CHAPTER 5.7 CONCRETE COLUMNS

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5.7.1 INTRODUCTION

Columns are structural elements that support the superstructure, transfer vertical loads from superstructure to foundation, and resist the lateral loads acting on the bridge due to seismic and various service loads.

5.7.2 TYPES OF COLUMNS

Columns are categorized along two parameters (Chen, 2014 and MacGregor, 1988): shape and height:

- Column sections are usually round, rectangular, solid, hollow, octagonal, or hexagonal.
- Columns may be short or tall. The column is called either short or tall according to its effective slenderness ratio (Kl_u/r).

5.7.3 DESIGN LOADS

The considered design loads, as specified in AASHTO 3.3.2, are:

BR = Vehicular braking force

CE = Vehicular centrifugal force

CR = Force effects due to creep

DC = Dead load of structural components and non-structural attachments

DW = Dead load of wearing surface and utilities

$LL_{HL\text{-}93}$ = HL-93 vehicular dynamic load, consisting of a combination of design truck + lane load or design tandem + lane load, including dynamic load allowance (IM)

LL_{Permit} = Permit vehicular dynamic Load, consisting of a P15 permit truck including the dynamic load allowance (IM)

PS = Secondary forces from post-tensioning for strength limits states; total prestress forces for service limit states

SH = Force effects due to shrinkage

TU = Force effects due to uniform temperature

WL = Wind on live load

WS = Wind load on structure

5.7.4 DESIGN CRITERIA

Columns are designed for Service, Strength, and Extreme Event limit states (AASHTO, 2017; Caltrans, 2019a). The Extreme Event I limit state must be in accordance with the current Caltrans Seismic Design Criteria (SDC) version 2.0 (Caltrans, 2019b). Columns should be designed as ductile members to deform inelastically for several cycles without significant degradation of strength or stiffness under the design earthquake demand (see SDC for more details).

5.7.4.1 Limit States

As stated above, columns are designed for three limit states:

- Service Limit State
- Strength Limit State
- Extreme Event Limit State

5.7.4.2 Forces

Bridge columns are subjected to axial loads, bending moments, and shears in both the longitudinal and transverse directions of the bridge.

5.7.5 APPROXIMATE EVALUATION OF SLENDERNESS EFFECTS

Analysis of concrete compression members is based on the slenderness ratio, Kl_u/r (AASHTO 5.6.4.3), where the effective length factor, K (AASHTO 4.6.2.5), is to compensate for rotational and translational boundary conditions other than pinned ends.

Theoretical and design values of K for individual members are given in AASHTO Table C4.6.2.5.-1.

Article 5.6.4.3 specifies that slenderness effects may be ignored if:

$$\frac{Kl_u}{r} < 22 \qquad \text{(members not braced against sidesway)}$$

$$\frac{Kl_u}{r} < 34 - 12\left(\frac{M_1}{M_2}\right) \qquad \text{(members braced against sidesway)}$$

where the ratio M_1 / M_2 is positive if the member is bent in single curvature and negative if bent in double curvature.

If the slenderness ratio exceeds the above-mentioned limits, the moment magnification procedure (AASHTO 4.5.3.2.2b) can approximate the analysis.

Note: If Kl_u/r exceeds 100, columns may experience appreciable lateral deflections resulting from vertical loads or the combination of vertical loads and lateral loads. For this case, a more detailed second-order non-linear analysis should be considered, including the significant change in column geometry and stiffness.

The factored moments may be increased to reflect the effects of deformation as follows:

$$M_c = \delta_b M_{2b} + \delta_s M_{2s} \quad \text{(AASHTO 4.5.3.2.2b-1)}$$

$$\delta_b = \frac{C_m}{1 - \dfrac{P_u}{\phi_k P_e}} \geq 1 \quad \text{(AASHTO 4.5.3.2.2b-3)}$$

$$\delta_s = \frac{1}{1 - \dfrac{\sum P_u}{\phi_k \sum P_e}} \quad \text{(AASHTO 4.5.3.2.2b-4)}$$

For members braced against sideway, δ_s is taken as 1.0 unless analysis indicates a lower value. For members not braced against sideway, δ_b shall be determined as for a braced member and δ_s for an unbraced member.

$$P_e = \frac{\pi^2 EI}{(Kl_u)^2} \quad \text{(AASHTO 4.5.3.2.2b-5)}$$

However, in the case where the member is braced against sidesway and without transverse loads between supports, C_m may be based on the following expression:

$$C_m = 0.6 + 0.4\frac{M_{1b}}{M_{2b}} \quad \text{(AASHTO 4.5.3.2.2b-6)}$$

To compute the flexural stiffness EI for a concrete column in determining P_e, Article 5.6.4.3 (AASHTO, 2017) recommends that the larger of the following be used:

$$EI = \frac{\dfrac{E_c I_g}{5} + E_s I_s}{1 + \beta_d} \quad \text{(AASHTO 5.6.4.3-1)}$$

$$EI = \frac{\dfrac{E_c I_g}{2.5}}{1 + \beta_d} \quad \text{(AASHTO 5.6.4.3-2)}$$

5.7.6 COMBINED AXIAL AND FLEXURAL STRENGTH

5.7.6.1 Interaction Diagrams

The flexural resistance of a concrete member is dependent upon the axial force acting on the member. Interaction diagrams for a reinforced concrete section are created assuming a series of strain distributions and computing the corresponding moments and axial forces. The resulting bending moments and axial forces are plotted to produce an interaction diagram for one-way bending as shown in Figure 5.7.6.1-1.

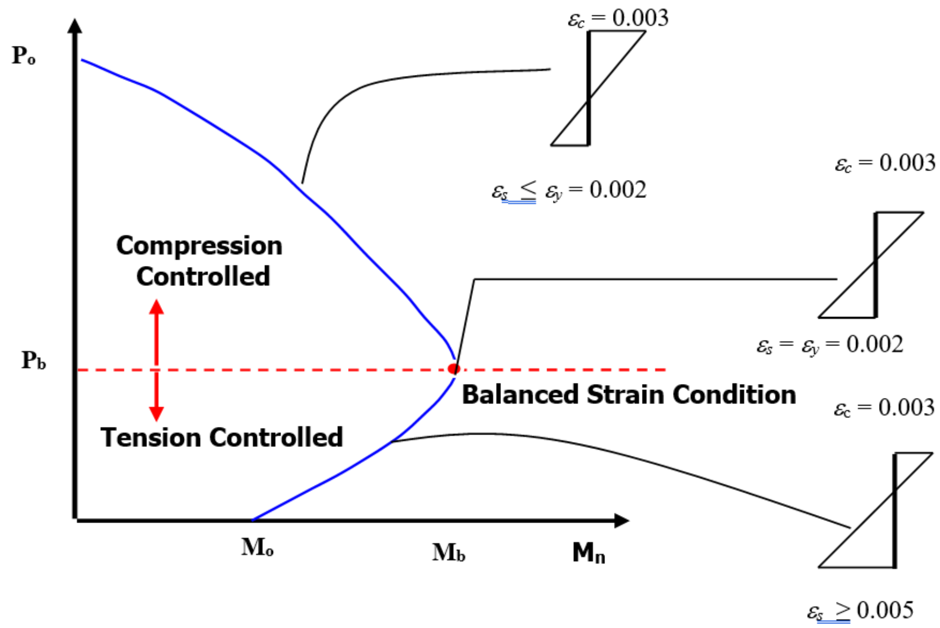


Figure 5.7.6.1-1 Generalized Strength Interaction Diagram for Reinforced Concrete Section with Grade 60 Reinforcement

When combined axial compression and bending moment act on a member having a low slenderness ratio and where column buckling is not a possible mode of failure, the strength of the member is governed by the material strength of the cross section. For a "short" column, the strength is achieved when the extreme concrete compression fiber reaches the strain of 0.003. In general, one of three modes of failure will occur: tension controlled, compression controlled, or balanced strain condition (AASHTO 5.6.2.1). These modes of failure are detailed below:

- Tension controlled: Sections are tension controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 (for Grade 60 reinforcement), just as the concrete in compression reaches its assumed strain limit of 0.003. This is a ductile failure.
- Compression controlled: Sections are compression controlled when the net tensile strain in the extreme tension steel is equal to or less than the net tensile strain in the reinforcement (for Grade 60 reinforcement ε_y = 0.002) at balanced strain condition at the time the concrete in compression reaches its assumed strain limit of 0.003. This is a brittle failure.
- Balanced strain condition: Where compression strain of the concrete (ε_c = 0.003) and yield strain of the steel (for Grade 60 reinforcement ε_y = 0.002) are

reached simultaneously, the strain is in a balanced condition.

5.7.6.2 Pure Compression

For members with spiral or closed hoop transverse reinforcement, the axial resistance is based on:

$$P_r = \phi P_n = \phi(0.85)P_o = \phi(0.85)\left[0.85f'_c\left(A_g - A_{st}\right) + A_{st}f_y\right]$$

(AASHTO 5.6.4.4-2)

For members with tie reinforcement, the axial resistance is based on:

$$P_r = \phi P_n = \phi(0.80)P_o = \phi(0.80)\left[0.85f'_c\left(A_g - A_{st}\right) + A_{st}f_y\right]$$

(AASHTO 5.6.4.4-3)

Note that the equations above assume non-prestressed columns for normal weight concrete with design compressive strengths up to 15 ksi and lightweight concrete up to 10 ksi.

5.7.6.3 Biaxial Flexure

AASHTO 5.6.4.5 specifies that in lieu of an analysis based on equilibrium and strain compatibility for biaxial flexure, noncircular members subjected to biaxial flexure and compression may be designed using one of the following approximate expressions for normal weight concrete with design compressive strengths up to 15.0 ksi:

For the factored axial load, $P_u \geq 0.1\phi f'_c A_g$

$$\frac{1}{P_{rxy}} = \frac{1}{P_{rx}} + \frac{1}{P_{ry}} + \frac{1}{\phi P_o} \quad \text{(AASHTO 5.6.4.5-1)}$$

$$P_o = 0.85f'_c\left(A_g - A_{st}\right) + A_{st}f_y \quad \text{(AASHTO 5.6.4.5-2)}$$

For the factored axial load, $P_u < 0.1\phi f'_c A_g$

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \leq 1.0 \quad \text{(AASHTO 5.6.4.5-3)}$$

5.7.7 COLUMN FLEXURAL DESIGN PROCEDURE

Column flexure design steps for permanent and transient loads are presented in the following sub-sections.

5.7.7.1 Longitudinal Analysis (CTBridge)

Perform a longitudinal analysis of the bridge under consideration using Caltrans CTBridge software (Caltrans, 2019c). Results will determine:

- Axial load (A_x) and longitudinal moment (M_z) at top of the column for DC and DW
- Maximum unfactored axial load (A_x) and associated longitudinal moment (M_z) of design vehicular live loads for one lane per bent
- Maximum unfactored longitudinal moment (M_z) and associated axial load (A_x) of design vehicular live loads for the one lane per bent

5.7.7.2 Transverse Analysis (CSiBridge)

Perform a transverse analysis of the bent cap (BDP Chapter 5.6) using commercial software CSiBridge (CSI, 2021). Results of the analysis are used to determine:

- Column axial load (P) and transverse moment (M_3) for DC and DW
- Maximum axial load (P) and associated transverse moment (M_3) for design vehicular live loads
- Maximum transverse moment (M_3) and associated axial load (P) for design vehicular live loads

Note that WinYIELD (Caltrans, 2008) uses the x-axis for longitudinal direction and the y-axis for the transverse direction. The CTBridge output designates M_z as M_x and A_x as P. The CSiBridge output designates the transverse moment, M_3, as M_y.

5.7.7.3 Column Live Load Input Procedure

5.7.7.3.1 Output from Longitudinal Analysis (CTBridge)

Column unfactored live load forces and moments for one lane from the longitudinal analysis (CTBridge) are summarized in Table 5.7.7.3-1 below:

Table 5.7.7.3-1 Unfactored Bent Reactions for One Lane, Dynamic Load Allowance Factors Not Included

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum axial load and associated longitudinal moment		Maximum axial load and associated longitudinal moment	
Force	A_x (kip)	M_z (kip-ft)	A_x (kip)	M_z (kip-ft)
Truck	$\left(A_{max}^{T}\right)_{CT}$	$\left[\left(M_z^{T}\right)_{assoc}\right]_{CT}$	$\left(A_{max}^{P}\right)_{CT}$	$\left[\left(M_z^{P}\right)_{assoc}\right]_{CT}$
Lane	$\left(A_{max}^{L}\right)_{CT}$	$\left[\left(M_z^{L}\right)_{assoc}\right]_{CT}$	--	--
Case	Maximum longitudinal moment and associated axial load		Maximum longitudinal moment and associated axial load	
Force	A_x (kip)	M_z (kip-ft)	A_x (kip)	M_z (kip-ft)
Truck	$\left(A_{assoc}^{T}\right)_{CT}$	$\left[\left(M_z^{T}\right)_{max}\right]_{CT}$	$\left(A_{assoc}^{P}\right)_{CT}$	$\left[\left(M_z^{P}\right)_{max}\right]_{CT}$
Lane	$\left(A_{assoc}^{L}\right)_{CT}$	$\left[\left(M_z^{L}\right)_{max}\right]_{CT}$	--	--

5.7.7.3.2 Output from Transverse Analysis (CSiBridge)

Axial forces presented in Table 5.7.7.3-1 are converted to two pseudo wheel loads spaced 6′-0″ apart. Pseudo wheel loads should include dynamic allowance factors (see BDP Chapter 5.6) used in the transverse analysis. For ease of application in CSiBridge, the HL-93 truck and lane loads should be combined:

$$\text{Design Pseudo Wheel Load} = \begin{cases} 1.33\left(\dfrac{\text{Bent Reaction}}{2}\right) & \text{for HL-93 Truck} \\ +1.00\left(\dfrac{\text{Bent Reaction}}{2}\right) & \text{for HL-93 Lane} \end{cases}$$

$$\text{Permit Pseudo Wheel Load} = 1.25\left(\frac{\text{Bent Reaction}}{2}\right) \quad \text{for P15}$$

The calculated pseudo wheel loads are the reaction for one lane. Multiple pseudo trucks must be applied transversely along the bent cap to determine the maximum reaction in the column. The wheel and truck spacings shown in Figure 5.7.7.3-1 shall be adjusted for skewed structures by dividing by the cosine of the skew angle.

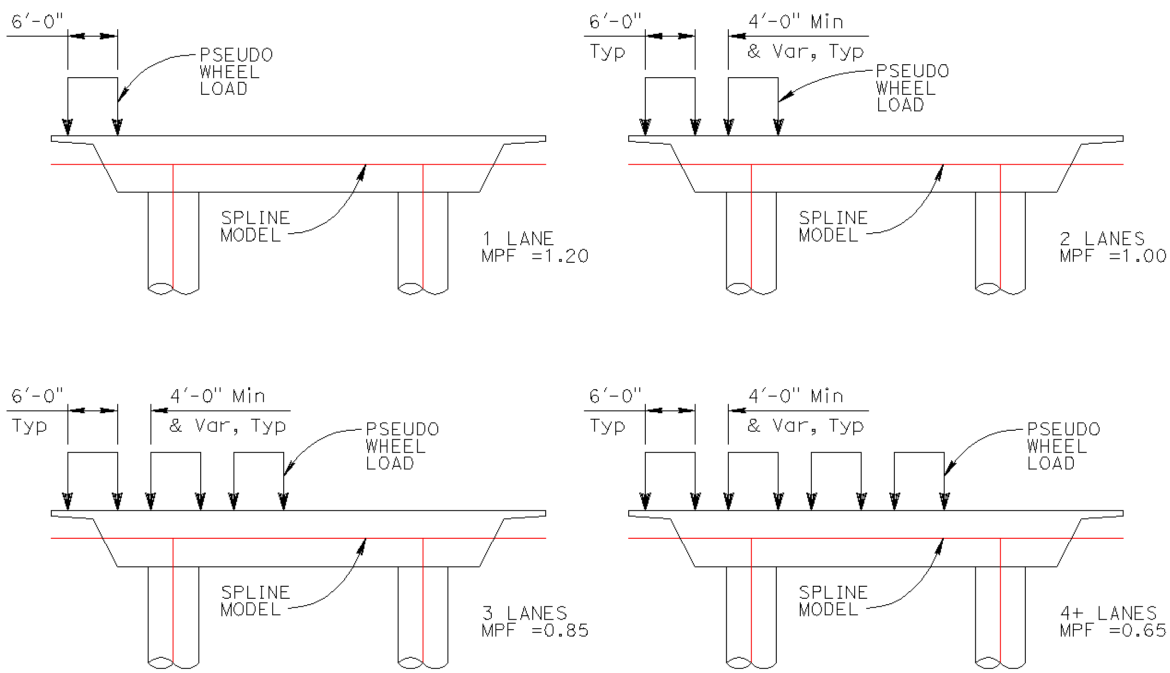


Figure 5.7.7.3-1 Pseudo Wheel Load Application in CSiBridge

The transverse analysis column forces for pseudo truck and permit wheel loadings are presented in Table 5.7.7.3-2.

Table 5.7.7.3-2 Unfactored Column Reaction, Including Dynamic Load Allowance Factors

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum axial load and associated transverse moment		Maximum axial load and associated transverse moment	
Force	P (kip)	M_3 (kip-ft)	P (kip)	M_3 (kip-ft)
Truck	$\left(P_{max}^{T}\right)_{CSi}$	$\left[\left(M_3^{T}\right)_{assoc}\right]_{CSi}$	$\left(P_{max}^{P}\right)_{CSi}$	$\left[\left(M_3^{P}\right)_{assoc}\right]_{CSi}$
Case	Maximum transverse moment and associated axial load		Maximum transverse moment and associated axial load	
Force	P (kip)	M_3 (kip-ft)	P (kip)	M_3 (kip-ft)
Truck	$\left(P_{assoc}^{T}\right)_{CSi}$	$\left[\left(M_3^{T}\right)_{max}\right]_{CSi}$	$\left(P_{assoc}^{P}\right)_{CSi}$	$\left[\left(M_3^{P}\right)_{max}\right]_{CSi}$

5.7.7.3.3 CTBridge Output Including Dynamic Load Allowance Factors

Multiply dynamic allowance factors by values in Table 5.7.7.3-1 divided by the number of columns to get reactions per column (Table 5.7.7.3-3):

$$\text{Column Reaction} = \begin{cases} 1.33\left(\dfrac{\text{Bent Reaction}}{\text{Number of Columns}}\right) & \text{for HL-93 Truck} \\ 1.00\left(\dfrac{\text{Bent Reaction}}{\text{Number of Columns}}\right) & \text{for HL-93 Lane} \\ 1.25\left(\dfrac{\text{Bent Reaction}}{\text{Number of Columns}}\right) & \text{for P15} \end{cases}$$

Table 5.7.7.3-3 Unfactored Column Reactions for One Lane, Including Dynamic Load Allowance Factors

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum axial load and associated longitudinal moment		Maximum axial load and associated longitudinal moment	
Force	P (kip)	M_x (kip-ft)	P (kip)	M_x (kip-ft)
Truck	$\left(P_{max}^{T}\right)_{CT}$	$\left[\left(M_{x}^{T}\right)_{assoc}\right]_{CT}$	$\left(P_{max}^{P}\right)_{CT}$	$\left[\left(M_{x}^{P}\right)_{assoc}\right]_{CT}$
Lane	$\left(P_{max}^{L}\right)_{CT}$	$\left[\left(M_{x}^{L}\right)_{assoc}\right]_{CT}$	--	--
Case	Maximum longitudinal moment and associated axial load		Maximum longitudinal moment and associated axial load	
Force	P (kip)	M_x (kip-ft)	P (kip)	M_x (kip-ft)
Truck	$\left(P_{assoc}^{T}\right)_{CT}$	$\left[\left(M_{x}^{T}\right)_{max}\right]_{CT}$	$\left(P_{assoc}^{P}\right)_{CT}$	$\left[\left(M_{x}^{P}\right)_{max}\right]_{CT}$
Lane	$\left(P_{assoc}^{L}\right)_{CT}$	$\left[\left(M_{x}^{L}\right)_{max}\right]_{CT}$	--	--

* The CTBridge output A_x and M_z are now shown in the WinYield coordinate system of P and M_x respectively.

