CHAPTER 20.1
SEISMIC DESIGN OF CONCRETE BRIDGES

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20.1.1 INTRODUCTION

This chapter is intended primarily to provide guidance on the seismic design of Ordinary Standard Concrete Bridges as defined in *Caltrans Seismic Design Criteria* (*SDC*), Version 2.0 (Caltrans 2019). Information presented herein is based on *SDC* (Caltrans 2019), AASHTO *LRFD Bridge Design Specifications* 8th Edition (AASHTO 2017) with *California Amendments* (Caltrans 2019), and other Caltrans bridge design guidance such as *Bridge Design Memos* (BDM). Criteria on the seismic design of nonstandard bridge features are developed on a project-by-project basis and are beyond the scope of this chapter.

The first part of the chapter, i.e., Section 20.1.2, describes general seismic design considerations including pertinent formulae, interpretation of Caltrans *SDC* provisions, and a procedural flowchart for seismic design of concrete bridges. In the second part, i.e., Section 20.1.3, a seismic design example of a three-span continuous cast-in-place, prestressed (CIP/PS) concrete box girder bridge is presented to illustrate various design applications following the seismic design procedure flowchart. The example is intended to serve as a model seismic design of an ordinary standard bridge using the current *SDC* Version 2.0 provisions.

20.1.2 DESIGN CONSIDERATIONS

20.1.2.1 Preliminary Member and Reinforcement Sizes

Bridge design is inherently an iterative process. It is common practice to design bridges for the Strength and Service Limit States and then, as necessary, to refine the design of various components to satisfy Extreme Events Limit States such as seismic performance requirements. Engineers should keep certain seismic requirements in mind even during the Strength and Service Limit States design. This is especially true while selecting the span configurations, column sizes, column reinforcements, and bent cap widths.

20.1.2.1.1 Sizing the Column and Bent Cap

(1) ***Column size***

For a superstructure with an integral bent cap, *SDC* Section 7.6.2 specifies that the column size should satisfy the following equations:

$$0.70 \le \frac{D_c}{D_s} \le 1.00 \qquad (SDC\ 7.6.2\text{-}1)$$

$$0.70 \le \frac{D_{ftg}}{D_c} \qquad (SDC\ 7.6.2\text{-}2)$$

where:

D_c = column cross sectional dimension in the direction of loading (in.)

D_s = depth of superstructure at the bent cap (in.)

D_{ftg} = depth of footing (in.)

If $D_c > D_s$, it may be difficult to accommodate joint shear stresses in the column-bent cap connections.

(2) ***Bent Cap Width***

SDC Section 7.4.3 specifies the minimum cap width required for adequate joint shear transfer shall be taken:

$$B_{cap} = D_c + 24 \quad \text{(in.)} \qquad (SDC\ 7.4.3\text{-}1)$$

20.1.2.1.2 Column Reinforcement Requirements

(1) ***Longitudinal Reinforcement***

Maximum Longitudinal Reinforcement Area, $A_{st,\max} = 0.04A_g$ (*SDC* 5.3.9.1-1)

Minimum Longitudinal Reinforcement Area: $A_{st,\min} = 0.01A_g$ (*SDC* 5.3.9.2-1)

where:

A_g = the column gross cross-sectional area (in.^{2})

Normally, choosing column $A_{st} = 0.015A_g$ is a good starting point.

(2) ***Transverse Reinforcement***

Either spirals or hoops can be used as transverse reinforcement in circular column. For rectangular column and pier wall ties and crossties are used. However, hoops are preferred because of their discrete nature in the case of local failure.

- Inside the Plastic Hinge Region

The amount of transverse reinforcement inside the analytical plastic hinge region (see *SDC* Section 5.3.4 for equivalent plastic hinge length formulas), expressed as volumetric ratio, ρ_s, shall be sufficient to ensure that the column meets the performance requirements as specified in *SDC* Section 3.5.

$$\rho_s = \frac{4(A_b)}{D'(s)} \text{ for columns with circular or interlocking cores} \qquad (SDC\ C5.3.8.2\text{-}1)$$

For rectangular columns with ties and cross ties, the corresponding equation for ρ_s , is:

$$\rho_s = \frac{A_v}{D_c^{'} s} \quad (SDC\ C5.3.8.2\text{-}2)$$

where:

A_v = sum of area of the ties and cross ties running in the direction perpendicular to the axis of bending (in.^{2})

A_b = area of individual confinement steel bar (in.^{2})

$D^{'}$ = cross-sectional dimension of concrete core measured between the centerlines of the peripheral hoop or spiral (in.)

$D_c^{'}$ = confined column cross-section dimension, measured out to out of ties, in the direction parallel to the axis of bending (in.)

s = spacing of transverse reinforcement spacing measured along the longitudinal axis of the SCM (in.)

SCM = A ductile structural member intentionally designed to deform inelastically through several cycles without significant degradation of strength.

In addition, the transverse reinforcement should meet the column shear requirements as specified in *SDC* Section 5.3.7.3 and 5.3.8

- Outside the Plastic Hinge Region

As specified in *SDC* Section 5.3.8.3, the volume of lateral reinforcement outside the plastic hinge region shall not be less than 50 % of the minimum amount required inside the plastic hinge region and meet the shear requirements.

(3) *Spacing Requirements*

The selected bar layouts should satisfy the following spacing requirements for effectiveness and constructability:

- Longitudinal Reinforcement

Maximum and minimum spacing requirements are given in *AASHTO* Article 5.10 (2017) and SDC 8.4.2

- Transverse Reinforcement

According to *SDC* Section 8.4.1, the maximum spacing inside the plastic hinge region shall not exceed the smallest of:

- 6 times the nominal diameter of the longitudinal reinforcement
- 8 in.

Outside this region, the hoop spacing can be and should be increased to economize the design.

20.1.2.1.3 Frame and Bent Proportioning

(1) *Balanced Stiffness*

For an acceptable seismic response, a structure with well-balanced mass and stiffness across various frames is highly desirable. Such a structure is likely to respond to a seismic activity in a simple mode of vibration and any structural damage will be well distributed among all the columns. The best way to increase the likelihood that the structure responds in its fundamental mode of vibration is to balance its stiffness and mass distribution. *SDC* recommends that the ratio of effective stiffness between any two bents within a frame or between any two columns within a bent satisfy the following:

$$\frac{k_i^e}{k_j^e} \geq 0.5 \qquad \text{for a constant width frame}$$

$$0.5 \leq \left(\frac{\dfrac{k_i^e}{m_i}}{\dfrac{k_j^e}{m_j}} \right) \leq 2.0 \qquad \text{for a variable width frame} \qquad (SDC\ 7.1.2\text{-}1)$$

SDC further recommends that the ratio of effective stiffness between *adjacent* bents within a frame or between *adjacent* columns within a bent satisfies the following:

$$\frac{k_i^e}{k_j^e} \geq 0.75 \qquad \text{for a constant width frame}$$

$$0.75 \leq \left(\frac{\dfrac{k_i^e}{m_i}}{\dfrac{k_j^e}{m_j}} \right) \leq 1.33 \qquad \text{for a variable width frame} \qquad (SDC\ 7.1.2.2)$$

where:

k_i^e = effective stiffness of bent or column *i* (kip/in.)

m_i = tributary mass of column or bent *i* (kip-sec^2/in)

k_j^e = effective stiffness of bent or column j (kip/in.)

m_j = tributary mass of column or bent j (kip-sec^2/in)

Bent stiffness shall be based on effective material properties for each of the framing components and shall include the effects of foundation flexibility if it is determined to be significant by the Geotechnical Engineer.

