for two lanes.

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

$$(M_L)_{associated} = (66 + 39) \times 2 = 210 \text{ kip-ft}$$

$$P_{associated} = 257 \times 2 = 514 \text{ kips}$$

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:

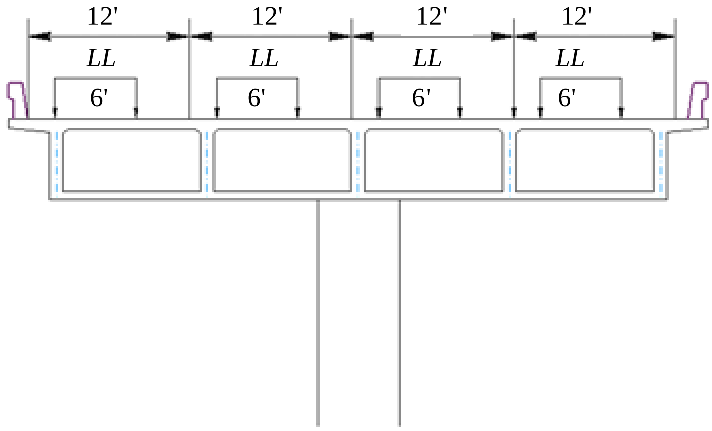


Figure 3.5-8 Vehicle Position for $(P)_{max}$

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

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

$$= 1{,}002 \text{ kip - ft}$$

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

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

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:

$$(M_T)_{associated} = (0.65)\frac{(100+61)}{2}(22.5+16.5+10.5+4.5-1.5-7.5-135.-19.5)$$
$$= 628 \text{ kip} - \text{ft}$$

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

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

2. Permit Vehicle

Next calculate the live load forces at the bottom 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.

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

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

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

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

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

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

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

Summary of the live load forces at the bottom 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	6939	628	1002
M_L	210	1802	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	12285	8991	12285
M_L	480	2638	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.

Table 3.5-9 Skew correction of shear forces

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*	$1.0+\frac{\theta}{50}$ for exterior girder $1.0+\frac{\theta}{300}$ for first interior girder	$0 < \theta \le 60^{o}$ $6.0 < S \le 13.0$ $20 \le L \le 240$ $35 \le d \le 110$ $N_c \ge 3$

The example bridge has a 20 degree skew. Correction Factors are as follows:

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

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

To illustrate the application of these correction factors, apply them to dead load (*DC*) shear forces only on the northern most exterior girder. Correction would also be made to *DW* and *LL* in general (as well as the other exterior girders). Figure 3.5-9 shows the girder layout and Table 3.5-10 lists *DC* correction factors for the example bridge.

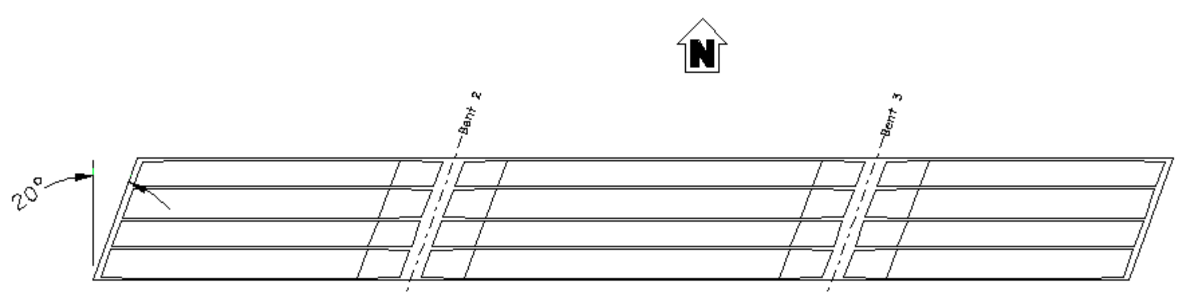


Figure 3.5-9 Girder Layout

Table 3.5-10 Example Bridge *DC* Skew Correction (Northern Most Girder)