5.7.7.3.4 HL-93 Truck and Lane Loads for Transverse Analysis (CSiBridge)

Split HL-93 vehicle reaction results from the transverse analysis (Table 5.7.7.3-3) into truck and lane loads as follows:

- Ratio of HL-93 Truck Load per Design Vehicle:

$$R1 = \frac{\left(P_{max}^{T}\right)_{CT}}{\left(P_{max}^{T}\right)_{CT} + \left(P_{max}^{L}\right)_{CT}}$$

- Ratio of HL-93 Lane Load per Design Vehicle:

$$R2 = \frac{\left(P_{max}^{L}\right)_{CT}}{\left(P_{max}^{T}\right)_{CT} + \left(P_{max}^{L}\right)_{CT}}$$

Unfactored column reactions (Table 5.7.7.3-4), including dynamic load allowance (CSiBridge):

Table 5.7.7.3-4 Unfactored Column Reactions, Including Dynamic Load Allowance Factors

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum axial load and associated longitudinal moment		Maximum axial load and associated longitudinal moment	
Force	P (kip)	M_y (kip-ft)	P (kip)	M_y (kip-ft)
Truck	$\left(P_{max}^{T}\right)_{CSi}$	$\left[\left(M_{y}^{T}\right)_{assoc}\right]_{CSi}$	$\left(P_{max}^{P}\right)_{CSi}$	$\left[\left(M_{y}^{P}\right)_{assoc}\right]_{CSi}$
Lane	$\left(P_{max}^{L}\right)_{CSi}$	$\left[\left(M_{y}^{L}\right)_{assoc}\right]_{CSi}$	--	--
Case	Maximum longitudinal moment and associated axial load		Maximum longitudinal moment and associated axial load	
Force	P (kip)	M_y (kip-ft)	P (kip)	M_y (kip-ft)
Truck	$\left(P_{assoc}^{T}\right)_{CSi}$	$\left[\left(M_{y}^{T}\right)_{max}\right]_{CSi}$	$\left(P_{assoc}^{P}\right)_{CSi}$	$\left[\left(M_{y}^{P}\right)_{max}\right]_{CSi}$
Lane	$\left(P_{assoc}^{L}\right)_{CSi}$	$\left[\left(M_{y}^{L}\right)_{max}\right]_{CSi}$	--	--

5.7.7.3.5 Combination of Longitudinal and Transverse Output

Combine forces and moments of Table 5.7.7.3-3 and Table 5.7.7.3-4.

- Case 1: Maximum Transverse Moment, M_y (Table 5.7.7.3-5)
- Case 2: Maximum Longitudinal Moment, M_x (Table 5.7.7.3-6)
- Case 3: Maximum Axial Load, P (Table 5.7.7.3-7)

Table 5.7.7.3-5 Case 1: Maximum Transverse Moment (My)

Vehicle	HL-93 Vehicle (Truck)	HL-93 Vehicle (Lane)	Permit Vehicle
M_y (kip-ft)	$\left[\left(M_y^T\right)_{max}\right]_{CSi}$	$\left[\left(M_y^L\right)_{max}\right]_{CSi}$	$\left[\left(M_y^P\right)_{max}\right]_{CSi}$
M_x (kip-ft)	$\left[\dfrac{\left(P_{assoc}^T\right)_{CSi}}{\left(P_{max}^T\right)_{CT}}\right]\left[\left(M_x^T\right)_{assoc}\right]_{CT}$	$\left[\dfrac{\left(P_{assoc}^L\right)_{CSi}}{\left(P_{max}^L\right)_{CT}}\right]\left[\left(M_x^L\right)_{assoc}\right]_{CT}$	$\left[\dfrac{\left(P_{assoc}^P\right)_{CSi}}{\left(P_{max}^P\right)_{CT}}\right]\left[\left(M_x^P\right)_{assoc}\right]_{CT}$
P (kip)	$\left(P_{assoc}^T\right)_{CSi}$	$\left(P_{assoc}^L\right)_{CSi}$	$\left(P_{assoc}^P\right)_{CSi}$

Table 5.7.7.3-6 Case 2: Maximum Longitudinal Moment (Mx)

Vehicle	HL-93 Vehicle (Truck)	HL-93 Vehicle (Lane)	Permit Vehicle
M_y (kip-ft)	$\left[\dfrac{\left(P_{assoc}^T\right)_{CT}}{\left(P_{max}^T\right)_{CT}}\right]\left[\left(M_y^T\right)_{assoc}\right]_{CSi}$	$\left[\dfrac{\left(P_{assoc}^L\right)_{CT}}{\left(P_{max}^L\right)_{CT}}\right]\left[\left(M_y^L\right)_{assoc}\right]_{CSi}$	$\left[\dfrac{\left(P_{assoc}^P\right)_{CT}}{\left(P_{max}^P\right)_{CT}}\right]\left[\left(M_y^P\right)_{assoc}\right]_{CSi}$
M_x (kip-ft)	$\left[\dfrac{\left(P_{max}^T\right)_{CSi}}{\left(P_{max}^T\right)_{CT}}\right]\left[\left(M_x^T\right)_{max}\right]_{CT}$	$\left[\dfrac{\left(P_{max}^L\right)_{CSi}}{\left(P_{max}^L\right)_{CT}}\right]\left[\left(M_x^L\right)_{max}\right]_{CT}$	$\left[\dfrac{\left(P_{max}^P\right)_{CSi}}{\left(P_{max}^P\right)_{CT}}\right]\left[\left(M_x^P\right)_{max}\right]_{CT}$
P (kip)	$\left[\dfrac{\left(P_{assoc}^T\right)_{CT}}{\left(P_{max}^T\right)_{CT}}\right]\left(P_{max}^T\right)_{CSi}$	$\left[\dfrac{\left(P_{assoc}^L\right)_{CT}}{\left(P_{max}^L\right)_{CT}}\right]\left(P_{max}^L\right)_{CSi}$	$\left[\dfrac{\left(P_{assoc}^P\right)_{CT}}{\left(P_{max}^P\right)_{CT}}\right]\left(P_{max}^P\right)_{CSi}$

Table 5.7.7.3-7 Case 3: Maximum Axial Load (P)

Vehicle	HL-93 Vehicle (Truck)	HL-93 Vehicle (Lane)	Permit Vehicle
M_y (kip-ft)	$\left[\left(M_y^T\right)_{assoc}\right]_{CSi}$	$\left[\left(M_y^L\right)_{assoc}\right]_{CSi}$	$\left[\left(M_y^P\right)_{assoc}\right]_{CSi}$
M_x (kip-ft)	$\left[\frac{\left(P_{max}^T\right)_{CSi}}{\left(P_{max}^T\right)_{CT}}\right]\left[\left(M_x^T\right)_{assoc}\right]_{CT}$	$\left[\frac{\left(P_{max}^L\right)_{CSi}}{\left(P_{max}^L\right)_{CT}}\right]\left[\left(M_x^L\right)_{assoc}\right]_{CT}$	$\left[\frac{\left(P_{max}^P\right)_{CSi}}{\left(P_{max}^P\right)_{CT}}\right]\left[\left(M_x^P\right)_{assoc}\right]_{CT}$
P (kip)	$\left(P_{max}^T\right)_{CSi}$	$\left(P_{max}^L\right)_{CSi}$	$\left(P_{max}^P\right)_{CSi}$

5.7.7.3.6 WinYIELD Live Load Input

Transfer Table 5.7.7.3-5, Table 5.7.7.3-6, and Table 5.7.7.3-7 data into Table 5.7.7.3-8, which will be used as load input for the WinYIELD program.

Table 5.7.7.3-8 Input for Column Live Load Analysis of WinYIELD Program

Vehicle	Case 1: Max Transverse (My)			Case 2: Max Longitudinal (Mx)			Case 3: Max Axial (P)		
	P15	HL-93 (Truck)	HL-93 (Lane)	P15	HL-93 (Truck)	HL-93 (Lane)	P15	HL-93 (Truck)	HL-93 (Lane)
M_y (kip-ft)	Table 5.7.7.3-5			Table 5.7.7.3-6			Table 5.7.7.3-7		
M_x (kip-ft)									
P (kip)									

5.7.7.4 Wind Loads (WS, WL)

Calculate wind moments and axial loads for column (see BDP Chapter 3 and Section 5.7.10.2.9).

5.7.7.5 Braking Force (BR)

Calculate braking force moments and axial load for column (see BDP Chapter 3 and Section 5.7.10.2.10).

5.7.7.6 Uniform Temperature (TU)

Calculate uniform temperature moments and axial load for column (See BDP Chapter 3 and Section 5.7.10.2.11)

5.7.7.7 Prestress Shortening Effects (CR, SH)

Calculate the prestress shortening moments and axial load for column (see BDP Chapter 3 and Section 5.7.10.2.12).

5.7.7.8 Prestressing Secondary Effect Forces (PS)

Calculate secondary prestress moments and axial loads (from CTBridge output, see Section 5.7.10.2.13).

5.7.7.9 Input Loads into WinYIELD

Transfer all loads into WinYIELD's load table.

5.7.7.10 Column Design/Check

Run WinYIELD to design or check the main vertical column reinforcement.

5.7.8 COLUMN SHEAR DESIGN PROCEDURE

Column shear demand values are calculated from longitudinal and transverse analyses.

5.7.8.1 Longitudinal Analysis

Perform a longitudinal analysis (CTBridge) to determine:

- Longitudinal shear (V_y) and moment (M_z) for DC and DW at the top and bottom of the column.
- Maximum longitudinal shear (V_y) and associated moment (M_z) for design vehicular live loads at the top and bottom of the bent unfactored reactions for one lane as shown in Table 5.7.8.1-1.

Table 5.7.8.1-1 Longitudinal Unfactored Bent Reactions for One Lane, Dynamic Load Allowance Factors Not Included

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum longitudinal shear and associated longitudinal moment at top of the column		Maximum longitudinal shear and associated longitudinal moment at top of the column	
Force	V_y (kip)	M_z (kip-ft)	V_y (kip)	M_z (kip-ft)
Truck	$\left[\left(V_y^T\right)_{max}\right]_{CT}$	$\left[\left(M_z^T\right)_{assoc}\right]_{CT}$	$\left[\left(V_y^P\right)_{max}\right]_{CT}$	$\left[\left(M_z^P\right)_{assoc}\right]_{CT}$
Lane	$\left[\left(V_y^L\right)_{max}\right]_{CT}$	$\left[\left(M_z^L\right)_{assoc}\right]_{CT}$	--	--
Case	Maximum longitudinal shear and associated longitudinal moment at bottom of the column		Maximum longitudinal shear and associated longitudinal moment at bottom of the column	
Force	V_y (kip)	M_z (kip-ft)	V_y (kip)	M_z (kip-ft)
Truck	$\left[\left(V_y^T\right)_{max}\right]_{CT}$	$\left[\left(M_z^T\right)_{assoc}\right]_{CT}$	$\left[\left(V_y^P\right)_{max}\right]_{CT}$	$\left[\left(M_z^P\right)_{assoc}\right]_{CT}$
Lane	$\left[\left(V_y^L\right)_{max}\right]_{CT}$	$\left[\left(M_z^L\right)_{assoc}\right]_{CT}$	--	--

5.7.8.2 Transverse Analysis

Perform a transverse analysis (CSiBridge) to determine:

1. Column transverse shears (V_2) and associated moment (M_3) for DC and DW
2. Maximum transverse shear (V_2) and associated moment (M_3) for design vehicular live loads at the top and bottom of the column with dynamic load allowance factors included, as shown in Table 5.7.8.2-1

Table 5.7.8.2-1 Transverse Unfactored Column Reactions Including Dynamic Load Allowance Factors

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum transverse shear and associated transverse moment at top of the column		Maximum transverse shear and associated transverse moment at top of the column	
Force	V_2 (kip)	M_3 (kip-ft)	V_2 (kip)	M_3 (kip-ft)
Truck	$\left[\left(V_2^T\right)_{max}\right]_{CSi}$	$\left[\left(M_3^T\right)_{assoc}\right]_{CSi}$	$\left[\left(V_2^P\right)_{max}\right]_{CSi}$	$\left[\left(M_3^P\right)_{assoc}\right]_{CSi}$
Case	Maximum transverse shear and associated transverse moment at bottom of the column		Maximum transverse shear and associated transverse moment at bottom of the column	
Force	V_2 (kip)	M_3 (kip-ft)	V_2 (kip)	M_3 (kip-ft)
Truck	$\left[\left(V_2^T\right)_{max}\right]_{CSi}$	$\left[\left(M_3^T\right)_{assoc}\right]_{CSi}$	$\left[\left(V_2^P\right)_{max}\right]_{CSi}$	$\left[\left(M_3^P\right)_{assoc}\right]_{CSi}$

5.7.8.3 Column Live Load Input Procedure

5.7.8.3.1 Output from Longitudinal 2D Analysis (CTBridge)

Include dynamic load allowance factors per column for CTBridge output (Table 5.7.8.1-1) and summarize the results in Table 5.7.8.3-1.

Table 5.7.8.3-1 Unfactored Column Longitudinal Shear and Associated Longitudinal Moment for One Lane, Including Dynamic Load Allowance Factors (CTBridge)

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum longitudinal shear and associated longitudinal moment at top of the column		Maximum longitudinal shear and associated longitudinal moment at top of the column	
Force	V_y (kip)	M_z (kip-ft)	V_y (kip)	M_z (kip-ft)
Truck	$\left[\left(V_y^T\right)_{max}\right]_{CT}$	$\left[\left(M_z^T\right)_{assoc}\right]_{CT}$	$\left[\left(V_y^P\right)_{max}\right]_{CT}$	$\left[\left(M_z^P\right)_{assoc}\right]_{CT}$
Lane	$\left[\left(V_y^L\right)_{max}\right]_{CT}$	$\left[\left(M_z^L\right)_{assoc}\right]_{CT}$	--	--
Case	Maximum longitudinal shear and associated longitudinal moment at bottom of the column		Maximum longitudinal shear and associated longitudinal moment at bottom of the column	
Force	V_y (kip)	M_z (kip-ft)	V_y (kip)	M_z (kip-ft)
Truck	$\left[\left(V_y^T\right)_{max}\right]_{CT}$	$\left[\left(M_z^T\right)_{assoc}\right]_{CT}$	$\left[\left(V_y^P\right)_{max}\right]_{CT}$	$\left[\left(M_z^P\right)_{assoc}\right]_{CT}$
Lane	$\left[\left(V_y^L\right)_{max}\right]_{CT}$	$\left[\left(M_z^L\right)_{assoc}\right]_{CT}$	--	--

5.7.8.3.2 Output from Transverse 2D Analysis (CSiBridge)

Re-form Table 5.7.8.2-1 to split truck reactions of CSiBridge analysis into truck and lane loads (Section 5.7.7.3.4) as shown in Table 5.7.8.3-2.

Table 5.7.8.3-2 Unfactored Column Reactions, Including Dynamic Load Allowance Factors (CSiBridge)

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum transverse shear and associated longitudinal moment at top of the column		Maximum transverse shear and associated longitudinal moment at top of the column	
Force	V_2 (kip)	M_3 (kip-ft)	V_2 (kip)	M_3 (kip-ft)
Truck	$\left[\left(V_2^T\right)_{max}\right]_{CSi}$	$\left[\left(M_3^T\right)_{assoc}\right]_{CSi}$	$\left[\left(V_2^P\right)_{max}\right]_{CSi}$	$\left[\left(M_3^P\right)_{assoc}\right]_{CSi}$
Lane	$\left[\left(V_2^L\right)_{max}\right]_{CSi}$	$\left[\left(M_3^L\right)_{assoc}\right]_{CSi}$	--	--
Case	Maximum transverse shear and associated longitudinal moment at bottom of the column		Maximum transverse shear and associated longitudinal moment at bottom of the column	
Force	V_2 (kip)	M_3 (kip-ft)	V_2 (kip)	M_3 (kip-ft)
Truck	$\left[\left(V_2^T\right)_{max}\right]_{CSi}$	$\left[\left(M_3^T\right)_{assoc}\right]_{CSi}$	$\left[\left(V_2^P\right)_{max}\right]_{CSi}$	$\left[\left(M_3^P\right)_{assoc}\right]_{CSi}$
Lane	$\left[\left(V_2^L\right)_{max}\right]_{CSi}$	$\left[\left(M_3^L\right)_{assoc}\right]_{CSi}$	--	--

Since the longitudinal shears and associated longitudinal moments are per one lane from CTBridge, the total longitudinal shears and associated longitudinal moments should be calculated as shown in Table 5.7.8.3-3.

Table 5.7.8.3-3 Total Longitudinal Shear (V_y) and Associated Longitudinal Moment (M_z)

Vehicle	HL-93 Vehicle (Truck)	HL-93 Vehicle (Lane)	P15 Vehicle
$(V_y)_{max}$ (kip)	$\left[\dfrac{\left(P_{max}^{T}\right)_{CSi}}{\left(P_{max}^{T}\right)_{CT}}\right]\left[\left(V_y^{T}\right)_{max}\right]_{CT}$	$\left[\dfrac{\left(P_{max}^{L}\right)_{CSi}}{\left(P_{max}^{L}\right)_{CT}}\right]\left[\left(V_y^{L}\right)_{max}\right]_{CT}$	$\left[\dfrac{\left(P_{max}^{P}\right)_{CSi}}{\left(P_{max}^{P}\right)_{CT}}\right]\left[\left(V_y^{P}\right)_{max}\right]_{CT}$
$(M_z)_{assoc}$ (kip-ft)	$\left[\dfrac{\left(P_{max}^{T}\right)_{CSi}}{\left(P_{max}^{T}\right)_{CT}}\right]\left[\left(M_z^{T}\right)_{assoc}\right]_{CT}$	$\left[\dfrac{\left(P_{max}^{L}\right)_{CSi}}{\left(P_{max}^{L}\right)_{CT}}\right]\left[\left(M_z^{L}\right)_{assoc}\right]_{CT}$	$\left[\dfrac{\left(P_{max}^{P}\right)_{CSi}}{\left(P_{max}^{P}\right)_{CT}}\right]\left[\left(M_z^{P}\right)_{assoc}\right]_{CT}$

- Determine factored shear and associated factored moment for Strength I, Strength II, Strength III, and Strength V Limit States.
- Design for shear for controlling case as per AASHTO 5.7.3. See the Design Example in Section 5.7.10 for a detailed shear design.

5.7.9 COLUMN SEISMIC DESIGN PROCEDURE

Column seismic design and details shall follow BDP Chapter 20 and the Caltrans Seismic Design Criteria 2.0.

5.7.10 DESIGN EXAMPLE

The following design example is for the strength design of the left column of Bent 2. The bridge shown in Figure 5.7.10.1-1 and Figure 5.7.10.1-3 is a three-span PS/CIP box girder bridge with 20º skew and two column bents. The superstructure depth is 6.75 ft. Columns' heights from the top of the footing to the superstructure soffit are 44 ft at Bent 2 and 47 ft at Bent 3. The columns are round with a diameter of 6 ft and a pinned connection at the footing. The centerline distance between columns is 34 ft.