If the requirements of a balanced effective stiffness are not met, some of the undesired consequences include:

- The stiffer bent or column will attract more force and hence will be susceptible to increased damage
- The inelastic response will be distributed non-uniformly across the structure
- Increased column torsion demands may be generated by rigid body rotation of the superstructure

(2) *Material and Effective Column Section Properties*

To estimate member ductility, column effective section properties as well as the moment-curvature (M-ϕ) relationship are determined by using a computer program such as CSI *Section Designer*, *xSECTION, CTColumn* or similar tool. Effort should be made to keep the dead load axial force ratio in columns to about 10%. The axial load ratio for SCMs in compression shall be less or equal to 15%, to ensure that the column does not experience brittle compression failure and also to ensure that any potential P-Δ effects remain within acceptable limits.

<u>Material Properties</u>

- Concrete

Concrete compressive strength f'_c = 4,000 psi is commonly used for superstructure, columns, piers, and pile shafts. For elastic bridge components like abutments, wingwalls, and footings, f'_c = 3,600 psi is typically specified.

SDC Section 3.3 requires that expected material properties shall be used to calculate section capacities for all ductile members. To be consistent between the demand and capacity, expected material properties shall be used to calculate member stiffness. For concrete, the expected compressive strength, f'_{ce}, is taken as:

$$f'_{ce} = 1.3 f'_c \text{ but not less than 5000 psi} \qquad (SDC\ 3.3.6\text{-}4)$$

Other concrete properties are listed in *SDC* Table 3.3.6-1

- Steel

  Grade A706/A706M is typically used for reinforcing steel bar. Material properties for Grade A706/A706M steel are given in *SDC* Table 3.3.3-1

Effective Moment of Inertia

It is known that concrete cover spalls off at very low ductility levels. Therefore, the effective (cracked) moment of inertia value is used to assess the seismic response of all ductile members. Per SDC 3.4.2-1 effective moment of inertia is obtained from a moment-curvature analysis of the member cross-section, see *SDC* (figure 5.3.6.2-1).

20.1.2.1.4 Balanced Frame Geometry

SDC Section 7.1.3 requires that the ratio of fundamental periods of vibration for adjacent frames in the longitudinal and transverse directions satisfy:

$$0.7 \le \frac{T_i}{T_j} \le 1.43 \qquad (SDC\ 7.1.3\text{-}1)$$

where:

T_i = natural period of frame i (sec.)

T_j = natural period of frame j (sec.)

The consequences of not meeting the ratio of fundamental periods requirements of *SDC* Equation 7.1.3-1 include a greater likelihood of out-of-phase response between adjacent frames leading to large relative displacements that increase the probability of longitudinal unseating and pounding between frames at the expansion joints.

For bents/frames that do not meet the *SDC* requirements for fundamental period of vibration and/or balanced stiffness, one or more of the following techniques (see *SDC* Section C7.1.2) may be employed to adjust the dynamic characteristics:

- Use oversized shafts
- Adjust the effective column length. This may be achieved by lowering footings or using isolation casings.
- Modify column end conditions
- Redistribute superstructure mass
- Vary column cross section and longitudinal reinforcement ratios
- Add or relocate columns
- Modify the hinge/expansion joint layout by moving the bents or frames.
- Use isolation bearings or dampers

20.1.2.2 Minimum Local Displacement Ductility Capacity

Before refining a comprehensive analysis to consider the effects of changes in column axial forces (for multi-column bents) due to seismic overturning moments and the effects of soil overburden on column footings, it is a good practice to ensure that basic *SDC* ductility requirements are met. *SDC* Section 5.2 for bridges where Equivalent Static Analysis (ESA) may be used to determine displacement demand as specified in Sections 4.2 and 4.2.1, "Local Displacements Capacity" equations may be used as the Inelastic Static Analysis (ISA). In previous editions of the *SDC*, a minimum local displacement ductility capacity of 3.0 was specified. *SDC* 2.0 recommends that the minimum volumetric ratios of transverse reinforcement specified in Table 5.3.8.2-1 shall be satisfied to provide a minimum local displacement ductility capacity equal to or greater than 3.0.

$$\Delta_c = \Delta_Y^{col} + \Delta_p \quad \text{(\textit{SDC} C5.2.2-1)}$$

$$\Delta_Y^{col} = \frac{L^2}{3}(\phi_Y) \quad \text{(\textit{SDC} C5.2.2-2)}$$

$$\Delta_p = \theta_p \left(L - \frac{L_p}{2} \right) \quad \text{(\textit{SDC} C5.2.2-3)}$$

$$\theta_p = L_p \phi_p \quad \text{(\textit{SDC} C5.2.5-4)}$$

$$\phi_p = \phi_u - \phi_Y \quad \text{(\textit{SDC} C5.2.2-5)}$$

$$\mu_D = \frac{\Delta_D}{\Delta_{Y(i)}}$$

$$\mu_C = \frac{\Delta_C}{\Delta_{Y(i)}}$$

where:

L = distance from the point of maximum moment to the point of contra-flexure (in.)

L_p = equivalent analytical plastic hinge length as defined in *SDC* Section 5.3.4 (in.)

Δ_p = idealized plastic displacement capacity due to rotation of the plastic hinge (in.)

Δ_Y^{col} = idealized yield displacement of the column at the formation of the plastic hinge (in.)

ϕ_Y = idealized yield curvature defined by an elastic-perfectly-plastic of the cross-section's *M-ϕ* curve, see *SDC* Figures C5.2.2-1 and C5.3.6.2-1 (rad/in.)

ϕ_p = idealized plastic curvature capacity (assumed constant over L_p) (rad/in.)

θ_p = plastic hinge rotation capacity (radian)

ϕ_u = curvature capacity when the concrete strain reaching ε_{cu} or the longitudinal reinforcing steel reaching the reduced ultimate strain ε_{su}^{R} (rad/in.)

μ_c = local displacement ductility capacity of SCM

μ_d = local displacement ductility demand of SCM

It is Caltrans' practice to use an idealized bilinear *M-ϕ* curve to estimate the idealized yield displacement demands and the global displacement capacities of ductile members.