Span	Tenth Point	V_{DC} (kips)	$(V_{DC})_{\text{per girder}}$ (kips)	Correction	$(V_{DC})_{\text{corrected}}$ (kips)
1	0.0	721	144	1.39	201
	0.1	540	108	1.32	143
	0.2	334	66.8	1.24	82.8
	0.3	128	25.6	1.16	29.7
	0.4	-78	-15.6	1.08	-16.8
	0.5	-284	-56.8	1.00	-56.8
	0.6	-490	-98.0	1.00	-98.0
	0.7	-696	-139	1.00	-139
	0.8	-906	-181	1.00	-181
	0.9	-1121	-224	1.00	-224
	1.0	-1285	-257	1.00	-257
2	0.0	1357	271	1.386	376
	0.1	1124	225	1.32	297
	0.2	840	168	1.24	208
	0.3	562	112	1.16	130
	0.4	288	57.5	1.08	62.1
	0.5	13.2	2.6	1.00	2.6
	0.6	-261	-52.3	1.00	-52.3
	0.7	-536	-107	1.00	-107
	0.8	-815	-163	1.00	-163
	0.9	-1098	-220	1.00	-220
	1.0	-1331	-266	1.00	-266
3	0.0	1219	244	1.38	336
	0.1	1068	214	1.32	282
	0.2	866	173	1.24	215
	0.3	669	134	1.16	155
	0.4	476	95.3	1.08	103
	0.5	284	56.7	1.00	56.7
	0.6	90.6	18.1	1.00	18.1
	0.7	-102	-20.4	1.00	-20.4
	0.8	-295	-59.0	1.00	-59.0
	0.9	-488	-97.6	1.00	-97.6
	1.0	-656	-131	1.00	-131

3.6 LOAD FACTORS AND COMBINATION

The Limit States of AASHTO (2012) and *CA* (Caltrans, 2014) 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} = 20,936 kip-ft

M_{DW} = 2,496 kip-ft

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

M_{PERMIT} = 26,029 kip-ft

M_{PS} = 7,023 kip-ft

Factored positive moments are calculated as follows:

Strength I:

$$M = 1.25(20{,}936) + 1.5(2{,}496) + 1.0(7{,}023) + 1.75(13{,}206) = 60{,}047 \text{ kip-ft}$$

Strength II:

$$M = 1.25(20{,}936) + 1.5(2{,}496) + 1.0(7{,}023) + 1.35(26{,}029) = 72{,}076 \text{ kip-ft}$$

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*	LL_{Permit} *IM* *CE*	*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	-	1.35	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 o Metal Box Culverts and Structural Plate Culverts with Deep Corrugations o Thermoplastic Culverts o 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

BR	=	vehicular braking force (3.4.3)
CE	=	vehicular centrifugal force (3.4.4)
CR	=	force effects due to creep (3.3.7)
CT	=	vehicular collision force (3.4.15)
CV	=	vessel collision force (3.4.16)
DC	=	dead load of components (3.3.1)
DD	=	downdrag (3.3.3)
DW	=	dead load of wearing surfaces and utilities (3.3.2)
EH	=	horizontal earth pressure load (3.3.4)
EQ	=	earthquake (3.4.17)
ES	=	earth surcharge load (3.3.6)
EV	=	vertical pressure from dead load of earth fill (3.3.5)
FR	=	friction (3.4.13)
IC	=	ice load (3.4.14)
IM	=	vehicular dynamic load allowance (3.4.2)
LL	=	vehicular live load (3.4.1)
LS	=	live load surcharge (3.4.5)
PL	=	pedestrian live load (3.4.6)
PS	=	secondary forces from post-tensioning (3.3.9)
SE	=	force effects due to settlement (3.4.9)
SH	=	force effects due to shrinkage (3.3.8)
TG	=	force effects due to temperature gradient (3.4.8)
TU	=	force effects due to uniform temperature (3.4.7)
WA	=	water load and stream pressure (3.4.10)
WL	=	wind on live load (3.4.12)
WS	=	wind load on structure (3.4.11)