5.7.10.1 Column 1 at Bent 2 Design Example Plans

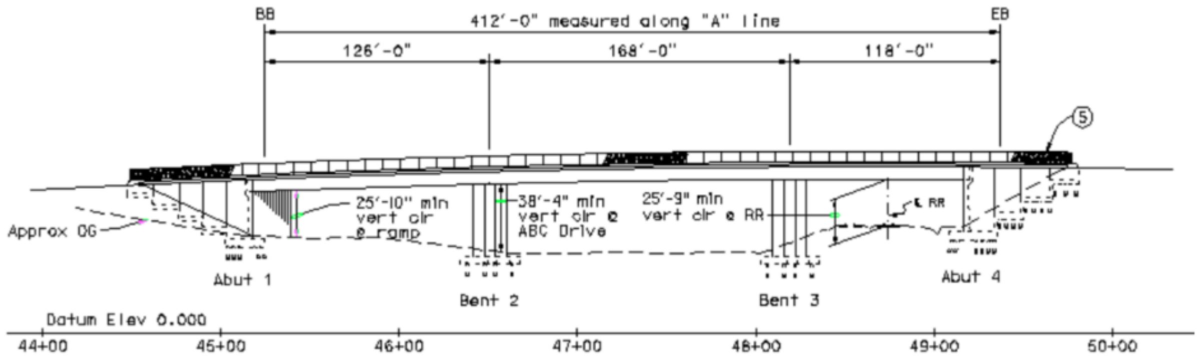


Figure 5.7.10.1-1 Elevation View of Example Bridge

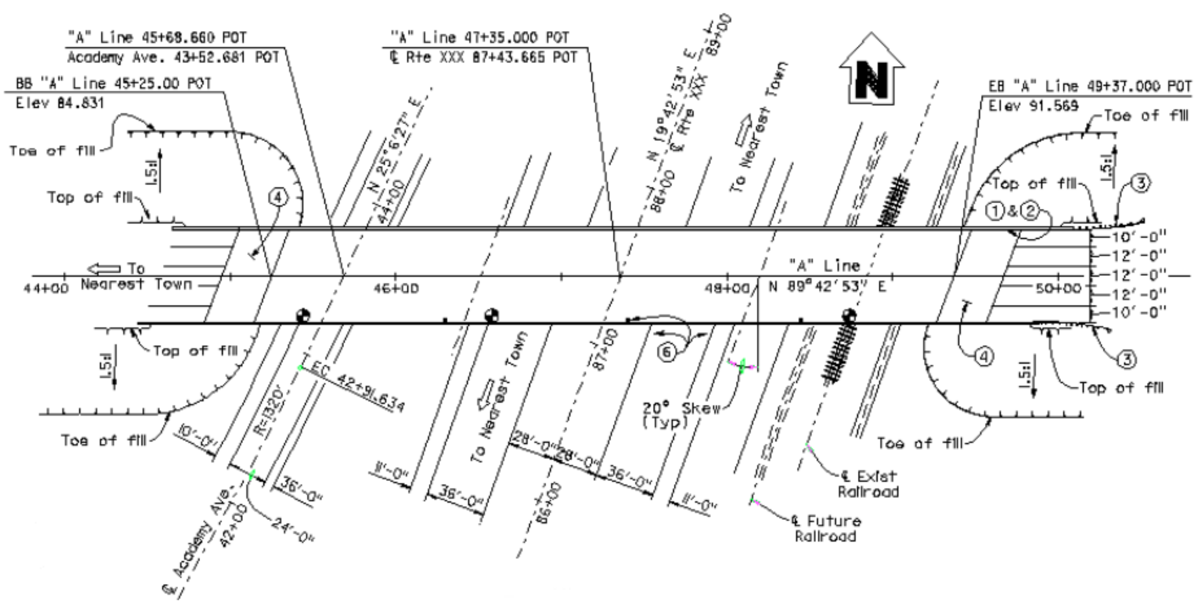


Figure 5.7.10.1-2 Plan View of Example Bridge

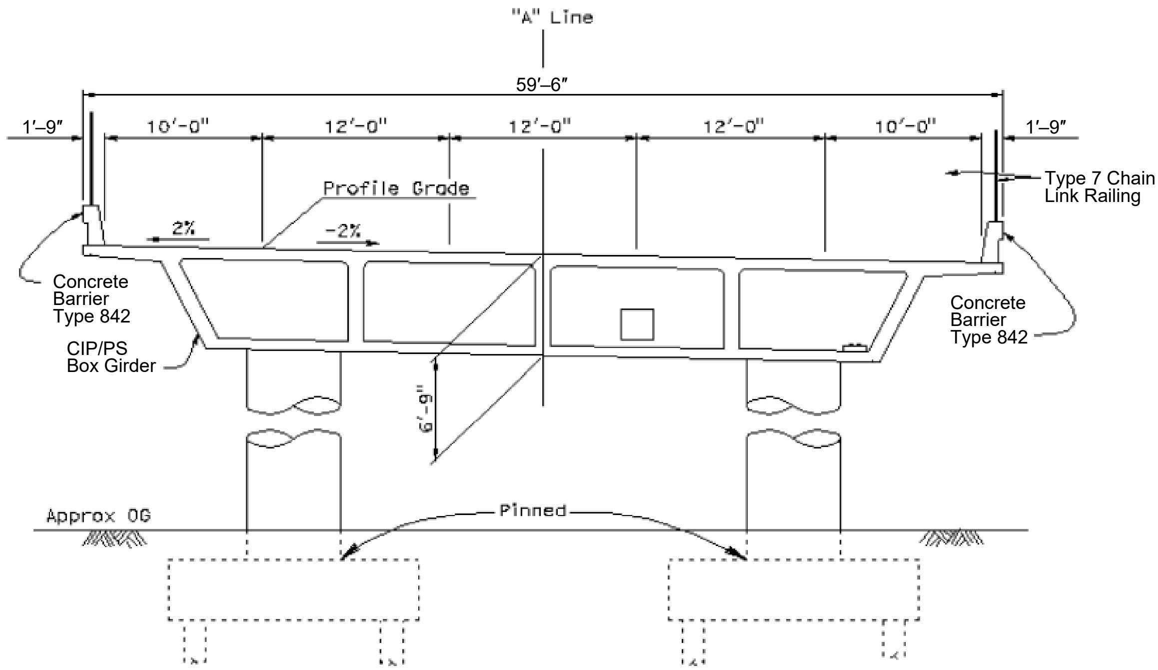


Figure 5.7.10.1-3 Typical Section of Example Bridge

5.7.10.2 Flexural Check of Main Column Reinforcement (A_s)

5.7.10.2.1 Longitudinal Analysis

From CTBridge output, determine M_z for Dead Load (DC) and Additional Dead Load (DW).

Table 5.7.10.2-1 Dead Load Unfactored Column Forces

Dead Load - Unfactored Column Forces - Final

Bent 2, Column 1

Location ft	AX* kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	-1577.8	24.3	7.8	0.0	0.0	-0.0
11.00	-1531.1	24.3	7.8	0.0	85.6	-267.7
22.00	-1484.5	24.3	7.8	0.0	171.3	-535.4
33.00	-14378	24.3	7.8	0.0	256.9	-803.2
44.00	-1391.2	24.3	7.8	0.0	342.5	$\boxed{-1070.9}$

* DC axial load from CTBridge doesn't incorporate the weight of the bent cap. Use axial load from the transverse analysis in CSiBridge.

Table 5.7.10.2-2 Additional Dead Load Unfactored Column Forces

Additional Dead Load - Unfactored Column Forces						
			Bent 2, Column 1			
Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	-158.7	3.10	1.0	0.0	0.0	-0.0
11.00	-158.7	3.10	1.0	0.0	11.0	-34.4
22.00	-158.7	3.10	1.0	0.0	22.0	-68.8
33.00	-158.7	3.10	1.0	0.0	33.0	-103.2
44.00	-158.7	3.10	1.0	0.0	44.0	-137.5

Controlling moments, M_z, are as follows:

$$M_z^{DC} = -1{,}070.9 \text{ kip} - \text{ft}$$

$$M_z^{DW} = -137.5 \text{ kip} - \text{ft}$$

5.7.10.2.2 HL-93 Vehicular Live Loads

From CTBridge output, determine bent two unfactored reactions for one lane (no dynamic load allowance factors) for the design vehicle as:

- Maximum A_x and associated M_z at top of the column
- Maximum M_z and associated A_x at top of the column

Table 5.7.10.2-3 HL-93 Live Load, Controlling Unfactored Bent Reactions

Live Load - Controlling Unfactored Bent Reactions

Bent 2 Reactions - LRFD Design Vehicle

No Dynamic Load Allowance - Single Lane

Location ft	Primary DOF	T / L	AX kip	VY kip	VZ kip	MY kip-ft	MZ kip-ft
Col Tops	AX-	Truck	-113.28	2.06	0.72	31.54	-90.56
		Lane	-98.22	4.21	1.47	64.48	-185.08
Col Tops	AX+	Truck	5.38	-4.01	-1.40	-61.63	176.63
		Lane	3.73	-2.79	-0.97	-42.84	122.78
Col Tops	MY-	Truck	-44.36	-7.45	-2.60	-114.37	327.85
		Lane	-41.44	-5.35	-1.87	-82.09	235.26
Col Tops	MY+	Truck	-71.07	10.91	3.80	167.39	-480.21
		Lane	-53.42	7.04	2.45	108.00	-309.84
Col Tops	MZ-	Truck	-71.07	10.91	3.80	167.39	-480.21
		Lane	-53.42	7.04	2.45	108.00	-309.84
Col Tops	MZ+	Truck	-44.36	-7.45	-2.60	-114.37	327.85
		Lane	-41.44	-5.35	-1.87	-82.09	235.26

From the CTBridge output, determine unfactored reactions at Bent 2 for one lane (no dynamic load allowance factors) of P15 vehicle load as follows:

- Maximum A_x and associated M_z at top of the column
- Maximum M_z and associated A_x at top of the column

Table 5.7.10.2-4 P15 Live Load, Controlling Unfactored Bent Reactions

Live Load - Controlling Unfactored Bent Reactions

Bent 2 Reactions - LRFD Permit Vehicle

No Dynamic Load Allowance - Single Lane

Location ft	Primary DOF	T / L	AX kip	VY kip	VZ kip	MY kip-ft	MZ kip-ft
Col Tops	AX-	Truck	-356.85	6.88	2.40	105.44	302.65
Col Tops	AX+	Truck	17.57	-13.14	-4.59	-201.80	578.29
Col Tops	MY-	Truck	-234.13	-14.92	-5.21	-229.04	656.50
Col Tops	MY+	Truck	-222.39	34.55	12.04	529.87	-1520.13
Col Tops	MZ-	Truck	-222.39	34.55	12.04	529.87	-1520.13
Col Tops	MZ+	Truck	-234.13	-14.92	-5.21	-229.04	656.50

5.7.10.2.3 Transverse Analysis

The pseudo wheel loads used in the transverse analysis are as follows:

$$\text{Design Pseudo Wheel Load} = 1.33\left(\frac{113.28}{2}\right) + 1.00\left(\frac{98.22}{2}\right) = 124.44 \text{ kips}$$

$$\text{Permit Pseudo Wheel Load} = 1.25\left(\frac{356.85}{2}\right) = 223.03 \text{ kips}$$

From CSiBridge output, determine the axial loads and transverse moments for DC and DW.

Table 5.7.10.2-5 Axial loads and Transverse Moment for DC and DW

TABLE: Element Forces – Frames									
Frame	**Station**	**Output Case**	**Step Type**	**P ***	**V2**	**V3**	**T**	**M2**	**M3**
Text	ft	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
1	0	DC		-1693.6	-7.4	0	0	0	0
1	11	DC		-1646.9	-7.4	0	0	0	81.9
1	11	DC		-1646.9	-7.4	0	0	0	81.9
1	22	DC		-1600.3	-7.4	0	0	0	163.8
1	22	DC		-1600.3	-7.4	0	0	0	163.8
1	33	DC		-1553.7	-7.4	0	0	0	245.7
1	33	DC		-1553.7	-7.4	0	0	0	245.7
1	44	DC		-1507.0	-7.4	0	0	0	327.6
1	0	DW		-161.4	-0.6	0	0	0	0
1	11	DW		-161.4	-0.6	0	0	0	7.1
1	11	DW		-161.4	-0.6	0	0	0	7.1
1	22	DW		-161.4	-0.6	0	0	0	14.2
1	22	DW		-161.4	-0.6	0	0	0	14.2
1	33	DW		-161.4	-0.6	0	0	0	21.3
1	33	DW		-161.4	-0.6	0	0	0	21.3
1	44	DW		-161.4	-0.6	0	0	0	28.4

Table 5.7.10.2-6 WinYIELD Column Dead and Prestressing Load Input

Vehicle	DC Dead Load	DW Added Dead Load
M_y - Trans (kip-ft)	328	28
M_x - Long (kip-ft)	-1071	-138
P - Axial (kip)	1507	161

5.7.10.2.4 HL-93 Vehicular Live Loads

From CSiBridge output, determine the unfactored column reactions for the HL-93 vehicle, including the dynamic load allowance factors, which are:

- Maximum *P* and associated M_3
- Maximum M_3 and associated *P*

Table 5.7.10.2-7 Maximum Axial Load (*P*) for HL-93 Vehicle

TABLE: Element Forces – Frames									
Frame	**Station**	**Output Case**	**Step Type**	**P**	**V2**	**V3**	**T**	**M2**	**M3**
Text	ft	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
1	0	HLL	Max P	55.7	5.3	0	0	0	0
1	11	HLL	Max P	55.7	5.3	0	0	0	-57.8
1	11	HLL	Max P	55.7	5.3	0	0	0	-57.8
1	22	HLL	Max P	55.7	5.3	0	0	0	-115.6
1	22	HLL	Max P	55.7	5.3	0	0	0	-115.6
1	33	HLL	Max P	55.7	5.3	0	0	0	-173.3
1	33	HLL	Max P	55.7	5.3	0	0	0	-173.3
1	44	HLL	Max P	55.7	5.3	0	0	0	-231.1
1	0	HLL	Min P	-563.6	-2.8	0	0	0	0
1	11	HLL	Min P	-563.6	-2.8	0	0	0	30.4
1	11	HLL	Min P	-563.6	-2.8	0	0	0	30.4
1	22	HLL	Min P	-563.6	-2.8	0	0	0	60.9
1	22	HLL	Min P	-563.6	-2.8	0	0	0	60.9
1	33	HLL	Min P	-563.6	-2.8	0	0	0	91.3
1	33	HLL	Min P	-563.6	-2.8	0	0	0	91.3
1	44	HLL	Min P	-563.6	-2.8	0	0	0	121.8

Table 5.7.10.2-8 Maximum Transverse Moment (M_3) for HL-93 Vehicle

TABLE: Element Forces – Frames									
Frame	**Station**	**Output Case**	**Step Type**	**P**	**V2**	**V3**	**T**	**M2**	**M3**
Text	ft	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
1	0	HLL	Max M3	-367.8	-12.8	0	0	0	0
1	11	HLL	Max M3	-367.8	-12.8	0	0	0	140.5
1	11	HLL	Max M3	-367.8	-12.8	0	0	0	140.5
1	22	HLL	Max M3	-367.8	-12.8	0	0	0	280.9
1	22	HLL	Max M3	-367.8	-12.8	0	0	0	280.9
1	33	HLL	Max M3	-367.8	-12.8	0	0	0	421.4
1	33	HLL	Max M3	-367.8	-12.8	0	0	0	421.4
1	44	HLL	Max M3	-367.8	-12.8	0	0	0	561.8
1	0	HLL	Min M3	-248.9	5.3	0	0	0	0
1	11	HLL	Min M3	-248.9	5.3	0	0	0	-96.3
1	11	HLL	Min M3	-248.9	5.3	0	0	0	-96.3
1	22	HLL	Min M3	-248.9	5.3	0	0	0	-192.6
1	22	HLL	Min M3	-248.9	5.3	0	0	0	-192.6
1	33	HLL	Min M3	-248.9	5.3	0	0	0	-288.9
1	33	HLL	Min M3	-248.9	5.3	0	0	0	-288.9
1	44	HLL	Min M3	-248.9	5.3	0	0	0	-385.2

From CSiBridge output, determine the unfactored column reactions for the P15 vehicle including the dynamic load allowance factors which are:

- Maximum P and associated M_3
- Maximum M_3 and associated P

Table 5.7.10.2-9 Maximum Axial Load (*P*) for P15 Vehicle

TABLE: Element Forces – Frames									
Frame	**Station**	**Output Case**	**Step Type**	**P**	**V2**	**V3**	**T**	**M2**	**M3**
Text	ft	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
1	0	PLL	Max P	83.2	7.8	0	0	0	0
1	11	PLL	Max P	83.2	7.8	0	0	0	-86.3
1	11	PLL	Max P	83.2	7.8	0	0	0	-86.3
1	22	PLL	Max P	83.2	7.8	0	0	0	-172.6
1	22	PLL	Max P	83.2	7.8	0	0	0	-172.6
1	33	PLL	Max P	83.2	7.8	0	0	0	-258.9
1	33	PLL	Max P	83.2	7.8	0	0	0	-258.9
1	44	PLL	Max P	83.2	7.8	0	0	0	-345.2
1	0	PLL	Min P	-925.4	4.0	0	0	0	0
1	11	PLL	Min P	-925.4	4.0	0	0	0	-44.0
1	11	PLL	Min P	-925.4	4.0	0	0	0	-44.0
1	22	PLL	Min P	-925.4	4.0	0	0	0	-88.1
1	22	PLL	Min P	-925.4	4.0	0	0	0	-88.1
1	33	PLL	Min P	-925.4	4.0	0	0	0	-132.1
1	33	PLL	Min P	-925.4	4.0	0	0	0	-132.1
1	44	PLL	Min P	-925.4	4.0	0	0	0	-176.1

Table 5.7.10.2-10 Maximum Transverse Moment (M_3) for P15 Vehicle

TABLE: Element Forces – Frames									
Frame	**Station**	**Output Case**	**Step Type**	**P**	**V2**	**V3**	**T**	**M2**	**M3**
Text	ft	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
1	0	PLL	Max M3	-446.1	-18.5	0	0	0	0
1	11	PLL	Max M3	-446.1	-18.5	0	0	0	203.3
1	11	PLL	Max M3	-446.1	-18.5	0	0	0	203.3
1	22	PLL	Max M3	-446.1	-18.5	0	0	0	406.5
1	22	PLL	Max M3	-446.1	-18.5	0	0	0	406.5
1	33	PLL	Max M3	-446.1	-18.5	0	0	0	609.8
1	33	PLL	Max M3	-446.1	-18.5	0	0	0	609.8
1	44	PLL	Max M3	-446.1	-18.5	0	0	0	813.0
1	0	PLL	Min M3	-446.1	15.7	0	0	0	0
1	11	PLL	Min M3	-446.1	15.7	0	0	0	-172.6
1	11	PLL	Min M3	-446.1	15.7	0	0	0	-172.6
1	22	PLL	Min M3	-446.1	15.7	0	0	0	-345.2
1	22	PLL	Min M3	-446.1	15.7	0	0	0	-345.2
1	33	PLL	Min M3	-446.1	15.7	0	0	0	-517.8
1	33	PLL	Min M3	-446.1	15.7	0	0	0	-517.8
1	44	PLL	Min M3	-446.1	15.7	0	0	0	-690.4

5.7.10.2.5 Output from Longitudinal Analysis (CTBridge)

Column unfactored live load forces and moments for one lane from longitudinal analysis (CTBridge) are presented in Table 5.7.10.2-11.

Table 5.7.10.2-11 Unfactored Bent Reactions for One Lane, Dynamic Load Allowance Factors Not Included

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum axial load and associated longitudinal moment		Maximum axial load and associated longitudinal moment	
Force	A_x (kip)	M_z (kip-ft)	A_x (kip)	M_z (kip-ft)
Truck	-113.3	-90.6	-356.9	-302.7
Lane	-98.2	-185.1	--	--
Case	Maximum longitudinal moment and associated axial load		Maximum longitudinal moment and associated axial load	
Force	A_x (kip)	M_z (kip-ft)	A_x (kip)	M_z (kip-ft)
Truck	-71.1	-480.2	-222.4	-1520.1
Lane	-53.4	-309.8	--	--

5.7.10.2.6 Output from Transverse Analysis (CSiBridge)

Two pseudo wheel loads, including dynamic allowance factors, are used in the transverse analysis model in CSiBridge (see Section 5.7.7.3.2).

The results from the transverse analysis are presented in Table 5.7.10.2-12.