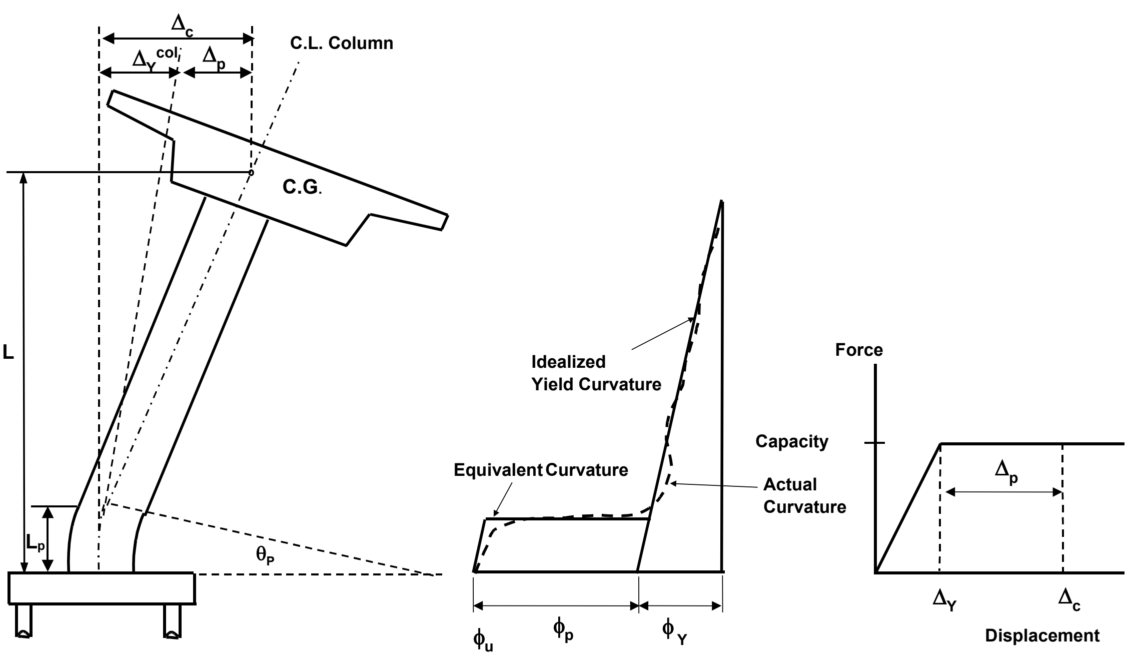


Figure *SDC* C5.2.2-1 Local Displacement Capacity of a Typical – Cantilever Column with Fixed Base

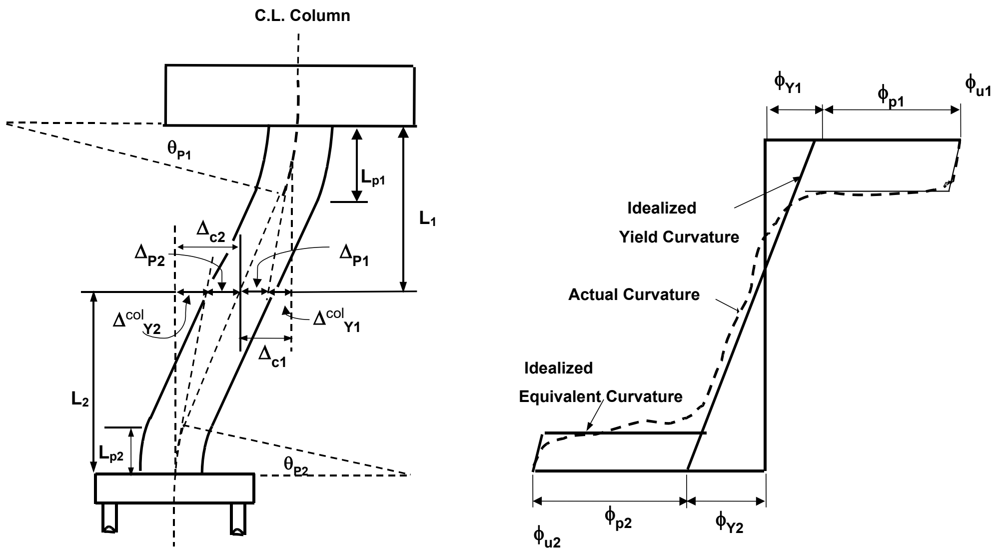


Figure *SDC* C5.2.2-2 Local Displacement Capacity of a Typical Fixed-Fixed Column

20.1.2.3 Displacement Ductility Demand Requirements

The displacement ductility demand is defined as

$$\mu_D = \frac{\Delta_D}{\Delta_{Y(i)}} \qquad (SDC\ 4.4.1\text{-}1)$$

where:

Δ_D = frame or bent displacement demand in the local principle/critical axis of member.

$\Delta_{Y(i)}$ = the yield displacement of the frame or bent from its initial position to the formation of plastic hinge (*i*)

To economize the plastic response of ductile members and minimize the demand imparted to adjacent capacity protected components, *SDC* Table 4.4.1-1 specifies target upper limits of displacement ductility demand values, μ_D, for various bridge components.

Single Column Bents supported on a footing or Type II shaft $\Delta_D \leq 4$

Multi-Column Bents supported on a footing or Type II shaft $\Delta_D \leq 5$

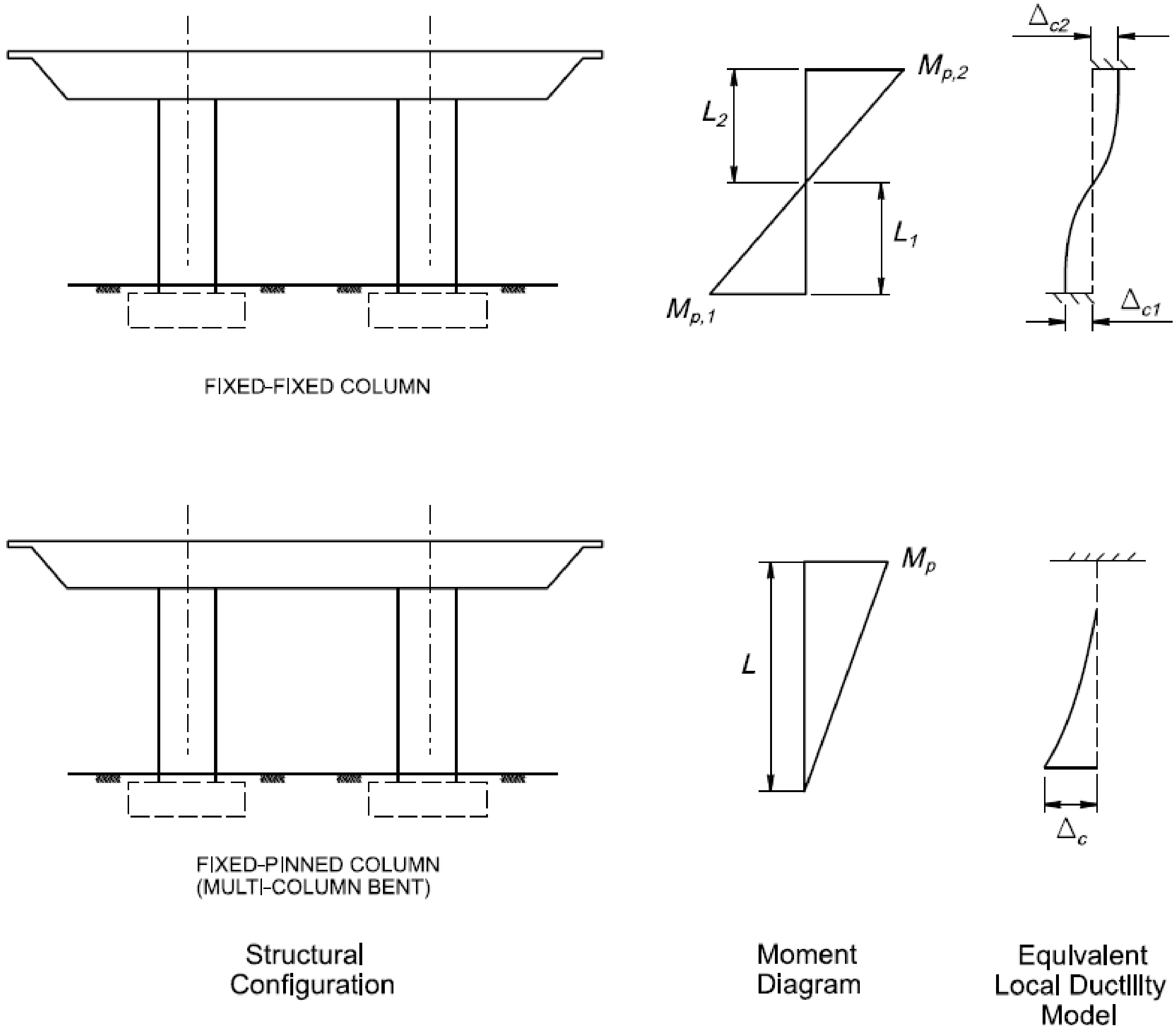
In addition, *SDC* Section 4.1 requires each bridge or frame to satisfy the following equation:

$$\Delta_C \geq \Delta_D \qquad (SDC\ 3.5.1\text{-}1)$$

where:

Δ_C = frame or bent displacement capacity in the local principle/critical axis of member.

The seismic demand can be estimated using Equivalent Static Analysis (ESA). As described in *SDC* Section 4.2.1, this method is most suitable for structures with well-balanced spans and uniformly distributed stiffness where the response can be captured by a simple predominantly translational mode of vibration. Effective properties shall be used to obtain the structure's period and displacement demands.



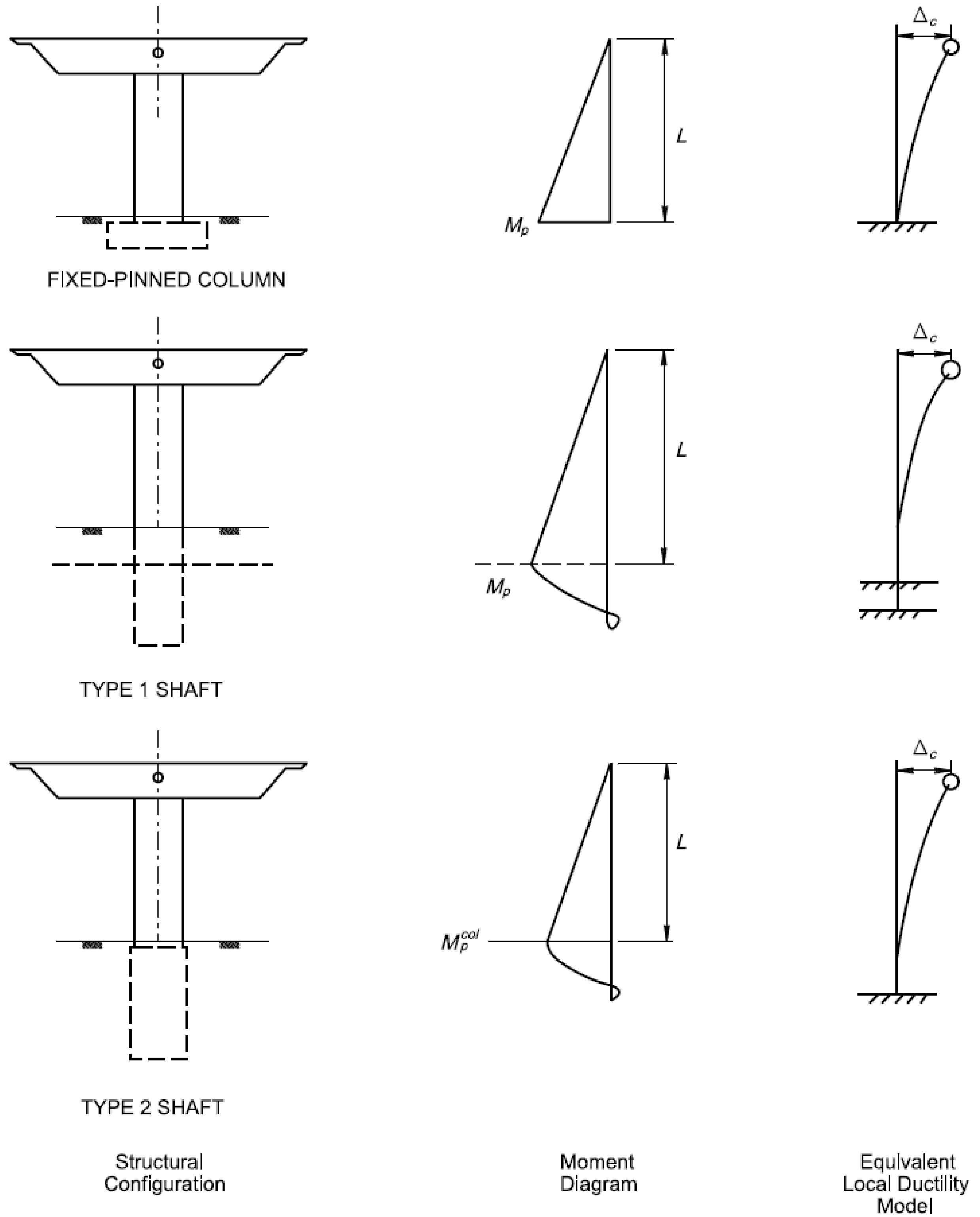


Figure *SDC* 5.2.2-3 Local Ductility Assessment

For SDOF system, the displacement demand, Δ_D, can be calculated from (*SDC* 4.2.1-1)

$$\Delta_D = \frac{WS_a}{K}$$

where:

W = tributary weight of the structure

S_a = design spectral acceleration coefficient at the structure period

K = effective stiffness of the bent or frame

For ordinary bridges that do not meet the criteria for ESA or where ESA does not provide an adequate level of sophistication to estimate the dynamic behavior, Elastic Dynamic Analysis (EDA) may be used. Refer to *SDC* Section 4.2.2 for more details regarding EDA.