General Symbols

A	=	area of section (ft^2) (3.3.1)
C	=	centrifugal force factor (3.4.4)
C_D	=	drag coefficient (3.4.10)
C_L	=	lateral drag coefficient (3.4.10)
d	=	structure depth (in.) (3.5.2.2)
d_e	=	distance from cl exterior girder and face of barrier (ft) (3.5.2.3)
e	=	girder *LL* distribution factor multiplier for exterior girders (3.5.2.3)
e_g	=	vertical distance from c.g. beam to c.g. deck (in.) (3.5.2.3)
E	=	modulus of elasticity (ksi) (3.4.7)
f	=	*CE* fatigue factor (3.4.4)
F_t	=	transverse barrier collision force (kip) (3.4.15)
F_L	=	longitudinal barrier collision force (kip) (3.4.15)
g	=	gravitational acceleration (32.2 ft/sec) (3.4.4)
g_M	=	girder *LL* distribution factor for moment (3.5.2.2)
g_S	=	girder *LL* distribution factor for shear (3.5.2.2)
h_{eq}	=	equivalent height of soil for vehicular load (ft) (3.4.5)
H	=	height of element (ft) (3.3.4)
I	=	moment of inertia (ft^4) (3.4.7)
k	=	coefficient of lateral earth pressure (3.4.5)
k_a	=	active earth pressure coefficient (3.3.4)
k_s	=	earth pressure coefficient due to surcharge (3.3.6)
K_g	=	longitudinal stiffness parameter (in.^{4}) (3.5.2.3)
L	=	span length (ft) (3.4.7)
M_{CT}	=	vehicular collision moment on barrier (kip-ft) (3.4.15)
M_{LL}	=	moment due to live load (kip-ft) (3.5.2.1)
M_T	=	transverse moment on column (kip-ft) (3.5.3)
M_L	=	longitudinal moment on column (kip-ft) (3.5.3)
M_{DC}	=	moment due to dead load (kip-ft) (3.6)
M_{DW}	=	moment due to dead load wearing surface (kip-ft) (3.6)
$M_{HL\text{-}93}$	=	moment due to design vehicle (kip-ft) (3.6)

M_{PERMIT}	=	moment due to permit vehicle (kip-ft) (3.6)
M_{PS}	=	moment due to secondary pre-stress forces (kip-ft) (3.6)
n	=	modular ratio (3.5.2.3)
N_b	=	number of beams (3.5.2.3)
N_c	=	number of cells in the box girder section (3.5.2.2)
p	=	stream force pressure (ksf) (3.4.10)
p	=	pressure against wall (3.3.4)
P	=	axial load on column (k) (3.5.3)
P_B	=	base wind pressure (ksf) (3.4.11)
P_D	=	wind pressure (ksf) (3.4.11)
q_s	=	uniform surcharge applied to upper surface of the active earth wedge (ksf) (3.3.6)
R	=	radius of curvature of traffic lane (ft) (3.4.4)
S	=	center to center girder spacing (ft) (3.5.2.2)
t_s	=	top slab thickness (in.) (3.5.2.3)
v	=	highway design speed (ft/sec) (3.4.4)
V	=	design velocity of water (ft/sec) (3.4.10)
V_{DC}	=	shear due to dead load (kip) (3.5.4)
V_{DZ}	=	design wind velocity at elevation z (mph) (3.4.11)
V_o	=	friction velocity (mph) (3.4.11)
V_{30}	=	wind velocity at 30 ft above ground (mph) (3.4.11)
V_B	=	base wind velocity of 100 mph at 30 ft height (mph) (3.4.11)
w	=	uniform load (kip/ft) (3.3.1)
X	=	moment arm for overhang load (ft) (3.5.2.1)
z	=	depth to point below ground surface (ft) (3.3.4)
Z	=	height of structure at which wind loads are being calculated (ft) (3.4.11)
Z_o	=	friction length of upstream fetch (ft) (3.4.11)
α	=	coefficient of thermal expansion (3.4.7)
Δ_p	=	earth surcharge load (3.3.6)
γ_s	=	density of soil (pcf) (3.3.4)
θ	=	skew angle (degrees) (3.5.4)

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