Table 5.7.10.2-12 Unfactored Column Reaction, Including Dynamic Load Allowance Factors

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum axial load and associated transverse moment		Maximum axial load and associated transverse moment	
Force	P (kip)	M_3 (kip-ft)	P (kip)	M_3 (kip-ft)
Truck + Lane	-563.6	121.8	-925.4	-176.1
Case	Maximum transverse moment and associated axial load		Maximum transverse moment and associated axial load	
Force	P (kip)	M_3 (kip-ft)	P (kip)	M_3 (kip-ft)
Truck + Lane	-367.8	561.8	-446.1	813.0

5.7.10.2.7 Unfactored Column Reactions for One Lane, Including Impact (CTBridge)

Multiply dynamic allowance factors by values in Table 5.7.10.2-11 and divide by the number of columns to calculate the longitudinal column reactions (Table 5.7.10.2-13).

Table 5.7.10.2-13 Unfactored Column Reactions for One Lane, Including Dynamic Load Allowance Factors (CTBridge)

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum axial load and associated longitudinal moment		Maximum axial load and associated longitudinal moment	
Force	A_x (kip)	M_z (kip-ft)	A_x (kip)	M_z (kip-ft)
Truck	-75	-60	-223	-189
Lane	-49	-93	--	--
Case	Maximum longitudinal moment and associated axial load		Maximum longitudinal moment and associated axial load	
Force	A_x (kip)	M_z (kip-ft)	A_x (kip)	M_z (kip-ft)
Truck	-47	-319	-139	-950
Lane	-27	-155	--	--

5.7.10.2.8 Unfactored Column Reactions, Including Dynamic Load Allowance Factors (CSiBridge)

Separate the HL-93 vehicle reaction results from the transverse analysis (Section 5.7.7.3.4) into truck and lane loads as follows:

$$R1 = \frac{75}{75 + 49} = 0.61$$

$$R2 = \frac{49}{75 + 49} = 0.39$$

$$\text{Truck load of Design Vehicle} = 0.61 \times (\text{values of Table 5.7.10.2-12})$$

$$\text{Lane load of Design Vehicle} = 0.39 \times (\text{values of Table 5.7.10.2-12})$$

Table 5.7.10.2-14 summarizes the truck and lane loads for both HL-93 and P15 vehicles of transverse analysis.

Table 5.7.10.2-14 Unfactored Column Reactions, Including Dynamic Load Allowance Factors (CSiBridge)

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum axial load and associated transverse moment		Maximum axial load and associated transverse moment	
Force	A_x (kip)	M_3 (kip-ft)	A_x (kip)	M_3 (kip-ft)
Truck	-344	74	-925	-176
Lane	-220	48	--	--
Case	Maximum transverse moment and associated axial load		Maximum transverse moment and associated axial load	
Force	A_x (kip)	M_3 (kip-ft)	A_x (kip)	M_3 (kip-ft)
Truck	-224	343	-446	813
Lane	-143	219	--	--

Combine load results as shown in Table 5.7.7.3-5, Table 5.7.7.3-6, Table 5.7.7.3-7, and Table 5.7.7.3-8 to get WinYIELD input loads as shown in Table 5.7.10.2-15.

Table 5.7.10.2-15 WinYIELD Column Live Load Input

	Case-1 Max Transverse - My			Case-2 Max Longitudinal - Mx			Case-3 Max Axial - P		
Vehicle	P-15 Truck PL+IM	HL-93 Truck HL+IM	HL-93 Lane Load LL	P-15 Truck PL+IM	HL-93 Truck HL+IM	HL-93 Lane Load LL	P-15 Truck PL+IM	HL-93 Truck HL+IM	HL-93 Lane Load LL
M_y - Trans (kip-ft)	813	343	219	-110	46	26	-176	74	48
M_x - Long (kip-ft)	-378	-179	-271	-3941	-1463	-696	-784	-275	-418
P - Axial (kip)	446	224	143	577	216	121	925	344	220

5.7.10.2.9 Wind Load (WS, WL)

- Wind on Structure (WS) - Horizontal Wind Load:

$$Z = \text{Max Column Height} + \text{Structure Depth} + \text{Barrier} - \text{Cover}$$
$$= 47 + 6.75 + 3.5 - 3.0 = 54.25 \text{ ft}$$

$\text{Ground Surface Roughness} = \text{C}(\text{"Open Terrain"})$ (AASHTO 3.8.1.1.4)

$\text{Wind Exposure Category} = \text{C}$ (AASHTO 3.8.1.1.5)

Calculate the Strength III wind load:

$V_{Str\ III} = 110$ mph (AASHTO Fig 3.8.1.1.2-1)

$K_z = 1.12$ (AASHTO Table C3.8.1.2.1-1)

$G_{Str\ III} = 1.0$ (AASHTO Table 3.8.1.2.1-1)

$C_{D,super} = 1.3$ *("Box-Girder")* (AASHTO Table 3.8.1.2.1-2)

$C_{D,sub} = 1.6$ ("Substructure") (AASHTO Table 3.8.1.2.1-2)

$$P_{z,super} = 2.56x10^{-6} V^2 K_z G C_D$$ (AASHTO Eq 3.8.1.2.1-1)

$$= 2.56\text{x}10^{-6} (110)^2 (1.12)(1.0)(1.3) = 0.045 \text{ ksf}$$

$$P_{z,sub} = 2.56x10^{-6} V^2 K_z G C_D$$

$$= 2.56\text{x}10^{-6} (110)^2 (1.12)(1.0)(1.6) = 0.056 \text{ ksf}$$

Calculate the Strength V wind load:

$V_{Str\ V} = 80$mph (AASHTO Table 3.8.1.1.2-1)

$K_z = 1.00$ (AASHTO Table C3.8.1.2.1-1)

$G_{Str\ V} = 1.0$ (AASHTO Table 3.8.1.2.1-1)

$C_{D,super} = 1.3$ *("Box-Girder")* (AASHTO Table 3.8.1.2.1-2)

$C_{D,sub} = 1.6$ ("Substructure") (AASHTO Table 3.8.1.2.1-2)

$$P_{z,super} = 2.56x10^{-6} V^2 K_z G C_D$$ (AASHTO Eq 3.8.1.2.1-1)

$$= 2.56\text{x}10^{-6} (80)^2 (1.00)(1.0)(1.3) = 0.021 \text{ ksf}$$

$$P_{z,sub} = 2.56x10^{-6} V^2 K_z G C_D$$

$$= 2.56\text{x}10^{-6} (80)^2 (1.00)(1.0)(1.6) = 0.026 \text{ ksf}$$

The design wind pressure from the superstructure, $P_{z,super}$, is combined with skew coefficients to determine wind loading for various angles of attack. The skew coefficient for wind loading from the superstructure is found in AASHTO Table 3.8.1.2.3a-1. Note that the terms "windward" and "leeward" apply only to superstructure types that have two or more separate unobstructed structural elements, such as trusses and arches. The term "column" used within AAHSTO 3.8.1.2.1 refers to superstructure columns such as "spandrel columns" and is NOT used to refer to bridge substructure elements.

Table 5.7.10.2-16 and Table 5.7.10.2-17 list the design wind pressure from the superstructure for various angles of attack for the Strength III and Strength V load combinations, respectively.

Table 5.7.10.2-16 Strength III Wind Pressure at Various Angles of Attack

Angle of Attack (degree)	Superstructure	
	Transverse Wind Pressure ($P_{z,trans}$) (ksf)	Longitudinal Wind Pressure ($P_{z,long}$) (ksf)
0	0.045	0.000
15	0.040	0.005
30	0.037	0.011
45	0.030	0.014
60	0.015	0.017

Table 5.7.10.2-17 Strength V Wind Pressure at Various Angles of Attack

Angle of Attack (degree)	Superstructure	
	Transverse Wind Pressure ($P_{z,trans}$) (ksf)	Longitudinal Wind Pressure ($P_{z,long}$) (ksf)
0	0.021	0.000
15	0.018	0.003
30	0.017	0.005
45	0.014	0.007
60	0.007	0.008

For wind loads applied directly to the substructure, the wind pressures are resolved into components relative to the end and front elevation of the column. The wind load components act perpendicular to the projected column widths, as shown in Figure 5.7.10.2-1.

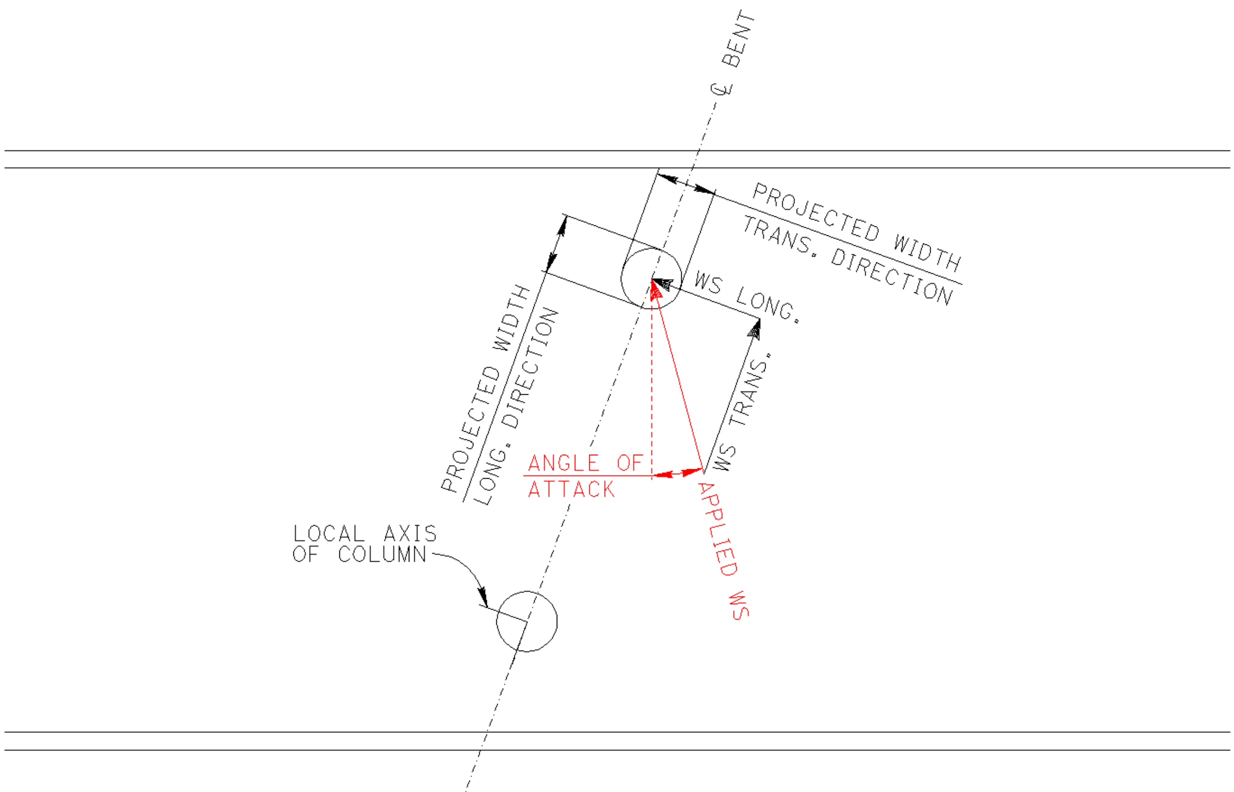


Figure 5.7.10.2-1 Wind Load Applied Directly to the Substructure

Calculate the uniformly distributed wind load applied to the superstructure and substructure based on each element's depth or width. In this example, the projected width of the substructure is the same in both the transverse and longitudinal direction. For non-symmetric sections, the projected widths may need to be determined for each direction.

It should be noted that the dead load and live load column moment demands are positive in the MY direction and negative in the MZ direction. Since the wind loads can be applied at any angle of attack, the wind loads should be input into CTBridge so that they are additive with the dead and live loads. Applying wind loads in the wrong direction can result in reduced column demands. For this reason, multiple models must be run to determine the controlling column loads.

For this design example, the wind load acting on the fence was ignored.

- For the superstructure:

$$F_{z,trans} = \left(D_{super} + D_{barrier}\right)P_{z,trans} = (6.75+3.50)P_{z,trans} = (10.25)P_{z,trans}$$

$$F_{z,long} = \left(D_{super} + D_{barrier}\right)P_{z,long} = (6.75+3.50)P_{z,long} = (10.25)P_{z,long}$$

- For the substructure:

$$F_{z,trans} = (D_{column})P_{z,sub}\sin(skew \pm AoA) = (6.0)P_{z,sub}\sin(skew \pm AoA)$$

$$F_{z,long} = (D_{column})P_{z,sub}\cos(skew \pm AoA) = (6.0)P_{z,sub}\cos(skew \pm AoA)$$

For this example, the superstructure forces were applied:

- Transversely in the span's local Z direction (+)
- Longitudinally in the span's local X direction (-)

The substructure forces were applied:

- Transverse in the column's local Z direction (+)
- Longitudinally in the column's local Y direction (+/-)

See Figure 5.7.10.2-2, an example application of the wind load for an angle of attack of 15°.

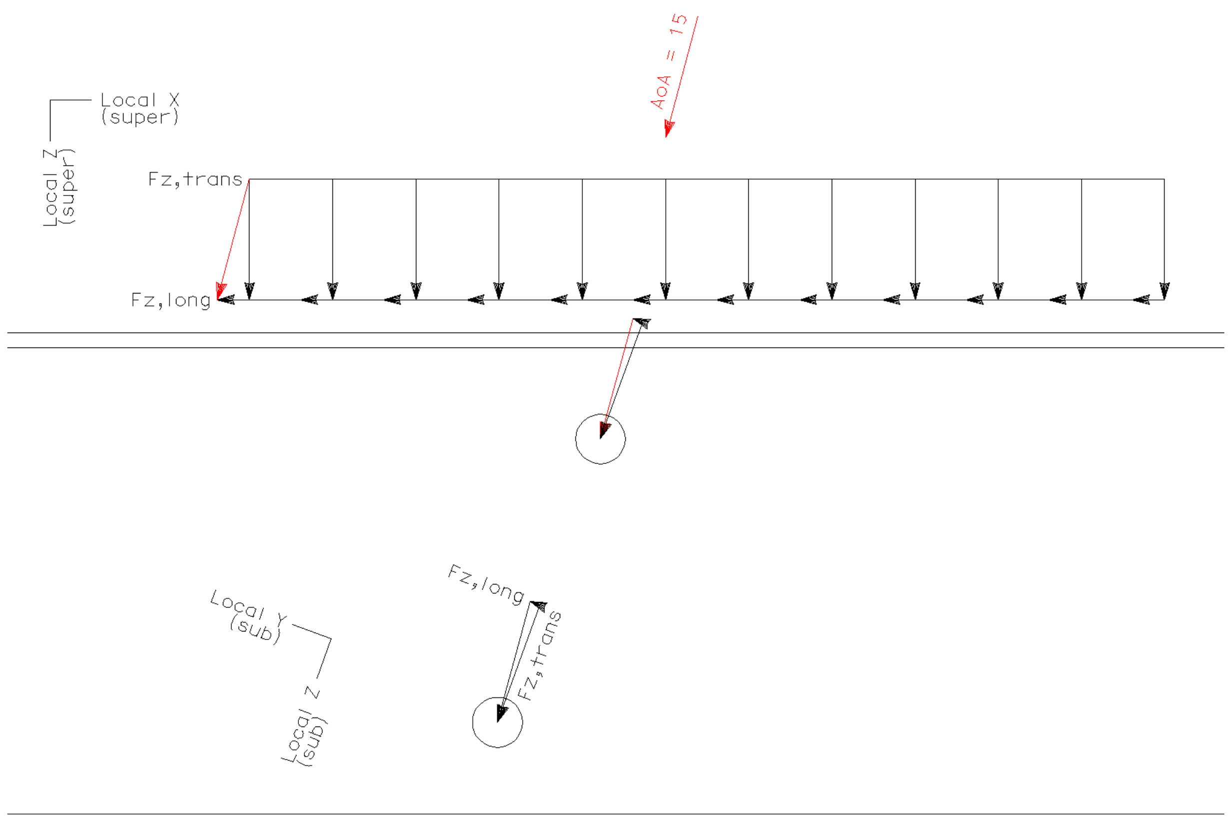


Figure 5.7.10.2-2 Wind Load Application for and Angle of Attack of 15°

Loads acting on both superstructure and substructure (columns) at various wind angles of attack are shown in Table 5.7.10.2-18, and * Forces applied to the local axis of the element, which may be different for superstructure and substructure elements.

Table 5.7.10.2-19 for the Strength III and Strength V load combinations, respectively.

Table 5.7.10.2-18 Strength III Wind Load at Various Angles of Attack

Angle of Attack (degree)	Superstructure*		Substructure*	
	Transverse Wind Force ($F_{z,trans}$) (klf)	Longitudinal Wind Force ($F_{z,long}$) (klf)	Transverse Wind Force ($F_{z,trans}$) (klf)	Longitudinal Wind Force ($F_{z,long}$) (klf)
0	0.46	-0.00	0.32	-0.11
15	0.41	-0.06	0.33	-0.03
30	0.38	-0.11	0.33	0.06
45	0.30	-0.15	0.30	0.14
60	0.16	-0.18	0.26	0.22

* Forces applied to the local axis of the element, which may be different for superstructure and substructure elements.

Table 5.7.10.2-19 Strength V Wind Load at Various Angles of Attack

Angle of Attack (degree)	Superstructure*		Substructure*	
	Transverse Wind Force ($F_{z,trans}$) (klf)	Longitudinal Wind Force ($F_{z,long}$) (klf)	Transverse Wind Force ($F_{z,trans}$) (klf)	Longitudinal Wind Force ($F_{z,long}$) (klf)
0	0.22	-0.00	0.15	-0.05
15	0.18	-0.03	0.16	-0.01
30	0.17	-0.05	0.15	0.03
45	0.14	-0.07	0.14	0.07
60	0.07	-0.08	0.12	0.10

* Forces applied to the local axis of the element, which may be different for superstructure and substructure elements.

- Wind on Structure (WS) - Vertical Wind Load (Strength III)

  Apply a 0.020 ksf pressure acting upwards on the full superstructure width (including sidewalks and barriers). The force should be applied as a line load acting at the windward quarter-point of the deck width and only applied when the direction of the horizontal wind is taken perpendicular to the longitudinal axis of the bridge. The vertical wind is only applied to the Service IV and Strength III load combinations. For the Strength III combination:

$$F_V = P_V W_D$$
$$= (0.020)(59.5) = 1.19 \text{ klf}$$

This force is applied to the windward quarter-point of the deck. The resulting moment applied to the spline model:

$$
\begin{aligned}
M_V &= 0.25 F_V W_D \\
&= 0.25(1.19)(59.5) = 17.7 \text{ kip-ft/ft}
\end{aligned}
$$

Model wind load as a user defined load set in CTBridge as shown in Figure 5.7.10.2-3.