20.1.2.4 Displacement Capacity Evaluation

SDC Section 5.2.2 specifies the use of Inelastic Static Analysis (ISA), or “pushover” analysis, to determine reliable displacement capacities of a structure or frame. ISA captures non-linear bridge responses such as the yielding of ductile components as well as the effects of foundation flexibility. The effect of soil-structure interaction can be significant in the case where footings are buried deep in the ground.

Pushover analysis shall be performed using the expected material properties of modeled members to provide a more realistic estimate of design strength. As required by *SDC* Section 5.4, capacity protected concrete components such as bent caps, superstructures, and footings shall be designed to remain essentially elastic when the column reaches its overstrength capacity. This is required in order to ensure that no plastic hinge forms in the capacity protected components.

Computer programs such *CSI Bridge*, *wFRAME,* or similar tools may be used to perform a pushover analysis. The following conventions are applicable to both the transverse and longitudinal analyses:

- The model is two or three-dimensional with beam elements along the c.g. of the superstructure/bent cap and columns.
- The dead load of superstructure/bent cap, and of columns, if desired, is applied as a uniformly distributed load along the length of the superstructure/bent cap.
- The element connecting the superstructure c.g. to the column end point at the soffit level is modeled as a super stiff element with stiffness much greater than the regular column section. The moment capacity for such element is also specified much higher than the plastic moment capacity of the column. This is done to ensure that for a column-to-superstructure rigid connection, the plastic hinge forms at the top of the column below the superstructure soffit.
- The soil resistance effect can be included as *p-y*, *t-z* and *q-z* springs.

Though “pushover” is mainly a capacity estimating procedure, it can also be used to estimate demand for structures having characteristics outlined previously in Section 20.1.2.3.

20.1.2.4.1 Foundation Soil Springs

The *p-y* curves are used in the lateral modeling of soil as it interacts with the bent/column foundations. The Geotechnical Engineer generally produces these curves, the values of which are converted to soil springs within the pushover analysis. The spacing of the nodes on the substructure changes each individual spring stiffness, however, a minimum of 10 elements per pile is advised (recommended optimum is 20 elements or 2 ft to 5 ft pile segments).

The *t-z* curves are used in the modeling of skin friction along the length of piles. Vertical springs are attached to the nodes to support the dead load of the bridge system and to resist overturning effects caused by lateral bridge movement. The bearing reaction at tip of the pile is usually modeled as a *q-z* spring.

20.1.2.4.2 Transverse Pushover Analysis

During the transverse movement of a multi-column frame, a cap beam provides a framing action. As a result of this framing action, the column axial force can vary significantly from the dead load state. If the seismic overturning forces are large, the top of the column might even go into tension.

With the changes in column axial loads, the section properties (M_p , I_e, ϕ_y *and* ϕ_p) should be updated and a subsequent iteration of the *CSI Bridge* or *wFRAME* program performed.

20.1.2.4.3 Longitudinal Pushover Analysis

Although the process of calculating the section capacity and estimating the seismic demand is similar for the transverse and longitudinal directions, there are some significant differences. For a longitudinal push analysis:

- For prestressed superstructures, gross moment of inertia is used for the superstructure
- Bent overturning is insignificant
- The abutment is modeled as a linear spring whose effective stiffness is calculated as described in this Section.

If the column or pier cross-section is rectangular, section properties along the longitudinal direction of the bridge as shown in Figure 20.1.2-1 must be calculated and used. *CSI Bridge* Section Designer, *xSECTION* or similar tools can be used. This can be achieved by specifying in the angle between the column section coordinate system and the longitudinal direction of the bridge as shown in the sketch below.

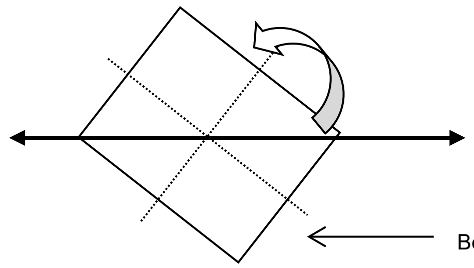


Figure 20.1.2-1 Bridge Longitudinal Direction

It is Caltrans' practice to design the seated abutment backwall so that it breaks off in shear during a seismic event. *SDC* Section 6.3.1 specifies that the linear elastic demand shall include an effective abutment stiffness that accounts for expansion gaps and incorporates a realistic value for the embankment fill response. The backfill passive force resisting movement at the abutment varies nonlinearly with longitudinal abutment displacement and is dependent upon the material properties of the backfill.

$$K_{abut} = w_{abut}\left(5.5h_{abut} + 20\right)R_{sk} \qquad \text{(SDC 6.3.1.2-5)}$$

where:

w_{abut} = projected width of the backwall or diaphragm for seat and diaphragm abutments, respectively (ft)

h_{abut} = height of the backwall or diaphragm for seat and diaphragm abutments, respectively (ft)

The passive pressure resisting movement at the abutment, F_{abut}, is given as:

$$F_{abut} = w_{abut}\left(\frac{5.5h_{abut}^{2.5}}{1 + 2.37h_{abut}}\right)R_{sk} \qquad \text{(SDC 6.3.1.2-4)}$$

where:

$$R_{sk} = e^{-\frac{\theta}{45}} \qquad \text{(SDC 6.3.1.2-6)}$$

$$\theta \le 66^\circ \qquad \text{(SDC 6.3.1.2-7)}$$

The above terms are defined in *SDC* 6.3.1.2 and Figures 6.3.1.2-2 and 6.3.1.2-3.

SDC Section 6.3.1.3 specifies that the effectiveness of the abutment shall be assessed by the abutment displacement coefficient:

$$R_A = \Delta_D / \Delta_{eff} \qquad \text{(SDC 6.3.1.3-1)}$$

where:

R_A = abutment displacement coefficient

Δ_D = the longitudinal displacement demand at the abutment from elastic analysis (in.)

Δ_{eff} = the effective longitudinal abutment displacement when the passive force reaches F_{abut} (in.)

Details on the interpretation and use of the coefficient R_A value are given in *SDC* Section 6.3.1.3.

20.1.2.5 P-Δ Effects

In lieu of a rigorous analysis to determine *P-Δ* effects, *SDC* recommends that such effects can be ignored if the following equation is satisfied:

$$P_{dl}\Delta_r \leq 0.25M_p^{col} \qquad (SDC\ 4.4.4\text{-}1)$$

where:

M_p^{col} = idealized plastic moment capacity of a column calculated from *M-Φ* analysis

P_{dl} = axial force attributed to dead load

Δ_r = relative lateral offset between the base of the plastic hinge and the point of contra-flexure as shown in *SDC* Figure 4.4.4-1.

20.1.2.6 Minimum Flexural Strength

SDC Section 5.3.6.1 specifies that each bent shall have a minimum plastic moment capacity (based on expected material properties) to resist a lateral force of $0.1P_{dl}$, where P_{dl} is the tributary dead load applied at the center of gravity of the superstructure.