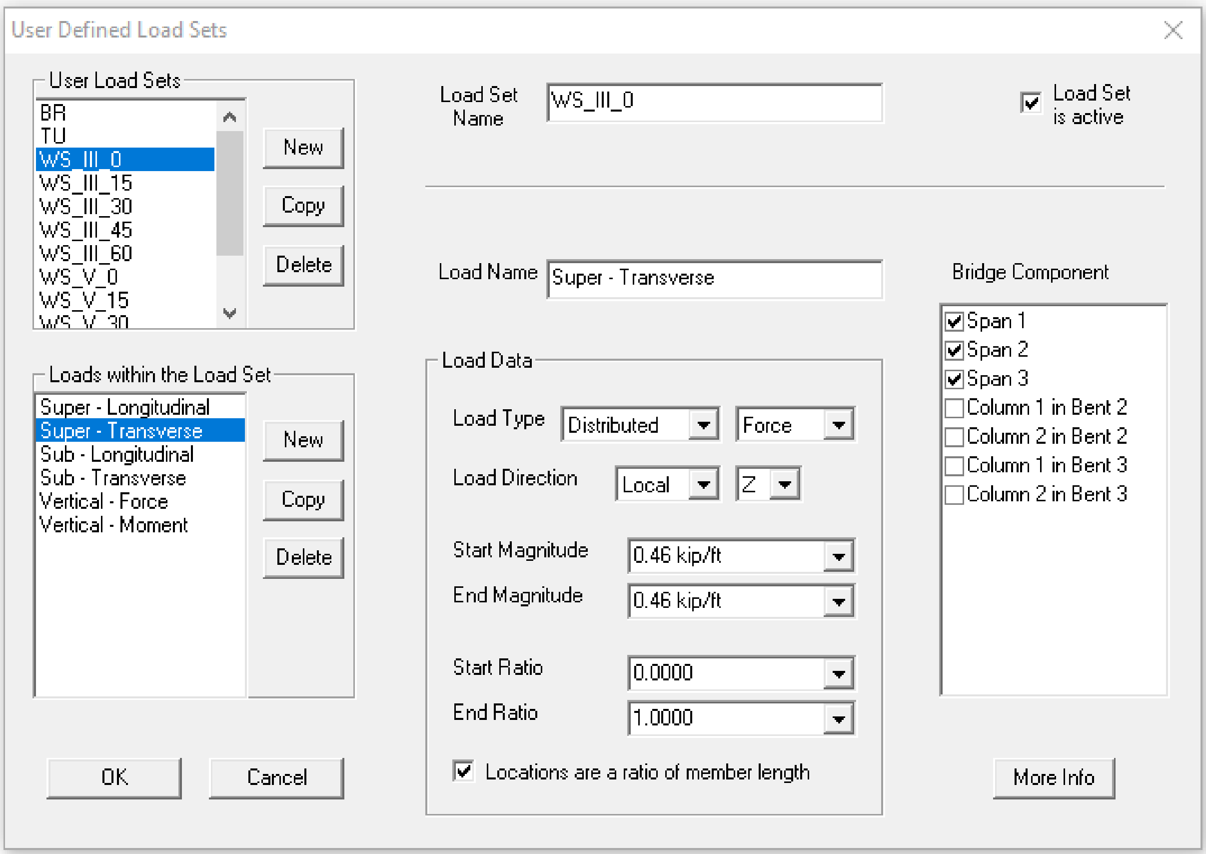


Figure 5.7.10.2-3 User Defined Loads for Wind Loads

It should be noted that CTBridge defines a "roller" support condition as having translational fixity in the direction transverse to the bridge layout line, while a "slider" support is free from translational restraint. If a "roller" is used when modeling the abutments, most of the wind load will be resisted by the abutments instead of the columns. For this reason, it is recommended that the abutments be modeled as "sliders", or that the engineer assigns a nominal bearing stiffness so that the columns become the main lateral force resisting system.

The results from the Strength III wind load analysis are as follows:

Table 5.7.10.2-20 User Loads, Unfactored Column Forces, WS, Strength III, Angle of Attack 0˚

User Loads - Unfactored Column Forces - WS_III_0

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	342.5	-24.2	60.3	0.0	0.0	0.0
11.00	342.5	-23.0	56.7	0.0	643.5	260.0
22.00	342.5	-21.8	53.2	0.0	1248.3	506.7
33.00	342.5	-20.6	49.7	0.0	1814.4	740.0
44.00	342.5	-19.4	46.2	0.0	2341.8	960.1

Table 5.7.10.2-21 User Loads, Unfactored Column Forces, WS, Strength III, Angle of Attack 15˚

User Loads - Unfactored Column Forces - WS_III_15

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	149.3	-10.4	58.6	0.0	0.0	0.0
11.00	149.3	-10.1	55.0		624.7	112.7
22.00	149.3	-9.8	51.3		1209.4	221.8
33.00	149.3	-9.4	47.7		1754.3	327.3
44.00	149.3	-9.1	44.1		2259.2	429.1

Table 5.7.10.2-22 User Loads, Unfactored Column Forces, WS, Strength III, Angle of Attack 30˚

User Loads - Unfactored Column Forces - WS_III_30

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	139.1	0.2	57.6	0.0	0.0	0.0
11.00	139.1	-0.5	53.9	0.0	613.1	1.7
22.00	139.1	-1.1	50.3	0.0	1186.3	10.7
33.00	139.1	-1.8	46.7	0.0	1719.5	27.0
44.00	139.1	-2.5	43.0	0.0	2212.8	50.5

Table 5.7.10.2-23 User Loads, Unfactored Column Forces, WS, Strength III, Angle of Attack 45˚

User Loads - Unfactored Column Forces - WS_III_45

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	111.4	11.1	49.7	0.0	0.0	0.0
11.00	111.4	9.6	46.4	0.0	528.5	-113.7
22.00	111.4	8.0	43.1	0.0	1020.6	-210.4
33.00	111.4	6.5	39.8	0.0	1476.5	-290.3
44.00	111.4	4.9	36.5	0.0	1896.0	-353.1

Table 5.7.10.2-24 User Loads, Unfactored Column Forces, WS, Strength III, Angle of Attack 60˚

User Loads - Unfactored Column Forces - WS_III_60

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	64.1	23.2	34.9	0.0	0.0	0.0
11.00	64.1	20.8	32.0	0.0	368.2	-242.2
22.00	64.1	18.4	29.2	0.0	705.0	-457.8
33.00	64.1	16.0	26.3	0.0	1010.3	-646.8
44.00	64.1	13.6	23.5	0.0	1284.2	-809.2

From CTBridge output for this example:

- The maximum transverse moment for wind on structure occurs at a wind direction with an angle of attack = 0˚
- The maximum longitudinal moment for wind on structure occurs at a wind direction with an angle of attack = 60˚ (the value of MZ is larger for angle of attack = 0˚, but is acting in the opposite direction of the dead load, resulting in a reduced load combination)

Similarly, the controlling wind loads for the Strength V load combination is as follows:

Table 5.7.10.2-25 User Loads, Unfactored Column Forces, WS, Strength V, Angle of Attack 0˚

User Loads - Unfactored Column Forces - WS_V_0

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	78.7	-10.6	29.0	0.0	0.0	0.0
11.00	78.7	-10.0	27.3	0.0	309.7	113.1
22.00	78.7	-9.5	25.7	0.0	601.3	220.1
33.00	78.7	-8.9	24.0	0.0	874.7	321.0
44.00	78.7	-8.4	22.4	0.0	1130.0	415.9

Table 5.7.10.2-26 User Loads, Unfactored Column Forces, WS, Strength V, Angle of Attack 60˚

User Loads - Unfactored Column Forces - WS_V_60

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	28.4	10.5	15.6	0.0	0.0	0.0
11.00	28.4	9.4	14.3	0.0	164.3	-109.1
22.00	28.4	8.3	13.0	0.0	314.2	-206.2
33.00	28.4	7.2	11.6	0.0	449.5	-291.1
44.00	28.4	6.1	10.3	0.0	570.3	-363.9

- Wind on Live Load (WL)
  Apply 0.10 k/ft acting at various angles (AASHTO Table 3.8.1.3-1). This force acts at the roadway surface for multi column bents per the CA Amendment, resulting in an applied moment:

  $$M_{WL} = (0.5D_{super} + 6)F_{WL} = (3.4 + 6)F_{WL} = 9.4F_{WL}$$

Table 5.7.10.2-27 Wind on Live Load (WL) at Various Angle of Attack

Angle of Attack * (degree)	Transverse Wind on Live Force (klf)	Longitudinal Wind on Live Force (klf)	Transverse Wind on Live Moment (k-ft/ft)	Longitudinal Wind on Live Moment (k-ft/ft)
0	0.100	-0.000	0.34	0.00
15	0.088	-0.012	0.30	0.04
30	0.082	-0.024	0.28	0.08
45	0.066	-0.032	0.22	0.11
60	0.034	-0.038	0.11	0.13

* Forces applied to the local axis of the superstructure.

Using CTBridge for wind on live load, the results are:

- Case of maximum transverse wind takes place at wind direction with angle of attack = 0˚
- Case of maximum longitudinal wind takes place at wind direction with angle of attack = 60˚

Table 5.7.10.2-28 User Loads, Unfactored Column Forces, WL Trans Angle of Attack 0˚

User Loads - Unfactored Column Forces - WL_0

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	33.0	-3.7	10.2	0.0	0.0	0.0
11.00	33.0	-3.7	10.2	0.0	111.8	41.1
22.00	33.0	-3.7	10.2	0.0	223.6	82.1
33.00	33.0	-3.7	10.2	0.0	335.4	123.2
44.00	33.0	-3.7	10.2	0.0	447.2	164.2

Table 5.7.10.2-29 User Loads, Unfactored Column Forces, WL Trans Angle of Attack 60˚

User Loads - Unfactored Column Forces - WL_60

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	10.9	3.1	4.7	0.0	0.0	0.0
11.00	10.9	3.1	4.7	0.0	52.0	-33.7
22.00	10.9	3.1	4.7	0.0	103.9	-67.4
33.00	10.9	3.1	4.7	0.0	155.9	-101.0
44.00	10.9	3.1	4.7	0.0	207.8	-134.8

Table 5.7.10.2-30 Summary of Wind Loads Reactions for Column 1 at Bent 2

Force	Wind on Structure (Horizontal)		Wind on Live Load	
	Max Transverse	Max Longitudinal	Max Transverse	Max Longitudinal
Strength III				
M_y (kip-ft)	2342	1284	N/A	N/A
M_x (kip-ft)	960	-809	N/A	N/A
P (kip)	-343	-64	N/A	N/A
Strength V				
M_y (kip-ft)	1130	570	447	208
M_x (kip-ft)	416	-364	164	-135
P (kip)	-79	-28	-33	-11

5.7.10.2.10 Braking Force (BR)

The braking force (AASHTO 3.6.4) shall be taken as the greater of:

$$25\%\ \text{Design Truck} = 0.25(72) = 18.0\ \text{kips}$$

$$25\%\ \text{Design Tandem} = 0.25(50) = 12.5\ \text{kips}$$

$$5\%\ \text{Design Truck + Lane} = 0.05\,[72 + 0.64(412)] = 16.8\ \text{kips}$$

$$5\%\ \text{Design Tandem + Lane} = 0.05\,[50 + 0.64(412)] = 15.7\ \text{kips}$$

$$\text{Controlling Force} = \max\begin{cases}18.0\\12.5\\16.8\\15.7\end{cases} = 18.0\ \text{kips}$$

$$\text{Number of Lanes} = \frac{\text{Clear Width}}{12} = \frac{59.5 - 2(1.75)}{12} = 4.67 = 4\ \text{lanes}$$

$$MPF_{4lanes} = 0.65$$

$$\text{Braking Force} = (18.0)(4)(0.65) = 46.8\ \text{kips}$$

Apply the braking force longitudinally, then design for the moment and shear force effects. The braking force can be modeled in CTBridge as a user defined load in the direction of the local X direction, as shown in Figure 5.7.10.2-4:

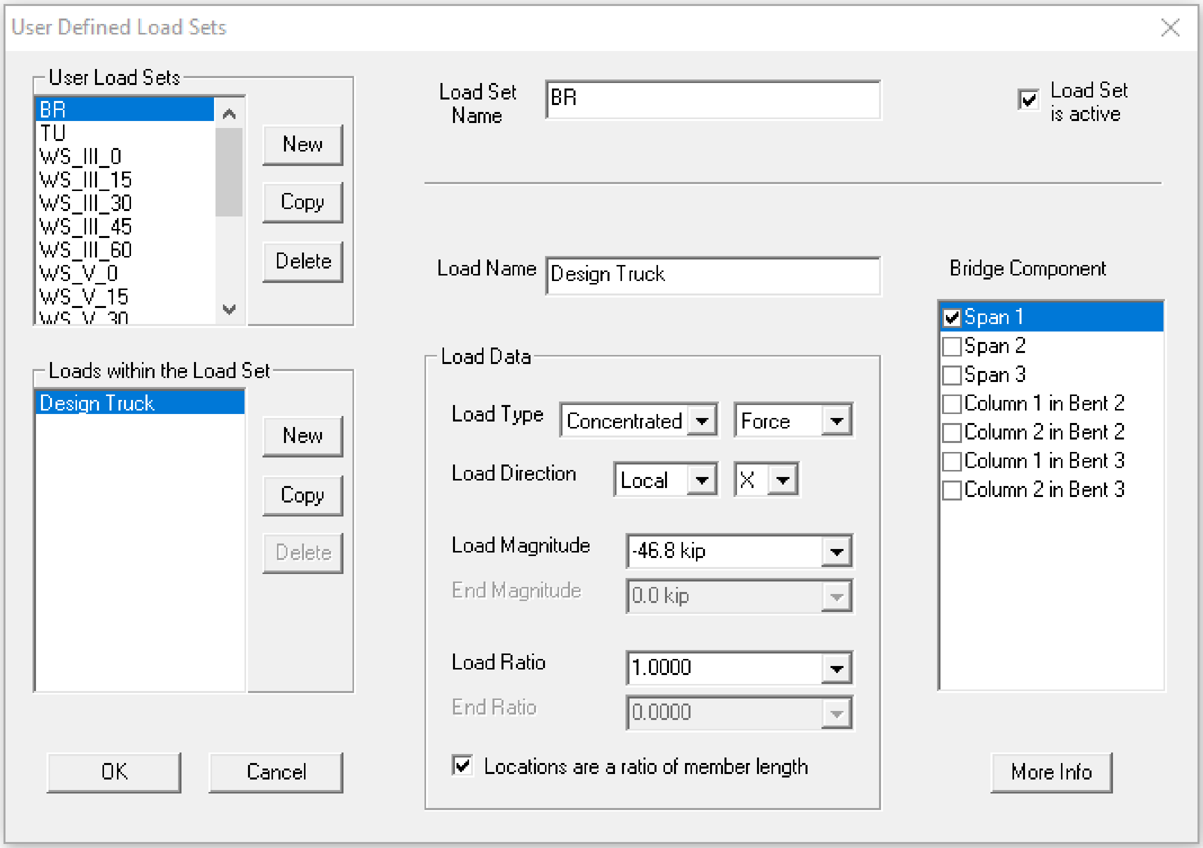


Figure 5.7.10.2-4 User Defined Loads for Braking Force

Braking forces output from CTBridge is shown in Table 5.7.10.2-31.

Table 5.7.10.2-31 User Loads, Unfactored Column Forces, Braking Force

User Loads - Unfactored Column Forces - Braking Force

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	1.8	-11.9	-4.3	0.0	0.0	0.0
11.00	1.8	-11.9	-4.3	0.0	47.8	-131.0
22.00	1.8	-11.9	-4.3	0.0	95.5	-261.9
33.00	1.8	-11.9	-4.3	0.0	143.3	-392.9
44.00	1.8	-11.9	-4.3	0.0	191.1	-523.9

5.7.10.2.11 Uniform Temperature (TU)

The uniform temperature effects are modeled in CTBridge. For a concrete bridge in moderate climate, the high and low temperature range specified in AASHTO is 80°F to 10°F, respectively. The California Amendment specifies that half of the temperature range shall be applied as a temperature rise and drop load. This results in a median temperature of:

$$T_{median} = \frac{(80-10)}{2} = 45^{\circ}\text{F}$$

The temperature force output from CTBridge is shown in Table 5.7.10.2-32 and Table 5.7.10.2-33.

Table 5.7.10.2-32 Temperature Rise, Unfactored Column Forces

Temperature Load - Unfactored Column Forces

Temperature Rise (or T1)

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	14.3	11.3	4.4	0	0.0	-0.0
11.00	14.3	11.3	4.4	0	48.9	-124.5
22.00	14.3	11.3	4.4	0	97.8	-249.0
33.00	14.3	11.3	4.4	0	146.7	-373.5
44.00	14.3	11.3	4.4	0	195.6	-498.1

Table 5.7.10.2-33 Temperature Drop, Unfactored Column Forces

Temperature Load - Unfactored Column Forces

Temperature Drop (or T2)

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	-14.3	-11.3	-4.4	0	-0.0	0.0
11.00	-14.3	-11.3	-4.4	0	-48.9	124.5
22.00	-14.3	-11.3	-4.4	0	-97.8	249.0
33.00	-14.3	-11.3	-4.4	0	-146.7	373.5
44.00	-14.3	-11.3	-4.4	0	-195.6	498.1

5.7.10.2.12 Prestress Shortening Effects (CR and SH)

The anticipated shortening due to prestressing effects occurs at a rate of 0.63 in. per 100 ft. This results in a shorting strain of:

$$\varepsilon_{csh} = \left(\frac{0.63}{100}\right)\left(\frac{1}{12}\right) = 0.000525$$

The creep and shrinkage force output from CTBridge is shown in:

Creep and Shrinkage Load - Unfactored Column Forces

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	-35.7	-28.3	-11.1	0	-0.0	0.0
11.00	-35.7	-28.3	-11.1	0	-122.2	311.3
22.00	-35.7	-28.3	-11.1	0	-244.4	622.6
33.00	-35.7	-28.3	-11.1	0	-366.7	933.8
44.00	-35.7	-28.3	-11.1	0	-488.9	1245.1

5.7.10.2.13 Prestress Secondary Effects (PS)

The secondary effect of prestressing after long term losses is shown in Table 5.7.10.2-34.

Table 5.7.10.2-34 Prestressing Secondary Effects

P/S Response After Long Term Losses in Bent 2, Column 1 (All Frames)						
Location	**AX**	**VY**	**VZ**	**TX**	**MY**	**MZ**
ft	**kip**	**kip**	**kip**	**kip-ft**	**kip-ft**	**kip-ft**
0.00	-57.7	-0.4	-0.6	0	-0.0	0.0
11.00	-57.7	-0.4	-0.6	0	-6.3	4.7
22.00	-57.7	-0.4	-0.6	0	-12.6	9.3
33.00	-57.7	-0.4	-0.6	0	-19.0	14.0
44.00	-57.7	-0.4	-0.6	0	-25.3	18.7

5.7.10.2.14 WinYIELD Input for Column 1 at Bent 2

Design of column reinforcement is performed by running WinYIELD starting by general form as shown in Figure 5.7.10.2-5.

Two column models must be created in WinYield (version 3.0.10 and older) due to wind load limitations in the current version:

- WinYield only supports one load input for Wind on Structure. A WinYield model must be created for both the Strength III and Strength V load combinations.
- WinYield uses a wind load factor of 1.40 and 0.40 for the Strength III and Strength V load combinations, respectively. The Wind on Structure load input must be modified since the current CA Amendment specifies a load factor of 1.00 for both load combinations per AASHTO 8th edition.

Table 5.7.10.2-35, Table 5.7.10.2-36, and Table 5.7.10.2-37 show the WinYield load input for the Strength III and Strength V column models:

Table 5.7.10.2-35 WinYield Moving Load Input Summary

Case-1 Max Transverse - M_y					
	P-Truck PPL+IM	H-Truck PPL+IM	Lane Load LL	Centrifugal	
				PC-Truck	HC-Truck
M_y - Trans	813	343	219	0	0
M_x - Long	-378	-179	-271	0	0
P - Axial	446	224	143	0	0
Case-2 Max Longitudinal - M_x					
	P-Truck PPL+IM	H-Truck PPL+IM	Lane Load LL	Centrifugal	
				PC-Truck	HC-Truck
M_y - Trans	-110	46	26	0	0
M_x - Long	-3941	-1463	-696	0	0
P - Axial	577	216	121	0	0
Case-3 Max Axial – P					
	P-Truck PPL+IM	H-Truck PPL+IM	Lane Load LL	Centrifugal	
				PC-Truck	HC-Truck
M_y - Trans	-176	74	48	0	0
M_x - Long	-784	-275	-418	0	0
P - Axial	925	344	220	0	0

Table 5.7.10.2-36 WinYield Load Input Summary, Strength III

	DC	DW	PS	PL	BR
	Dead Load	Added Dead Load	Prestress Secondary	Ped Live Load	Braking Force
M_y - Trans	328	28	-25	0	191
M_x - Long	-1071	-138	19	0	-524
P - Axial	1507	161	58	0	-2
	WA	WS-Wind Str Horiz		WL-Wind Live Load	
	Stream Pressure	Case-1** Max Trans	Case-2** Max Long	Case-1 Max Trans	Case-2 Max Long
M_y - Trans	0	1673	917	0	0
M_x - Long	0	686	-578	0	0
P - Axial	0	-245	-46	0	0
	WV	FR	CR	SH	TU
	Wind Str Vertical *	Friction	Creep	Shrinkage	Temp Uniform
M_y - Trans	0	0	0	-489	196
M_x - Long	0	0	0	1245	-498
P - Axial	0	0	0	36	-14

* The vertical wind load was applied in conjunction with the horizontal
** Loads have been adjusted to account for the discrepancy between the load factors used in WinYield and the California Amendments.