20.1.2.7 Column Shear Design

The seismic shear demand shall be based upon the overstrength shear V_o, associated with the column overstrength moment M_0^{col}. Since shear failure tends to be brittle, shear capacity for ductile members shall be conservatively determined using the minimum specified material properties as follows:

$$\phi V_n \geq V_o^{col} \qquad (SDC\ 5.3.7.1\text{-}1)$$

where:

$$V_n = V_c + V_s \qquad (SDC\ 5.3.7.1\text{-}2)$$

$$\phi = 1.00$$

20.1.2.7.1 Shear Demand

Shear demand associated with overstrength moment may be calculated from:

$$V_o^{col} = \frac{M_0^{col}}{L}$$

where:

$$M_0^{col} = 1.2M_p^{col} \qquad (SDC\ 4.4.2.1\text{-}1)$$

L = length of column from plastic hinge to contraflexure (zero moment)

Alternately, the maximum shear demand may be determined from CSI Bridge or wFRAME pushover analysis results. The maximum column shear demand obtained from the analysis is multiplied by a factor of 1.2 to obtain the shear demand associated with the overstrength moment.

20.1.2.7.2 Concrete Shear Capacity

$$V_c = \nu_c A_e \qquad (SDC\ 5.3.7.2\text{-}1)$$

where:

$$A_e = (0.8)A_g \qquad (SDC\ 5.3.7.2\text{-}2)$$

$$\nu_c = F_1 F_2 \sqrt{f_c^{'}} \le 4\sqrt{f_c^{'}} \quad \text{(Inside the plastic hinge region)} \qquad (SDC\ 5.3.7.2\text{-}3)$$

$$= 3F_2 \sqrt{f_c^{'}} \le 4\sqrt{f_c^{'}} \quad \text{(Outside the plastic hinge region)} \qquad (SDC\ 5.3.7.2\text{-}4)$$

$$0.3 \le F_1 = \frac{\rho_s f_{yh}}{0.150} + 3.67 - \mu_d \le 3 \qquad (f_{yh} \text{ in ksi}) \qquad (SDC\ 5.3.7.2\text{-}5)$$

Where the value of $\rho_s f_{yh} \le 0.35\text{ksi}$

$$F_2 = 1 + \frac{P_c}{2{,}000A_g} < 1.5 \quad \{P_c\ (\text{lbs}),\ A_g\ (\text{in.}^2)\} \qquad (SDC\ 5.3.7.2\text{-}6)$$

20.1.2.7.3 Transverse Reinforcement Shear Capacity V_s

$$V_s = \left(\frac{A_v f_{yh} D'}{s}\right) \qquad (SDC\ 5.3.7.3\text{-}1)$$

where:

$$A_v = n\left(\frac{\pi}{2}\right)A_b \qquad (SDC\ 5.3.7.3\text{-}2)$$

n = number of individual interlocking spiral or hoops in the core of the sections

20.1.2.7.4 Maximum Shear Reinforcement Strength, V_s

$$V_s \le 8\sqrt{f_c'}A_e \quad \text{(psi)} \qquad (SDC\ 5.3.7.4\text{-}1)$$

20.1.2.7.5 Minimum Shear Reinforcement

$$A_v \ge 0.025\frac{D's}{f_{yh}} \quad \left(\text{in.}^2\right) \qquad (SDC\ 5.3.7.5\text{-}1)$$

20.1.2.7.6 Column Key Design

Shear keys in pinned column connections shall be designed for the axial shear forces associated with the column's overstrength moment (M_0^{col}) including the effects of overturning. The moment generated by the reduced rebars pin connection must be included in the calculation of the overstrength column shear V_o^{col}.

$$V_o^{col} = \frac{(M_0^{coltop} + M_0^{colbot})}{L}$$

The area of interface shear key reinforcement, A_{sk} shall be taken as:

$$A_{sk} = \frac{1.2\left(V_o^{col} - 0.25P\right)}{f_y} \quad \text{if } P \text{ is compressive} \qquad (SDC\ 7.6.4\text{-}1)$$

$$A_{sk} = \frac{1.2\left(V_o^{col} + P\right)}{f_y} \quad \text{if } P \text{ is tensile} \qquad (SDC\ 7.6.4\text{-}2)$$

where:

$$A_{sk} \ge 4 \text{ in.}^2$$

V_o^{col} = shear force associated with the column overstrength moment, including overturning effects (kip)

P = absolute value of the net axial force normal to the shear plane (kip). P shall be equal to the lowest axial load if compressive or greatest axial load if tensile, considering the effects of overturning.

The hinge shall be proportioned such that the area of concrete engaged in interface shear transfer, A_{cv} satisfies the following equations:

$$A_{cv} \geq \frac{4.0V_o^{col}}{f_c^{'}} \qquad (SDC\ 7.6.4\text{-}3)$$

$$A_{cv} \geq 0.67V_o^{col} \qquad (SDC\ 7.6.4\text{-}3)$$

In addition, the area of concrete section used in the pin-hinge shall satisfy the axial resistance requirements as specified in Section 5 of AASHTO-CA BDS based on the column with the greatest axial load and a resistance factor ϕ of 1.0.

20.1.2.8 Bent Cap Flexural and Shear Capacity

According to *SDC* Section 5.4, a bent cap is considered a capacity protected member and shall be designed flexurally to remain essentially elastic when the column reaches its overstrength capacity. The expected nominal moment capacity M_{ne} for capacity protected members may be determined either by a traditional strength method or by a more complete M-ϕ analysis. The expected nominal moment capacity shall be based on expected concrete and steel strength values when either concrete strain reaches 0.003 or the steel strain reaches ε_{SU}^{R} as derived from the applicable stress-strain relationship. The nominal shear capacity of the bent cap shall be calculated in accordance with AASHTO-CA BDS provisions using the specified minimum material properties.

The seismic flexural and shear demands in the bent cap are calculated corresponding to the column overstrength moment. These demands are obtained from a pushover analysis with and then compared with the nominal flexural and shear capacity of the bent cap.

20.1.2.9 Seismic Strength of Concrete Bridge Superstructures

When moment-resisting superstructure-to-column details are used, seismic forces of significant magnitude are induced into the superstructure. If the superstructure does not have adequate capacity to resist such forces, unexpected and unintentional hinge formation may occur in the superstructure leading to potential failure of the superstructure. According to *SDC* Sections 5.4 and 4.4.3, a capacity design approach is adopted to ensure that the superstructure has an appropriate strength reserve above demands generated from probable column plastic hinging. Determination of the seismic demands and capacities in the superstructure are illustrated in the design example.

20.1.2.9.1 General Assumptions

The assumptions of calculating seismic demands in the superstructure are made as follows:

- The superstructure demands are based upon complete plastic hinge formation in all columns or piers within the frame.
- Effective section properties shall be used for modeling columns or piers while gross section properties may be used for superstructure elements.
- Additional column axial force due to overturning effects shall be considered when calculating effective section properties and the idealized plastic moment capacity of columns and piers.
- Superstructure dead load and secondary prestress demands are assumed to be uniformly distributed to each girder, except in the case of highly curved or highly skewed structures.