Table 5.7.10.2-37 WinYield Load Input Summary, Strength V

	DC	DW	PS	PL	BR
	Dead Load	Added Dead Load	Prestress Secondary	Ped Live Load	Braking Force
M_y - *Trans*	328	28	-25	0	191
M_x - *Long*	-1071	-138	19	0	-524
P - Axial	1507	161	58	0	-2
	WA	WS-Wind Str Horiz		WL-Wind Live Load	
	Stream Pressure	Case-1** Max Trans	Case-2** Max Long	Case-1 Max Trans	Case-2 Max Long
M_y - *Trans*	0	2825	1425	447	208
M_x - *Long*	0	1040	-910	164	-135
P - Axial	0	-198	-70	-33	-11
	WV	FR	CR	SH	TU
	Wind Str Vertical *	Friction	Creep	Shrinkage	Temp Uniform
M_y - *Trans*	0	0	0	-489	196
M_x - *Long*	0	0	0	1245	-498
P - Axial	0	0	0	36	-14

* The vertical wind load was applied in conjunction with the horizontal
** Loads have been adjusted to account for the discrepancy between the load factors used in WinYield and the California Amendments.

Compression forces should be input as a positive value in WinYield, which is the opposite sign convention used in CSiBridge and CTBridge.

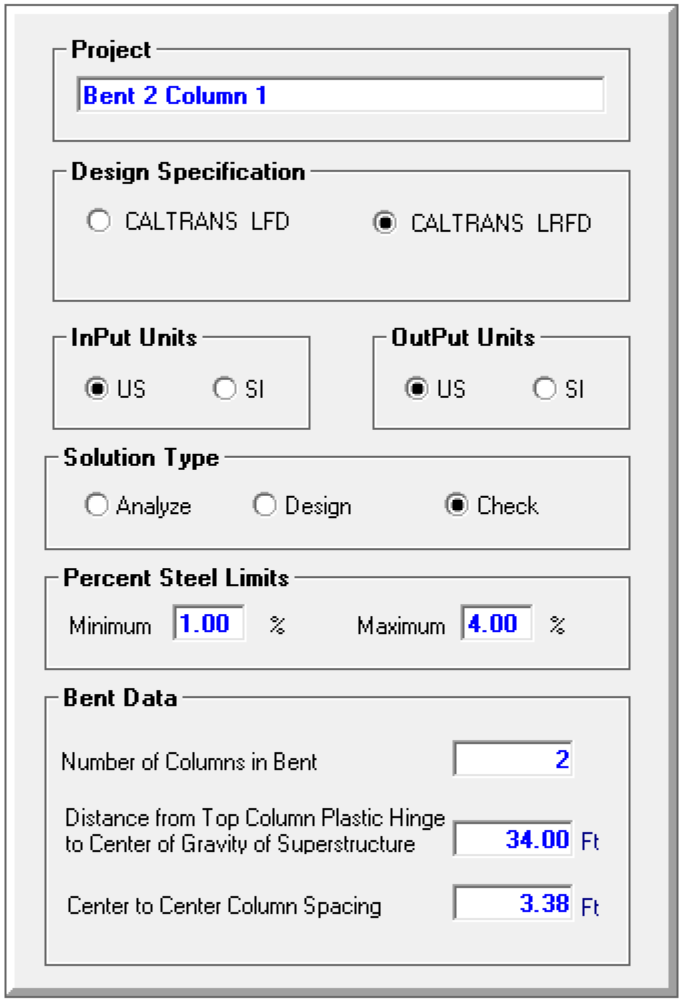


Figure 5.7.10.2-5 WinYIELD General Form

The column form for a circular column with a diameter of 72 inches is shown in Figure 5.7.10.2-6.

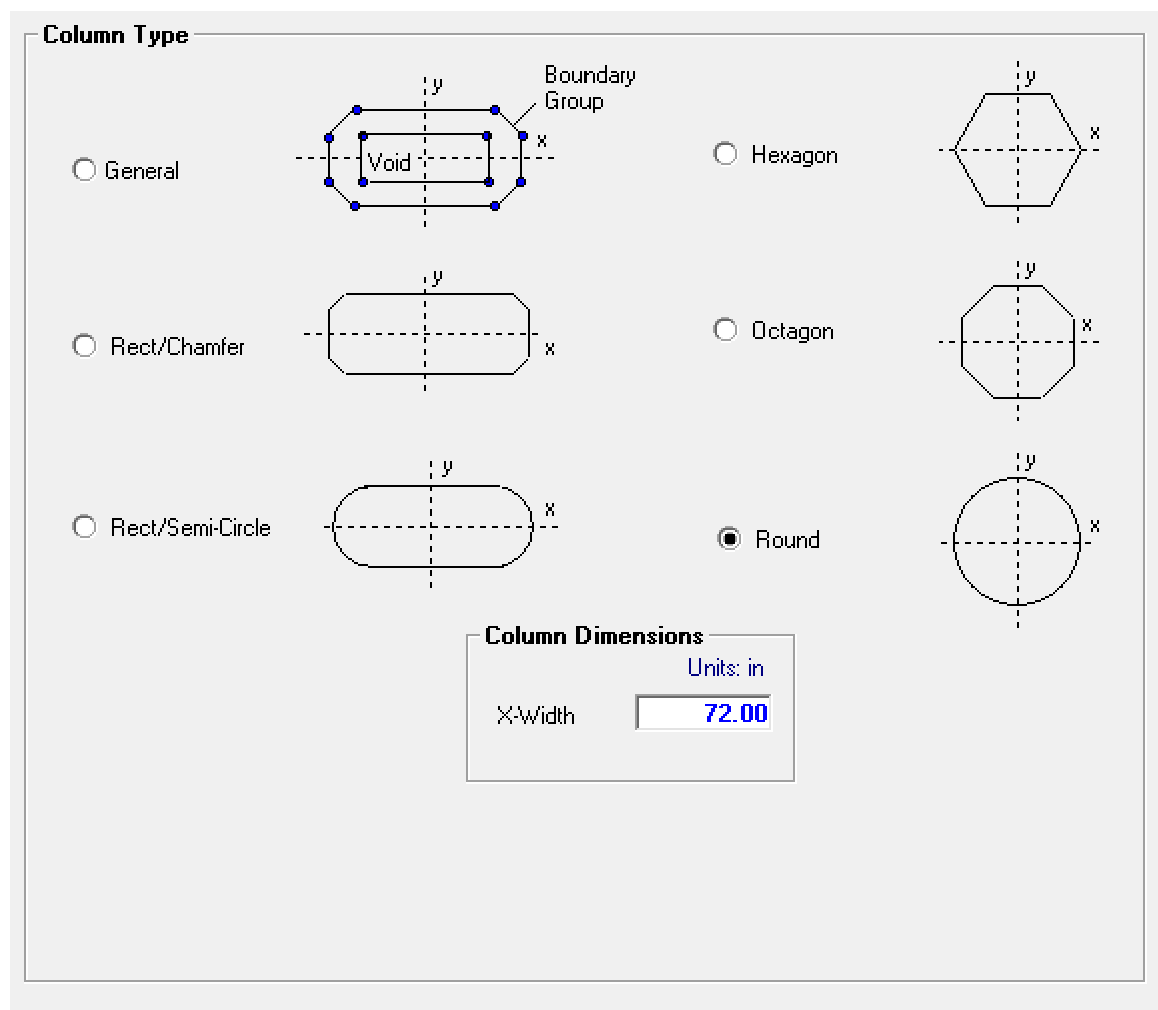


Figure 5.7.10.2-6 WinYIELD Column Form

The material form (Figure 5.7.10.2-7) shows concrete specified compressive strength, f'_c = 3.6 ksi and steel rebar specified minimum yield strength, f_y = 60 ksi.

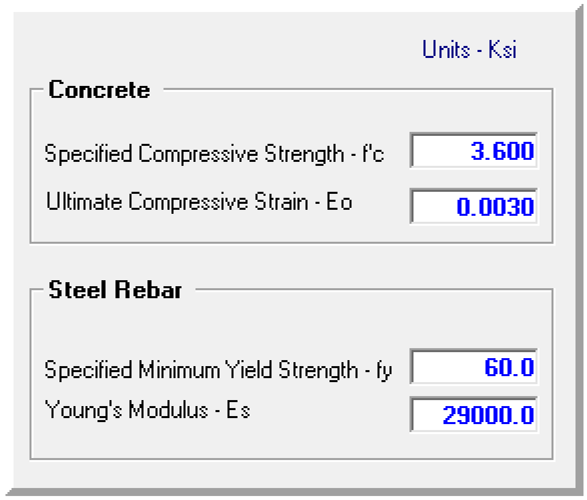


Figure 5.7.10.2-7 WinYIELD Material Form

Figure 5.7.10.2-8 shows the rebar form. Doubly symmetric layout for the main longitudinal column reinforcement is recommended, allowing for easier placement of bent cap reinforcement. For this example, assume #14 bars totaling 36 (18 bundles of 2) and #8 hoops.

When determining the loop (hoop) radius, use the deformed rebar diameters:

$$\text{Out to Out Distance} = D_c - 2\times Cover$$

$$= 72 - 2(2) = 68 \text{ in.}$$

$$\text{Loop Radius} = D_c - 2\times Cover - 2d_h - d_l$$

$$= 0.5\left[72 - 2(2) - 2(1.13) - (1.88)\right] = 31.93 \text{ in.}$$

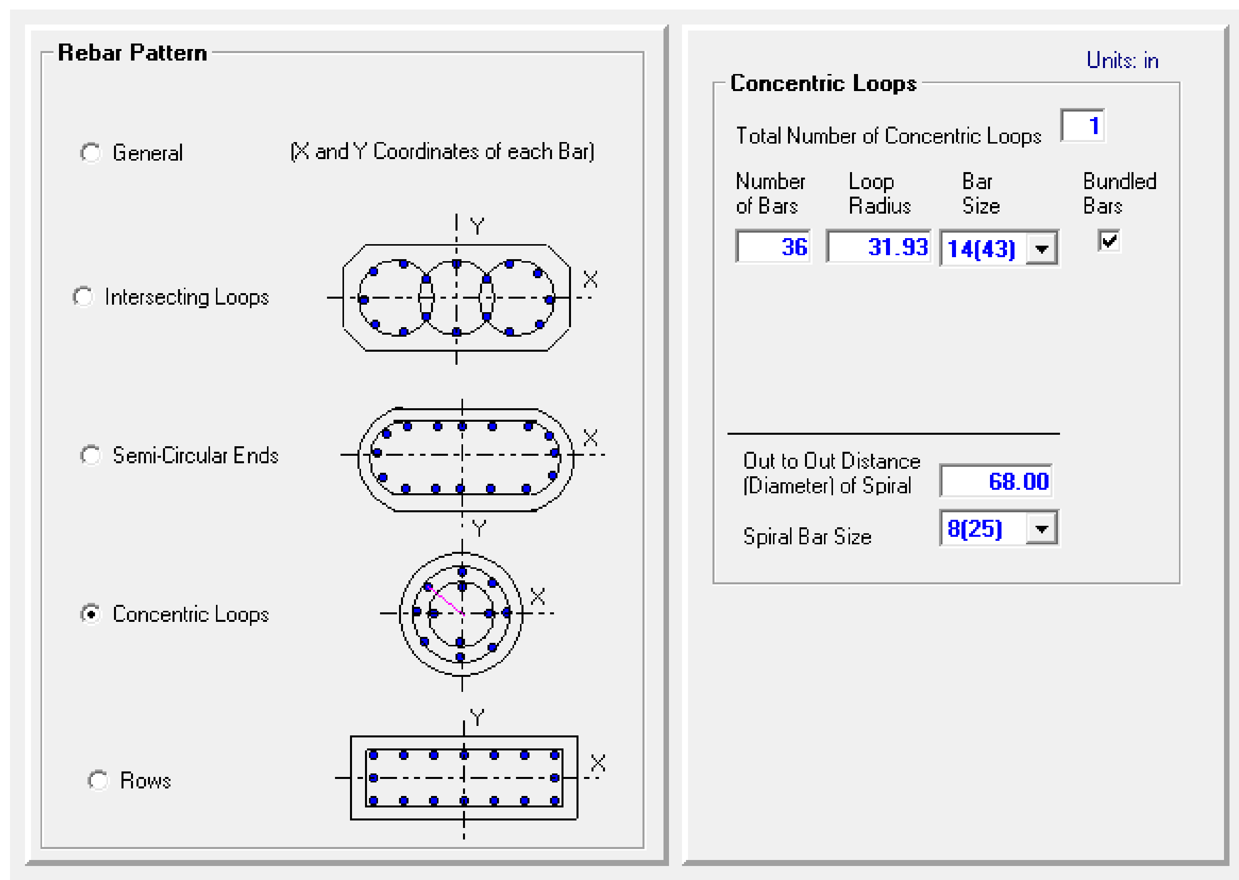
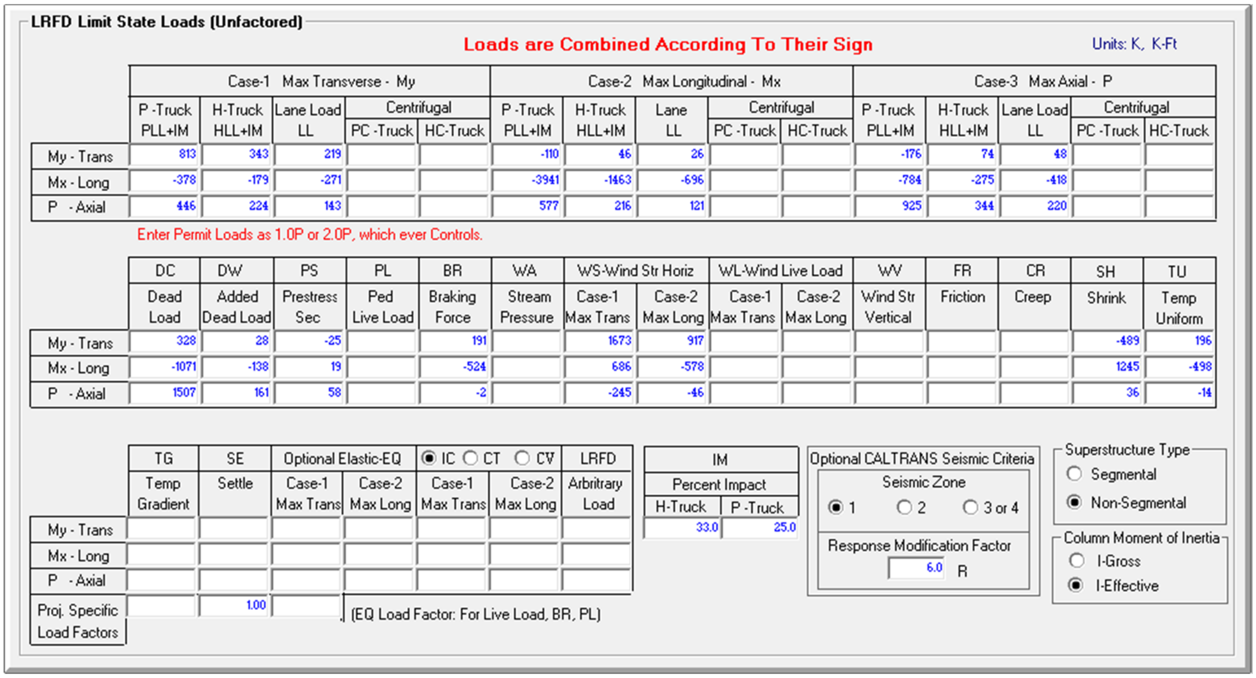


Figure 5.7.10.2-8 WinYIELD Rebar Form

Use AASHTO Table C4.6.2.5-1 to determine K_x and K_y used in the “Load-1” form (Figure 5.7.10.2-9):

Figure 5.7.10.2-9 WinYIELD Load Case 1 Form

"Load-2" form (Figure 5.7.10.2-10) input data is taken from Table 5.7.10.2-15. All axial compressive forces should be input as positive.



LRFD Limit State Loads (Unfactored)

Loads are Combined According To Their Sign — Units: K, K-Ft

	Case-1 Max Transverse - My					Case-2 Max Longitudinal - Mx					Case-3 Max Axial - P				
	P-Truck PLL+IM	H-Truck HLL+IM	Lane Load LL	Centrifugal PC-Truck	Centrifugal HC-Truck	P-Truck PLL+IM	H-Truck HLL+IM	Lane LL	Centrifugal PC-Truck	Centrifugal HC-Truck	P-Truck PLL+IM	H-Truck HLL+IM	Lane Load LL	Centrifugal PC-Truck	Centrifugal HC-Truck
My - Trans	813	343	219			-110	46	26			-176	74	48		
Mx - Long	-378	-179	-271			-3941	-1463	-696			-784	-275	-418		
P - Axial	446	224	143			577	216	121			925	344	220		

Enter Permit Loads as 1.0P or 2.0P, which ever Controls.

	DC Dead Load	DW Added Dead Load	PS Prestress Sec	PL Ped Live Load	BR Braking Force	WA Stream Pressure	WS-Wind Str Horiz Case-1 Max Trans	WS-Wind Str Horiz Case-2 Max Long	WL-Wind Live Load Case-1 Max Trans	WL-Wind Live Load Case-2 Max Long	WV Wind Str Vertical	FR Friction	CR Creep	SH Shrink	TU Temp Uniform
My - Trans	328	28	-25		191		1673	917						-489	196
Mx - Long	-1071	-138	19		-524		686	-578						1245	-498
P - Axial	1507	161	58		-2		-245	-46						36	-14

	TG Temp Gradient	SE Settle	Optional Elastic-EQ (◉ IC ○ CT ○ CV) Case-1 Max Trans	Case-2 Max Long	Case-1 Max Trans	Case-2 Max Long	LRFD Arbitrary Load
My - Trans							
Mx - Long							
P - Axial							
Proj. Specific Load Factors		1.00		(EQ Load Factor: For Live Load, BR, PL)			

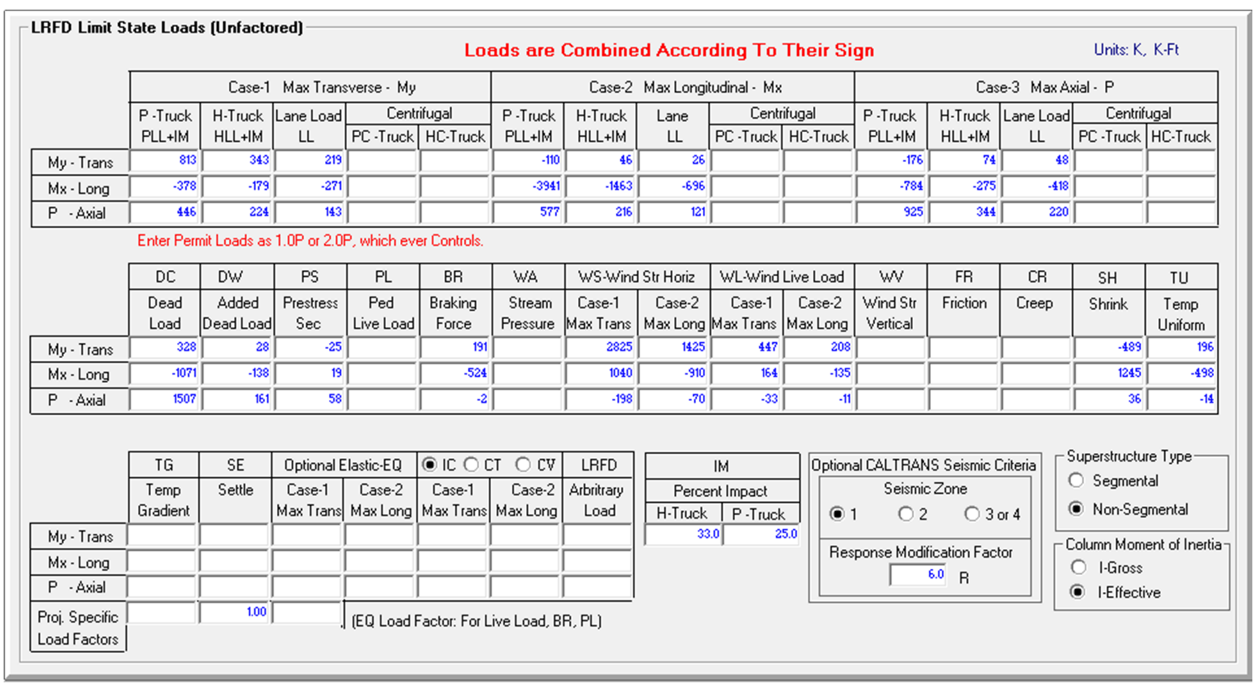
IM — Percent Impact: H-Truck 33.0, P-Truck 25.0

Optional CALTRANS Seismic Criteria — Seismic Zone: ◉ 1 ○ 2 ○ 3 or 4; Response Modification Factor 6.0 R

Superstructure Type: ○ Segmental ◉ Non-Segmental

Column Moment of Inertia: ○ I-Gross ◉ I-Effective

Figure 5.7.10.2-10 WinYIELD Load-2 Form for Strength III



LRFD Limit State Loads (Unfactored)

Loads are Combined According To Their Sign — Units: K, K-Ft

	Case-1 Max Transverse - My					Case-2 Max Longitudinal - Mx					Case-3 Max Axial - P				
	P-Truck PLL+IM	H-Truck HLL+IM	Lane Load LL	Centrifugal PC-Truck	Centrifugal HC-Truck	P-Truck PLL+IM	H-Truck HLL+IM	Lane LL	Centrifugal PC-Truck	Centrifugal HC-Truck	P-Truck PLL+IM	H-Truck HLL+IM	Lane Load LL	Centrifugal PC-Truck	Centrifugal HC-Truck
My - Trans	813	343	219			-110	46	26			-176	74	48		
Mx - Long	-378	-179	-271			-3941	-1463	-696			-784	-275	-418		
P - Axial	446	224	143			577	216	121			925	344	220		

Enter Permit Loads as 1.0P or 2.0P, which ever Controls.

	DC Dead Load	DW Added Dead Load	PS Prestress Sec	PL Ped Live Load	BR Braking Force	WA Stream Pressure	WS-Wind Str Horiz Case-1 Max Trans	WS-Wind Str Horiz Case-2 Max Long	WL-Wind Live Load Case-1 Max Trans	WL-Wind Live Load Case-2 Max Long	WV Wind Str Vertical	FR Friction	CR Creep	SH Shrink	TU Temp Uniform
My - Trans	328	28	-25		191		2825	1425	447	208				-489	196
Mx - Long	-1071	-138	19		-524		1040	-910	164	-135				1245	-498
P - Axial	1507	161	58		-2		-198	-70	-33	-11				36	-14

	TG Temp Gradient	SE Settle	Optional Elastic-EQ (◉ IC ○ CT ○ CV) Case-1 Max Trans	Case-2 Max Long	Case-1 Max Trans	Case-2 Max Long	LRFD Arbitrary Load
My - Trans							
Mx - Long							
P - Axial							
Proj. Specific Load Factors		1.00		(EQ Load Factor: For Live Load, BR, PL)			

IM — Percent Impact: H-Truck 33.0, P-Truck 25.0

Optional CALTRANS Seismic Criteria — Seismic Zone: ◉ 1 ○ 2 ○ 3 or 4; Response Modification Factor 6.0 R

Superstructure Type: ○ Segmental ◉ Non-Segmental

Column Moment of Inertia: ○ I-Gross ◉ I-Effective

Figure 5.7.10.2-11 WinYIELD Load-2 Form for Strength V

5.7.10.2.15 WinYIELD Output

WinYIELD output sheet (Figure 5.7.10.2-12) shows the steel reinforcement required for the column.

```
*****************
* Final Results *
*************************************************************************
Controlling loading            ...     Str-II  Case 2
Nominal axial load strength    ...     14716 kips
Total no. of bars input        ...        18 bars
Percent steel required         ...      1.00 percent
Adjusted area of each bar      ...      2.25 in^2
Total area of steel required   ...     40.55 in^2
Total number of bars required  ...       9.0 bars at   4.50 in^2 per bar

** Note: if the bar size is changed, bar locations will change,
     and the designer should consider adjusting the radius
     of main steel bar loop and re-run the program.

** Note: the designer must check to ensure that bar spacing limits
     of code are satisfied.

** Note: all group loads required less than 1.00 % steel.
```

These are bundled bars since the “bundled bars” option was checked

Figure 5.7.10.2-12 WinYIELD Output Results for #14 Total 36

The total area of steel required is much less than the initial assumption (only 9 bundles required). Using #11 bars totaling 36 (18 bundles of 2) would result in a more economical design and provide more than the 1% steel minimum.

```
*****************
* Final Results *
*************************************************************************
Controlling loading           ...    Str-II  Case 2
Nominal axial load strength   ...    14716 kips
Total no. of bars input       ...       18 bars
Percent steel required        ...     1.00 percent
Adjusted area of each bar     ...     2.25 in^2
Total area of steel required  ...    40.55 in^2
Total number of bars required ...     13.0 bars at   3.12 in^2 per bar

** Note: if the bar size is changed, bar locations will change,
     and the designer should consider adjusting the radius
     of main steel bar loop and re-run the program.

** Note: the designer must check to ensure that bar spacing limits
     of code are satisfied.

** Note: all group loads required less than 1.00 % steel.
```

Figure 5.7.10.2-13 WinYIELD Output Results for #11 Total 36

The final design may be summarized as:

- Provided rebar = 18 bundles > required number of bundles = 13.0 (OK)
- Min. recommended spacing for #11 bundle horizontally = 6.5 in. (OK)
- Distance between bundles = $\dfrac{2\pi(31.93)}{18} = 11.1 \text{ in.} \geq 6.5 \text{ in.}$ (OK)

5.7.10.3 Shear Design for Transverse Reinforcement (A_v)

The procedure of determining column transverse reinforcement is presented in the following sections.

5.7.10.3.1 Longitudinal Analysis

From CTBridge output (Table 5.7.10.3-1 and Table 5.7.10.3-2), determine longitudinal shear (V_y) and moment (M_z) at the top and bottom of columns for DC and DW. Combine output in Table 5.7.10.3-3.

Table 5.7.10.3-1 Dead Load, Unfactored Column Forces

Dead Load - Unfactored Column Forces - Final

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	-1577.8	24.3	7.8	0.0	0.0	-0.0
11.00	-1531.1	24.3	7.8	0.0	85.6	-267.7
22.00	-1484.5	24.3	7.8	0.0	171.3	-535.4
33.00	-14378	24.3	7.8	0.0	256.9	-803.2
44.00	-1391.2	24.3	7.8	0.0	342.5	-1070.9

Table 5.7.10.3-2 Additional Dead Load, Unfactored Column Forces

Additional Dead Load - Unfactored Column Forces

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip-ft	MY kip-ft	MZ kip-ft
0.00	-158.7	3.10	1.0	0.0	0.0	-0.0
11.00	-158.7	3.10	1.0	0.0	11.0	-34.4
22.00	-158.7	3.10	1.0	0.0	22.0	-68.8
33.00	-158.7	3.10	1.0	0.0	33.0	-103.2
44.00	-158.7	3.10	1.0	0.0	44.0	-137.5

Table 5.7.10.3-3 Longitudinal Shear (V_y) and Longitudinal Moment (M_z) for DC and DW

Load	Top of Column		Bottom of Column	
	DC	DW	DC	DW
V_y (kip)	24.3	3.1	24.3	3.1
M_z (kip-ft)	-1070.9	-137.5	0	0

Determine maximum longitudinal shear (V_y) and associated moment (M_z) for design vehicular live loads at the top and bottom of the bent unfactored reactions for one lane as shown in Table 5.7.10.3-4.

Table 5.7.10.3-4 Unfactored Bent Reactions For HL-93 Vehicle

Live Load - Controlling Unfactored Bent Reactions							
		Bent 2 Reactions - LRFD Design Vehicle					
		No Dynamic Load Allowance - Single Lane					
Location ft	**Primary DOF**	**T / L**	**AX kip**	**VY kip**	**VZ kip**	**MY kip-ft**	**MZ kip-ft**
Col Bots	VY-	Truck	-44.36	-7.45	-2.60	0.00	0.00
		Lane	-41.44	-5.35	-1.87	0.00	0.00
Col Bots	VY+	Truck	-71.07	10.91	3.80	0.00	0.00
		Lane	-53.42	7.04	2.45	0.00	0.00
Col Tops	VY-	Truck	-44.36	-7.45	-2.60	-114.37	327.85
		Lane	-41.44	-5.35	-1.87	-82.09	235.26
Col Tops	VY+	Truck	-71.07	10.91	3.80	167.39	-480.21
		Lane	-53.42	7.04	2.45	108.00	-309.84

Determine maximum longitudinal shear (V_y) and associated moment (M_z) for permit vehicular live loads at the top and bottom of the bent unfactored reactions for one lane as shown in Table 5.7.10.3-5.

Table 5.7.10.3-5 Unfactored Bent Reactions For P15 Vehicle

Live Load - Controlling Unfactored Bent Reactions							
Bent 2 Reactions - LRFD Permit Vehicle							
No Dynamic Load Allowance - Single Lane							
Location ft	**Primary DOF**	**T / L**	**AX kip**	**VY kip**	**VZ kip**	**MY kip-ft**	**MZ kip-ft**
Col Bots	VY-	Truck	-234.13	-14.92	-5.21	0.00	0.00
Col Bots	VY+	Truck	-222.39	34.55	12.04	0.00	0.00
Col Tops	VY-	Truck	-234.13	-14.92	-5.21	-229.04	656.50
Col Tops	VY+	Truck	-222.39	34.55	12.04	529.87	-1520.13

A summary of the controlling unfactored live load bent reactions can be found in Table 5.7.10.3-6.

Table 5.7.10.3-6 Unfactored Bent Reactions for One Lane, Dynamic Load Allowance Factors Not Included

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum longitudinal shear and associated longitudinal moment at top of the column		Maximum longitudinal shear and associated longitudinal moment at top of the column	
Force	$(V_y)_{max}$ (kip)	$(M_z)_{assoc}$ (kip-ft)	$(V_y)_{max}$ (kip)	$(M_z)_{assoc}$ (kip-ft)
Truck	10.9	-480.2	34.6	-1520.1
Lane	7.0	-309.8	--	--
Case	Maximum longitudinal shear and associated longitudinal moment at bottom of the column		Maximum longitudinal shear and associated longitudinal moment at bottom of the column	
Force	$(V_y)_{max}$ (kip)	$(M_z)_{assoc}$ (kip-ft)	$(V_y)_{max}$ (kip)	$(M_z)_{assoc}$ (kip-ft)
Truck	10.9	0	34.6	0
Lane	7.0	0	--	--

Apply dynamic allowance factor to Table 5.7.10.3-6 and divide by the number of columns as shown in Table 5.7.10.3-7.

Table 5.7.10.3-7 Unfactored Column Longitudinal Shear and Associated Longitudinal Moment for One Lane, Including Dynamic Load Allowance Factors

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum longitudinal shear and associated longitudinal moment at top of the column		Maximum longitudinal shear and associated longitudinal moment at top of the column	
Force	$(V_y)_{max}$ (kip)	$(M_z)_{assoc}$ (kip-ft)	$(V_y)_{max}$ (kip)	$(M_z)_{assoc}$ (kip-ft)
Truck	7.2	-319.3	21.6	-950.1
Lane	3.5	-154.9	--	--
Case	Maximum longitudinal shear and associated longitudinal moment at bottom of the column		Maximum longitudinal shear and associated longitudinal moment at bottom of the column	
Force	$(V_y)_{max}$ (kip)	$(M_z)_{assoc}$ (kip-ft)	$(V_y)_{max}$ (kip)	$(M_z)_{assoc}$ (kip-ft)
Truck	7.2	0	21.6	0
Lane	3.5	0	--	--

5.7.10.3.2 Transverse Analysis

CSiBridge output for load cases of dead load (*DC*) and dead load of wearing surface load (*DW*) is shown in Table 5.7.10.3-8.

Table 5.7.10.3-8 Transverse Shear (V_2) and Moment (M_3) at Top and Bottom of Columns due to Dead Load (DC) and Added Dead Load (DW)

TABLE: Element Forces – Frames									
Frame	**Station**	**Output Case**	**Step Type**	**P ***	**V2**	**V3**	**T**	**M2**	**M3**
Text	ft	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
1	0	DC		-1693.6	-7.4	0	0	0	0
1	11	DC		-1646.9	-7.4	0	0	0	81.9
1	11	DC		-1646.9	-7.4	0	0	0	81.9
1	22	DC		-1600.3	-7.4	0	0	0	163.8
1	22	DC		-1600.3	-7.4	0	0	0	163.8
1	33	DC		-1553.7	-7.4	0	0	0	245.7
1	33	DC		-1553.7	-7.4	0	0	0	245.7
1	44	DC		-1507.0	-7.4	0	0	0	327.6
1	0	DW		-161.4	-0.6	0	0	0	0
1	11	DW		-161.4	-0.6	0	0	0	7.1
1	11	DW		-161.4	-0.6	0	0	0	7.1
1	22	DW		-161.4	-0.6	0	0	0	14.2
1	22	DW		-161.4	-0.6	0	0	0	14.2
1	33	DW		-161.4	-0.6	0	0	0	21.3
1	33	DW		-161.4	-0.6	0	0	0	21.3
1	44	DW		-161.4	-0.6	0	0	0	28.4

Combine output in Table 5.7.10.3-9.

Table 5.7.10.3-9 Transverse Shear (V_2) and Moment (M_3) for DC and DW

Load	Top of Column		Bottom of Column	
	DC	DW	DC	DW
V_2 (kip)	-7.4	-0.6	-7.4	-0.6
M_3 (kip-ft)	327.6	28.4	0	0

CSiBridge output for maximum shear (V_2) and associated and moment (M_3) for design vehicle including dynamic load allowance as shown in Table 5.7.10.3-10.

Table 5.7.10.3-10 Maximum Shear (V_2) and Associated Moment (M_3) for HL-93 Vehicle

TABLE: Element Forces – Frames									
Frame	**Station**	**Output Case**	**Step Type**	**P**	**V2**	**V3**	**T**	**M2**	**M3**
Text	ft	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
1	0	HLL	Max V2	-248.9	8.8	0	0	0	0
1	11	HLL	Max V2	-248.9	8.8	0	0	0	-96.3
1	11	HLL	Max V2	-248.9	8.8	0	0	0	-96.3
1	22	HLL	Max V2	-248.9	8.8	0	0	0	-192.6
1	22	HLL	Max V2	-248.9	8.8	0	0	0	-192.6
1	33	HLL	Max V2	-248.9	8.8	0	0	0	-288.9
1	33	HLL	Max V2	-248.9	8.8	0	0	0	-288.9
1	44	HLL	Max V2	-248.9	8.8	0	0	0	-385.2
1	0	HLL	Min V2	-367.8	-12.8	0	0	0	0
1	11	HLL	Min V2	-367.8	-12.8	0	0	0	140.5
1	11	HLL	Min V2	-367.8	-12.8	0	0	0	140.5
1	22	HLL	Min V2	-367.8	-12.8	0	0	0	280.9
1	22	HLL	Min V2	-367.8	-12.8	0	0	0	280.9
1	33	HLL	Min V2	-367.8	-12.8	0	0	0	421.4
1	33	HLL	Min V2	-367.8	-12.8	0	0	0	421.4
1	44	HLL	Min V2	-367.8	-12.8	0	0	0	561.8

CSiBridge output for maximum shear (V_2) and associated and moment (M_3) for P15 vehicle including dynamic load allowance as shown in Table 5.7.10.3-11.

Table 5.7.10.3-11 Maximum Shear (V2) and Associated Moment (M3) for P15 Vehicle

TABLE: Element Forces – Frames									
Frame	**Station**	**Output Case**	**Step Type**	**P**	**V2**	**V3**	**T**	**M2**	**M3**
Text	ft	Text	Text	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
1	0	PLL	Max V2	-446.1	15.7	0	0	0	0
1	11	PLL	Max V2	-446.1	15.7	0	0	0	-172.6
1	11	PLL	Max V2	-446.1	15.7	0	0	0	-172.6
1	22	PLL	Max V2	-446.1	15.7	0	0	0	-345.2
1	22	PLL	Max V2	-446.1	15.7	0	0	0	-345.2
1	33	PLL	Max V2	-446.1	15.7	0	0	0	-517.8
1	33	PLL	Max V2	-446.1	15.7	0	0	0	-517.8
1	44	PLL	Max V2	-446.1	15.7	0	0	0	-690.4
1	0	PLL	Min V2	-446.1	-18.5	0	0	0	0
1	11	PLL	Min V2	-446.1	-18.5	0	0	0	203.3
1	11	PLL	Min V2	-446.1	-18.5	0	0	0	203.3
1	22	PLL	Min V2	-446.1	-18.5	0	0	0	406.5
1	22	PLL	Min V2	-446.1	-18.5	0	0	0	406.5
1	33	PLL	Min V2	-446.1	-18.5	0	0	0	690.8
1	33	PLL	Min V2	-446.1	-18.5	0	0	0	690.8
1	44	PLL	Min V2	-446.1	-18.5	0	0	0	813.0

Re-arrange the transverse shear and moment output from CSiBridge in Table 5.7.10.3-12.

Table 5.7.10.3-12 Combined Unfactored Column Reaction, Including Dynamic Load Allowance Factors

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum transverse shear and associated transverse moment at top of the column		Maximum transverse shear and associated transverse moment at top of the column	
Force	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)
Truck	-12.8	561.8	-18.5	813
Case	Maximum transverse shear and associated transverse moment at bottom of the column		Maximum transverse shear and associated transverse moment at bottom of the column	
Force	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)
Truck	-12.8	0	-18.5	0

Use the procedure shown in Section 5.7.7.3.4 and arrange the output in Table 5.7.10.3-13.

Table 5.7.10.3-13 Separated Unfactored Column Reactions, Including Dynamic Load Allowance Factors

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum transverse shear and associated longitudinal moment at top of the column		Maximum transverse shear and associated longitudinal moment at top of the column	
Force	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)
Truck	-7.8	342.7	-18.5	813
Lane	-5.0	219.1	--	--
Case	Maximum transverse shear and associated longitudinal moment at bottom of the column		Maximum transverse shear and associated longitudinal moment at bottom of the column	
Force	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)
Truck	-7.8	0	-18.5	0
Lane	-5.0	0	--	--

5.7.10.3.3 Total Longitudinal Shear and Associated Moments

Total column longitudinal shear and associated moment as per Section 5.7.8.3 is presented in Table 5.7.10.3-14.

Table 5.7.10.3-14 Unfactored Total Longitudinal Shear and Associated Longitudinal Moment, Column 1, Including Dynamic Load Allowance Factors

Vehicle	HL-93 Vehicle		P15 Vehicle	
Case	Maximum longitudinal shear and associated longitudinal moment at top of the column		Maximum longitudinal shear and associated longitudinal moment at top of the column	
Force	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)
Truck	33.2	-1465	89.7	3941
Lane	15.7	-695	--	--
Case	Maximum longitudinal shear and associated longitudinal moment at bottom of the column		Maximum longitudinal shear and associated longitudinal moment at bottom of the column	
Force	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)
Truck	33.2	0	89.7	0
Lane	15.7	0	--	--

For this example, the controlling longitudinal shear in the column due to wind load and wind on live load occur at an angle of attack of 0°. A summary of the shear forces and associated moments can be found in Table 5.7.10.3-15 and Table 5.7.10.3-16.