While assessing the superstructure member demands and available section capacities, an effective width, B_{eff} as defined in *SDC* Section 7.2.1.1 will be used.

$$B_{eff} = \begin{cases} D_c + 2D_s & \text{Box girders and slab superstructures} \\ D_c + D_s & \text{Open soffit superstructures} \end{cases} \qquad (SDC\ 7.2.1.1\text{-}1)$$

where:

D_c = column cross sectional dimension in the transverse direction (in.)

D_s = depth of the superstructure (in.)

20.1.2.9.2 Superstructure Seismic Demand

The force demand in the superstructure corresponds to its Collapse Limit State. The Collapse Limit State is defined as the condition when all the potential plastic hinges in the columns and/or piers have been formed. When a bridge reaches such a state during a seismic event, the following loads are present: Dead Loads, Secondary Forces from Post-tensioning (i.e., prestress secondary effects), and Seismic Loads. Since the prestress tendon is treated as an internal component of the superstructure and is included in the member strength calculation, only the secondary effects which are caused by the support constraints in a statically indeterminate prestressed frame are considered to contribute to the member demand.

The procedure for determining extreme seismic demands in the superstructure considers each of these load cases separately and the final member demands are obtained by superposition of the individual load cases.

Since different tools may be used to calculate these demands, it is very important to use a consistent sign convention while interpreting the results.

Prior to the application of seismic loading, the columns are “pre-loaded” with moments and shears due to dead loads and secondary prestress effects. At the Collapse Limit State, the “earthquake moment” applied to the superstructure may be greater or less than the overstrength moment capacity of the column or pier depending on the direction of these “pre-load” moments and the direction of the seismic loading under consideration.

Figure 20.1.2-3 shows schematically this approach of calculating columns seismic forces.

Due to the uncertainty in the magnitude and distribution of secondary prestress moments and shears at the extreme seismic limit state, it is conservative to consider such effects only when their inclusion results in increased demands in the superstructure.

Once the column moment, M_{eq}, is known at each potential plastic hinge location below the joint regions, the seismic moment demand in the superstructure can be determined using currently available Caltrans' analysis tools. One such method entails application of M_{eq} at the column-superstructure joints and then using computer program *CSI Bridge* (CSI 2021) to perform a pushover analysis until all the plastic hinges have formed to compute the moment demand in the superstructure members.

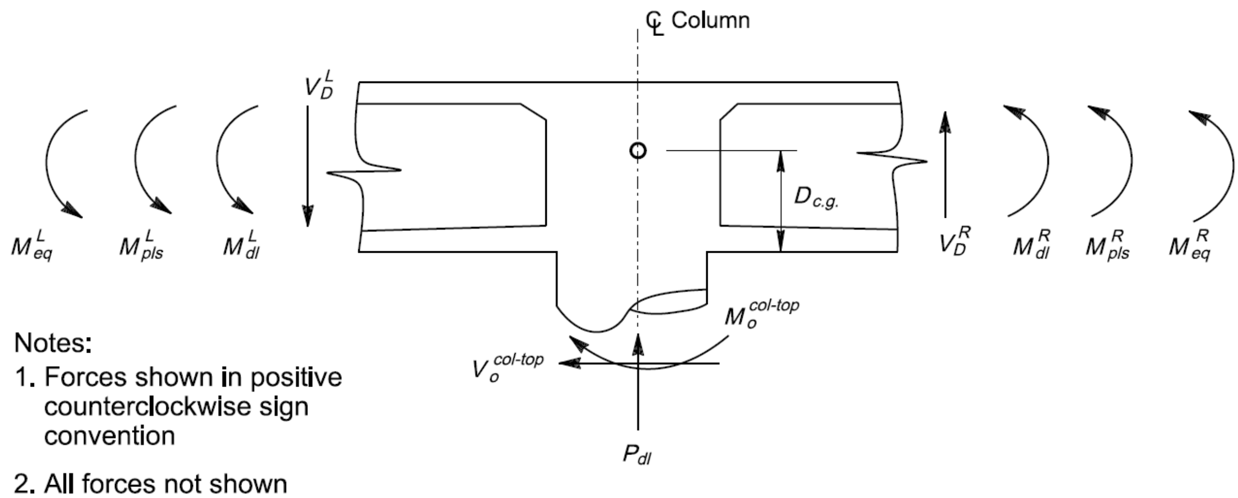


Figure 20.1.2-3 Superstructure Demand Generated by Column Overstrength Moment

Note that *CTBridge* is a three-dimensional analysis program where force results are oriented in the direction of each member's local axis.

(1) *Dead Load Moments, Additional Dead Load Moments, and Prestress Secondary Moments*

These moments are readily available from *CTBridge* output and are assumed to be uniformly distributed along each girder.

(2) *Earthquake Moments in the Superstructure* (*SDC* 4.4.3)

The aim here is to determine the amount of seismic loading needed to ensure that potential plastic hinges have formed in all the columns of the framing system. To form a plastic hinge in the column, the seismic load needs to produce a moment at the potential plastic hinge location of such a magnitude that, when combined with the "pre-loaded" dead load and prestress moments, the column will reach its overstrength plastic moment capacity, M_0^{col}.

$$M_0^{col-top} = M_{dl}^{col-top} + M_{ps}^{col-top} + M_{eq}^{col-top}$$

It should be kept in mind that dead load moments will have positive or negative values depending on the location along the span length. Also, the direction of seismic loading will determine the nature of the seismic moments.

Two cases of longitudinal earthquake loading shall be considered, namely,

(a) bridge movement to the right, and

(b) bridge movement to the left.

The column seismic load moments, $M_{eq}^{col-top}$, are calculated based upon the principle of superposition as follows:

$$M_{eq}^{col-top} = M_0^{col-top} - \left(M_{dl}^{col-top} + M_{ps}^{col-top}\right) \qquad \text{(}\textit{SDC}\text{ 4.4.3.1-3)}$$

In the above equation, the overstrength column moment M_o^{col} is given as:

$$M_o^{col} = 1.2 M_p^{col} \qquad \text{(}\textit{SDC}\text{ 4.4.2.1-1)}$$

(3) ***Earthquake Shear Forces in the Superstructure***

A procedure similar to that used for moments can be followed to calculate the seismic shear force demand in the superstructure. As in the case of moments, the shear forces in the superstructure member due to dead load, additional dead load, and secondary prestress are readily available from *CTBridge* output.

The superstructure seismic shear forces due to seismic moments can be calculated by using the previously computed values of the superstructure seismic moments, M_{eq}^L and M_{eq}^R, for each span.