Table 5.7.10.3-15 Longitudinal Shear (V_y) and Longitudinal Moment (M_z) for DC and DW

Load	Top of Column			Bottom of Column		
	WS Str III	WS Str V	WL	WS Str III	WS Str V	WL
V_y (kip)	-19.4	-8.4	-3.7	-24.2	-10.6	-3.7
M_z (kip-ft)	960.1	415.9	164.2	0	0	0

Table 5.7.10.3-16 Transverse Shear (V_z) and Transverse Moment (M_y) for DC and DW

Load	Top of Column			Bottom of Column		
	WS Str III	WS Str V	WL	WS Str III	WS Str V	WL
V_z (kip)	46.2	22.4	10.2	60.3	29.0	10.2
M_y (kip-ft)	2341.8	1130.0	447.2	0	0	0

5.7.10.3.4 Summary of Column Shear Loads

Column shear loads are summarized in Table 5.7.10.3-17 and Table 5.7.10.3-18.

Table 5.7.10.3-17 Longitudinal Shear and Associated Longitudinal Moment

Load Case	Top of Column		Bottom of Column	
	$(V_y)_{max}$ (kip)	$(M_z)_{assoc}$ (kip-ft)	$(V_y)_{max}$ (kip)	$(M_z)_{assoc}$ (kip-ft)
DC	24.3	1070.9	24.3	0
DW	3.1	137.5	3.1	0
PS	-0.4	18.7	-0.4	0
HL-93 (Truck)	33.2	-1465	33.2	0
HL-93 (Lane)	15.7	-695	15.7	0
P15 Truck	89.7	-3941	89.7	0
BR	-11.9	-523.9	-11.9	0
$WS_{STR\ III}$	-19.4	960.1	-24.2	0
$WS_{STR\ V}$	-8.4	415.9	-10.6	0
WL	-3.7	164.2	-3.7	0
TU	-11.3	498.1	-11.3	0
CR	-28.3	1245.1	-28.3	0

Table 5.7.10.3-18 Transverse Shear and Associated Transverse Moment

Load Case	Top of Column		Bottom of Column	
	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)	$(V_2)_{max}$ (kip)	$(M_3)_{assoc}$ (kip-ft)
DC	-7.4	327.6	-7.4	0
DW	-0.6	28.4	-0.6	0
PS	-0.6	-25.3	-0.6	0
HL-93 (Truck)	-7.8	342.7	-7.8	0
HL-93 (Lane)	-5.0	219.1	-5.0	0
P15 Truck	-18.5	813	-18.5	0
BR	-4.3	119.1	-4.3	0
$WS_{STR\ III}$	46.2	2341.8	60.3	0
$WS_{STR\ V}$	22.4	1130.0	29.0	0
WL	10.2	447.2	10.2	0
TU	-4.4	-195.6	-4.4	0
CR	-11.1	-488.9	-11.1	0

Since this example uses circular columns, the design shears and moments should be taken as the square root of the sum of the squares:

Table 5.7.10.3-19 Square Root of the Sum of the Squares

Load Case	Top of Column		Bottom of Column	
	V (kip)	$(M)_{assoc}$ (kip-ft)	V (kip)	$(M)_{assoc}$ (kip-ft)
DC	25	1120	25	0
DW	3	140	3	0
PS	1	31	1	0
HL-93 (Truck)	34	1505	34	0
HL-93 (Lane)	16	729	16	0
P15 Truck	92	4024	92	0
BR	13	537	13	0
$WS_{STR\ III}$	50	2531	65	0
$WS_{STR\ V}$	24	1204	31	0
WL	11	476	11	0
TU	12	535	12	0
CR	30	1338	30	0

5.7.10.3.5 Strength Shear Limit States

Determine Strength limit states for shear and associated moments at the top of column.

- Strength I force (controlling):

$$V_u = 1.25DC + 1.50DW + 0.50PS + 1.75(LL_{HL-93} + BR) + 0.50(TU + CR)$$

$$= 1.25(25) + 1.50(3) + 0.50(1) + 1.75(34 + 16 + 13) + 0.50(12 + 30) = 169 \text{ kips}$$

$$M_u = 1.25DC + 1.50DW + 0.50PS + 1.75(LL_{HL-93} + BR) + 0.50(TU + CR)$$

$$= 1.25(1120) + 1.50(140) + 0.50(31) + 1.75(1505 + 729 + 537)$$

$$+ 0.50(535 + 1338) = 7411 \text{ kip-ft}$$

- Strength II force Demands:

$$V_u = 1.25DC + 1.50DW + 0.50PS + 1.35LL_{Permit} + 0.50(TU + CR)$$

$$= 1.25(25) + 1.50(3) + 0.50(1) + 1.35(92) + 0.50(12 + 30) = 182 \text{ kips}$$

$$M_u = 1.25DC + 1.50DW + 0.50PS + 1.35LL_{Permit} + 0.50(TU + CR)$$

$$= 1.25(1120) + 1.50(140) + 0.50(31) + 1.35(4024)$$

$$+0.50(535+1338)=7995 \text{ kip-ft}$$

- Strength III force Demands:

$$V_u = 1.25DC + 1.50DW + 0.50PS + 1.00WS_{STR\ III} + 0.50(TU + CR)$$

$$= 1.25(25) + 1.50(3) + 0.50(1) + 1.00(50) + 0.50(12+30) = 108 \text{ kips}$$

$$M_u = 1.25DC + 1.50DW + 0.50PS + 1.00WS_{STR\ III} + 0.50(TU + CR)$$

$$= 1.25(1120) + 1.50(140) + 0.50(31) + 1.00(2531)$$

$$+0.50(535+1338) = 5094 \text{ kip-ft}$$

- Strength V force Demands:

$$V_u = 1.25DC + 1.50DW + 0.50PS + 1.35(LL_{HL-93} + BR) + 1.00(WS_{STR\ V} + WL)$$

$$+0.50(TU + CR)$$

$$= 1.25(25) + 1.50(3) + 0.50(1) + 1.35(34+16+13) + 1.00(24+11)$$

$$+0.50(12+30) = 178 \text{ kips}$$

$$M_u = 1.25DC + 1.50DW + 0.50PS + 1.35(LL_{HL-93} + BR) + 1.00(WS_{STR\ V} + WL)$$

$$+0.50(TU + CR)$$

$$= 1.25(1120) + 1.50(140) + 0.50(31) + 1.35(1505+729+537)$$

$$+1.00(1204+476) + 0.50(535+1338) = 7983 \text{ kip-ft}$$

The controlling shear force demands are from the Strength II load combination:

$$V_u = 182 \text{ kips}$$

$$M_u = 7995 \text{ kip-ft}$$

Determine the shear capacity of the column.

- Column section properties:

$$D = b_v = 72.0 \text{ in.}$$

$$D_r = 63.86 \text{ in.}$$

$$f_c' = 3.6 \text{ ksi}$$

$f_y = 60$ ksi

$A_v = 0.79$ in.^{2} (assume #8 hoops)

$\beta = 2.0$ (AASHTO 5.7.3.4.1)

$\theta = 45°$ (AASHTO 5.7.3.4.1)

$\phi_f = 0.90$ (AASHTO 5.5.4.2)

$\phi_v = 0.90$ (AASHTO 5.5.4.2)

- Check the spacing for the minimum transverse reinforcement per AASHTO 5.7.2.5:

$\lambda = 1.0$ (AASHTO 5.4.2.8)

$$d_e = \frac{D}{2} + \frac{D_r}{\pi}$$ (AASHTO Eq. C5.7.2.8-2)

$$= \frac{72.0}{2} + \frac{63.86}{\pi} = 56.3 \text{ in.}$$

$d_v = 0.9d_e$ (AASHTO C5.7.2.8)

$$= 0.9(56.3) = 50.7 \text{ in.}$$

$$\frac{A_v}{s} = 0.0316\lambda\sqrt{f_c^{'}}\frac{b_v}{f_y}$$ (AASHTO Eq. 5.7.2.5-1)

$$= 0.0316(1.0)\sqrt{3.6}\frac{72.0}{60} = 0.072 \text{ in.}^2\text{/in.}$$

$$\therefore s = \frac{0.79}{0.072} = 11.0 \text{ in. (Use=6.0 in.)}$$

- Check maximum spacing of transverse reinforcement per AASHTO 5.7.2.6:

$$v_u = \frac{\left|V_u - \phi V_p\right|}{\phi b_v d_v}$$ (AASHTO Eq. 5.7.2.8-1)

$$= \frac{\left|182 - 0\right|}{0.90(72.0)(50.7)} = 0.0553 \text{ ksi}$$

$$\frac{v_u}{f_c^{'}} = \frac{0.0553}{3.6} = 0.0154 < 0.125$$ (AASHTO 5.7.2.6)

$s_{max} = 0.8d_v \le 24.0$ in. (AASHTO Eq. 5.7.2.6-1)

$$= 0.8(50.7) = 40.6 \le 24.0$$

$$= 24.0 \text{ (Design uses=6.0 in.)}$$

- Check the column shear capacity using the simplified procedure for non-prestressed sections per AASHTO 5.7.3.3 and AASHTO C5.7.3.3:

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}b_v d_v \qquad \text{(AASHTO Eq. 5.7.3.3-3)}$$

$$= 0.0316(2.0)(1.0)\sqrt{3.6}(72.0)(50.7) = 437.7 \text{ kips}$$

$$V_s = \frac{A_v f_y d_v cot\theta}{s} \qquad \text{(AASHTO Eq. C5.7.3.3-4)}$$

$$= \frac{(0.79)(60)(50.7)cot(45)}{6} = 400.5 \text{ kips}$$

$$V_p = 0 \text{ kips}$$

$$V_n = \min\begin{cases} V_c + V_s + V_p \\ 0.25f'_c b_v d_v + V_p \end{cases} \qquad \text{(AASHTO Eq. 5.7.3.3-1 and 2)}$$

$$= \min\begin{cases} 437.7 + 400.5 + 0 \\ 0.25(3.6)(72.0)(50.7) + 0 \end{cases}$$

$$= \min\begin{cases} 838.2 \\ 3285.0 \end{cases}$$

$$= 838.2 \text{ kips}$$

$$V_r = \phi V_n \qquad \text{(AASHTO Eq. 5.7.2.1-1)}$$

$$= 0.9(838.2) = 754 \text{ kips} > 182 \text{ kips} = V_u$$

Use #8 hoops @ 6 in. since it meets the Strength shear capacity checks. Seismic shear demands should be checked per the current SDC. Column confinement/shear steel, in most normal cases, will be governed by the plastic hinge shear force.

- Check column shear-flexure interaction per AASHTO 5.7.3.5-1. N_u and V_s were conservatively taken as 0 kip for this example:

$$A_s f_y = 36(1.56)(60) = 3369 \text{ kips}$$

$$A_s f_y \geq \frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)cot(\theta) \qquad \text{(AASHTO Eq. 5.7.3.3-3)}$$

$$3369 \geq \frac{(7995)(12)}{(50.7)(0.90)} + \left[\left|\frac{182}{0.90}\right| - 0.5(0)\right]cot(45^\circ)$$

$$3369 \text{ kips} \geq 2305 \text{ kips}$$

Since the column reinforcing meets the longitudinal reinforcement requirements, #11 total 36 (18 bundles of 2) as shown in Figure 5.7.10.3-1 are OK.

Figure 5.7.10.3-1 Column Reinforcement Details

NOTATIONS

AoA	=	angle of attack (degrees)
A_g	=	gross area of section (in.^{2})
A_{st}	=	total area of main column reinforcement (in.^{2})
A_v	=	area of shear reinforcement within a distance s (in.^{2})
A_x	=	axial load (kip)
b_v	=	effective width (in.)
C_D	=	drag coefficient
C_m	=	a factor, which relates the actual moment diagram to an equivalent uniform moment diagram, is typically taken as 1.0
CSi	=	CSiBridge
CT	=	CTBridge
D	=	column diameter (in.)
$D_{barrier}$	=	barrier height (ft.)
D_{column}	=	column diameter (ft.)
d_e	=	effective depth from extreme compression fiber to the centroid of the tensile forces in the tensile reinforcement (in.)
d_h	=	diameter of column hoop reinforcement (in.)
d_l	=	diameter of column longitudinal reinforcement (in.)
D_r	=	diameter of the circle passing through the centers of the longitudinal reinforcement (in.)
D_{super}	=	superstructure depth (ft.)
d_v	=	effective shear depth in AASHTO C5.7.2.8 (in.)
E_c	=	the elastic modulus of concrete (ksi)
E_s	=	the elastic modulus of steel (ksi)
EI	=	flexural stiffness (kip-in.^{2})
f'_c	=	specified strength of concrete at 28 days, unless another age is specified (ksi)
f_y	=	specified yield strength of reinforcement (ksi)
F_v	=	vertical design wind force applied in the Strength III load combination (klf)
F_z	=	design wind force (klf)
$F_{z,long}$	=	design wind force acting along (parallel) to the local axis of the superstructure or substructure (klf)

$F_{z,trans}$	=	design wind force acting transverse (normal) to the local axis of the superstructure or substructure (klf)
G	=	gust effect factor
$G_{str\ III}$	=	gust effect factor used for the Strength III load combination in AASHTO Table 3.8.1.2.1-1
$G_{str\ V}$	=	gust effect factor used for the Strength V load combination in AASHTO Table 3.8.1.2.1-1
I	=	moment of inertia about axis under consideration (in.^{4})
I_g	=	gross moment of inertia (in.^{4})
I_s	=	moment of inertia of longitudinal reinforcement about neutral axis (in.^{4})
K	=	effective length factor in AASHTO C4.6.2.5-1
K_x	=	effective length factor in the longitudinal direction
K_y	=	effective length factor in the transverse direction
K_z	=	pressure exposure and elevation coefficient
L	=	column length (ft)
l_u	=	unsupported length of a compression member (in.)
M_1	=	smaller end moment, should be positive for single curvature flexure (kip-in.)
M_{1b}	=	smaller factored end moment (kip-in.)
M_2	=	larger end moment, should be positive for single curvature flexure (kip-in.)
M_{2b}	=	larger factored end moment, or moment on compression member due to factored gravity loads that result in no sidesway, always positive (kip-in.)
M_{2s}	=	moment on compression member due to factored lateral or gravity loads that result in sidesway, Δ, greater than l_u/1500, always positive (kip-in.)
M_3	=	transverse moment (kip-in.)
M_b	=	balanced moment resistance at balanced strain condition (kip-in.)
M_c	=	magnified factored moment (kip-in.)
M_n	=	nominal flexural resistance (kip-in.)
M_o	=	nominal flexural resistance of a section at zero eccentricity (kip-in.)
M_{rx}	=	uniaxial factored flexural resistance of a section in the direction of the x-axis (kip-in.)
M_{ry}	=	uniaxial factored flexural resistance of a section in the direction of the y-axis (kip-in.)
M_u	=	factored moment (kip-in.)
M_{ux}	=	factored applied moment about the x axis (kip-in.)

M_{uy} = factored applied moment about the y axis (kip-in.)

M_{WL} = wind load on live load moment due to the offset between the application point and the center of gravity of the structure (kip-ft/ft)

M_x = maximum longitudinal moment (kip-in.)

M_y = maximum transverse moment (kip-in.)

M_z = maximum longitudinal moment (kip-in.)

MPF = multiple presence factor

N_u = factored axial force, taken as positive if tensile and negative if compressive (kip)

P = maximum column axial load (kip)

P_b = balanced axial resistance at balanced strain condition (kip)

P_n = nominal axial resistance, with or without flexure (kip)

P_o = nominal axial resistance of a section at zero eccentricity (kip)

P_r = factored axial resistance (kip)

P_{rx} = factored axial resistance determined on the basis that only eccentricity e_y is present (kip)

P_{rxy} = factored axial resistance in biaxial flexure (kip)

P_{ry} = factored axial resistance determined on the basis that only eccentricity e_x is present (kip)

P_u = factored axial load (kip)

P_v = vertical wind pressure (ksf)

P_z = design wind pressure (ksf)

$P_{z,sub}$ = design wind pressure acting on the substructure (ksf)

$P_{z,super}$ = design wind pressure acting on the superstructure (ksf)

$PoNM$ = point of no movement (ft)

$R1$ = truck load ratio of HL-93 vehicle

$R2$ = lane load ratio of HL-93 vehicle

r = radius of gyration in AASHTO C5.6.4.3 (in.)

= 0.25 times the column diameter for circular columns

= 0.30 times the column dimension in the direction of buckling for rectangular columns

s = spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement (in.)

T_{median} = median temperature used in CTBridge (°F)

V	=	design 3-second gust wind speed (mph)
V_2	=	transverse shear force (kip)
V_c	=	concrete shear capacity (kip)
V_n	=	nominal shear capacity (kip)
V_p	=	component of prestressing force in the direction of the shear force (kip)
V_r	=	factored shear resistance (kip)
V_s	=	transverse shear reinforcement capacity (kip)
$V_{str\ III}$	=	design 3-second gust wind speed used for the Strength III load combination in AASHTO Table 3.8.1.1.2-1 (mph)
$V_{str\ V}$	=	design 3-second gust wind speed used for the Strength V load combination in AASHTO Table 3.8.1.1.2-1 (mph)
V_u	=	factored shear force (kip)
v_u	=	shear stress (ksi)
V_y	=	longitudinal shear (kip)
W_D	=	width of the superstructure (ft.)
β	=	factor indicating the ability of diagonally cracked concrete to transmit tension and shear as specified in AASHTO 5.7.3.4
β_d	=	ratio of maximum factored permanent load moment to the maximum factored total load moment, always positive
γ_p	=	load factor for permanent load due to creep and shrinkage
ε_c	=	compression strain of the concrete
ε_{csh}	=	prestress shortening strain due to creep and shrinkage
ε_s	=	net longitudinal tensile strain in the tension reinforcement
ε_y	=	yield strain of the steel
δ_b	=	moment magnification factor for compression member braced against sidesway
δ_s	=	moment magnification factor for compression member not braced against sidesway
θ	=	skew angle (degrees)
Δ_{csh}	=	shortening movement due to creep and shrinkage (in.)
Δ_T	=	temperature ranges (°F)
ϕ_c	=	axial resistance factor for compression controlled sections with spirals or ties
ϕ	=	resistance factor specified in AASHTO 5.5.4.2 and AASHTO CA 5.5.4.2

ϕ_f = flexural resistance factor

ϕ_k = stiffness reduction factor; 0.75 for concrete members and 1.0 for steel members

ϕ_v = shear resistance factor

λ = concrete density modification factor

REFERENCES

1. AASHTO. (2017). *AASHTO LRFD Bridge Design Specifications*, 8th Edition, American Association of State Highway and Transportation Officials, Washington DC.
2. Caltrans. (2019a). *California Amendments to AASHTO LRFD Bridge Design Specifications*, 8th Edition, California Department of Transportation, Sacramento, CA.
3. Caltrans. (2019b). *Caltrans Seismic Design Criteria*, Version 2.0, California Department of Transportation, Sacramento, CA.
4. Caltrans, (2019c). CTBridge, Caltrans Bridge Analysis, and Design v.3.0.10, California Department of Transportation, Sacramento, CA.
5. Caltrans. (2008). *WinYIELD (2008): Column Live Load Input Procedure*, California Department of Transportation, Sacramento, CA.
6. Chen, W.F. and Duan, L. Ed. (2014). *Bridge Engineering Handbook*, 2nd Edition, CRC Press, Boca Raton, FL.
7. CSI, Inc. (2021). *CSiBridge 2021*, Version 23, Computers and Structures, Inc. Walnut Creek, CA.
8. Wight, J.K. (2021). *Reinforced Concrete: Mechanics and Design,* 8th Edition, Pearson, New York, NY.