(4) ***Moment and Shear Demand at Location of Interest***

The extreme seismic moment demand in the superstructure is calculated as the summation of all the moments obtained from the above sections, taking into account the proper direction of bending in each case as well as the effective section width. The superstructure demand moments at the adjacent left and right superstructure span are given by:

$$M_D^L = M_{dl}^L + M_{ps}^L + M_{eq}^L \qquad \text{(}\textit{SDC}\text{ 4.4.3.1-2)}$$

$$M_D^R = M_{dl}^R + M_{ps}^R + M_{eq}^R \qquad \text{(}\textit{SDC}\text{ 4.4.3.1-1)}$$

Similarly, the extreme seismic shear force demand in the superstructure is calculated as the summation of shear forces due to dead load, secondary prestress effects and the seismic loading, taking into account the proper direction of bending in each case and the effective section width. The superstructure demand shear forces at the adjacent left and right superstructure spans are defined as:

$$V_D^L = V_{dl}^L + V_{ps}^L + V_{eq}^L \qquad (SDC\ 4.4.3.2\text{-}2)$$

$$V_D^R = V_{dl}^R + V_{ps}^R + V_{eq}^R \qquad (SDC\ 4.4.3.2\text{-}1)$$

As stated previously in this section, the secondary effect due to the prestress will be considered only when it results in an increased seismic demand.

Dead load and secondary prestress moment and shear demand in the superstructure are proportioned on the basis of the number of girders falling within the effective section width. The earthquake moment and shear imparted by column is also assumed to act within the same effective section width.

(5) *Vertical Acceleration*

In addition to the superstructure demands discussed above, *SDC* Sections 3.2.1.4 and 7.2.2 require an equivalent static vertical load to be applied to the superstructure to estimate the effects of vertical acceleration in the case of sites with Peak Ground Acceleration (PGA) greater than or equal to 0.6g. For such sites, the effects of vertical acceleration may be accounted for by designing the superstructure to resist an additional uniformly applied vertical force equal to 25% of the dead load applied upward and downward. Just follow *SDC* 2.0 will be fine.

20.1.2.9.3 Superstructure Section Capacity

(1) *General*

To ensure that the superstructure has sufficient capacity to resist the extreme seismic demands determined in Section 20.1.2.9.2, *SDC* Section 5.4.4 requires the superstructure capacity in the longitudinal direction to be greater than the demand distributed to it (the superstructure) on each side of the column by the largest combination of dead load moment, secondary prestress moment, and column earthquake moment, i.e.,

$$\phi M_{ne}^{\sup(R)} \geq M_{dl}^R \pm M_{p/s}^R + M_{eq}^R \qquad (SDC\ 5.4.4\text{-}1)$$

$$\phi M_{ne}^{\sup(L)} \geq M_{dl}^L \pm M_{p/s}^L + M_{eq}^L \qquad (SDC\ 5.4.4\text{-}2)$$

$$\phi V_n^{\sup(R)} \geq V_{dl}^R \pm V_{p/s}^R + V_{eq}^R \qquad (SDC\ 5.4.4\text{-}3)$$

$$\phi V_n^{\sup(L)} \geq V_{dl}^L \pm V_{p/s}^L + V_{eq}^L \qquad (SDC\ 5.4.4\text{-}4)$$

where:

$M_{ne}^{\sup R,L}$ = expected nominal moment capacity of the adjacent right (*R*) or left (*L*) superstructure span at concrete strain of 0.003

$V_{n}^{\sup R,L}$ = expected nominal shear capacity of the adjacent right (*R*) or left (*L*) superstructure span

(2) *Superstructure Flexural Capacity*

Per *SDC* 5.4.4, expected material properties are used to calculate the flexural capacity of the superstructure. The member strength and curvature capacities are assessed using a stress-strain compatibility analysis. Failure is reached when either the ultimate concrete, mild steel or prestressing ultimate strain is reached. The internal resistance force couple is shown in Figure 20.1.2-4.

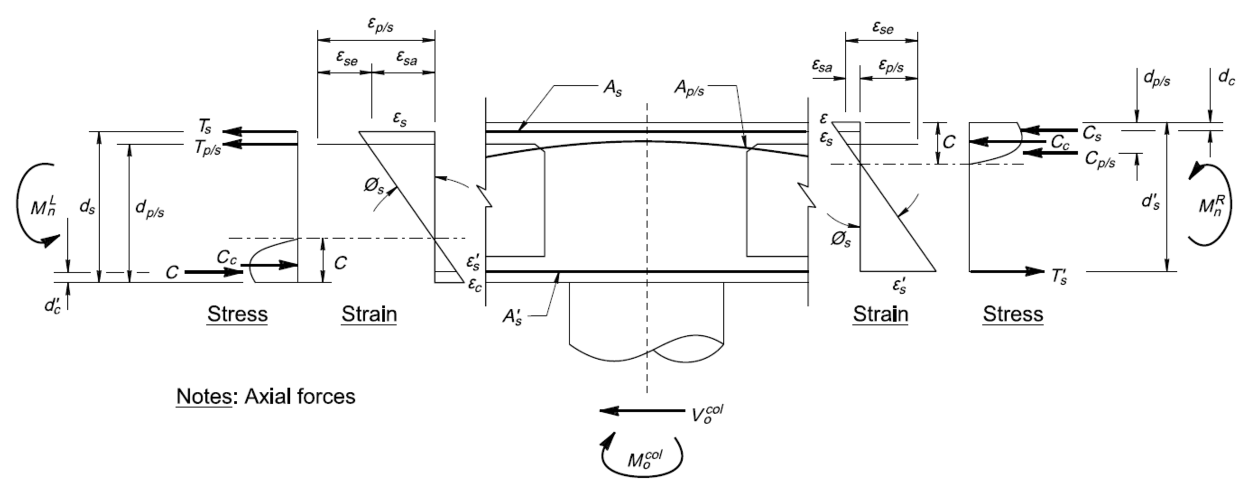


Figure 20.2-4 Superstructure Capacity Provided by Internal Couple

Caltrans in-house computer program *PSSECx* or similar program, may be used to calculate the section flexural capacity. The material properties for 270 ksi and 250 ksi prestressing strands are given in *SDC* Section 3.3.4. At locations where additional longitudinal mild steel is not required by analysis, a minimum of #8 bars spaced 12 in. (maximum spacing) should be placed in the top and bottom slabs at the bent cap. The mild steel reinforcement should extend beyond the inflection points of the seismic moment demand envelope.

As specified in *SDC* Section 5.4.2 the expected nominal moment capacity, M_{ne}, for capacity protected concrete components shall be determined by either M-ϕ analysis or strength design. Also, *SDC* Section 5.4.2 specifies that expected material properties shall be used in determining flexural capacity. Expected nominal moment capacity for capacity-protected concrete members shall be based on the expected concrete and steel strengths when either the concrete strain reaches its ultimate value based on the stress-strain model or the reduced ultimate prestress steel strain, ε_{su}^{R} =0.03 is reached.

In addition to these material properties, the following information is required for the capacity analysis:

- Eccentricity of prestressing steel - obtained from *CTBridge* output file. This value is referenced from the CG of the section.
- Prestressing force - obtained from *CTBridge* output file under the
- "*P/S* Response After Long-Term Losses" Tables.
- Prestressing steel area, A_{ps} - calculated for 270ksi steel as

$$A_{ps} = \frac{P_{jack}}{(0.75)(270)}$$

- Reinforcement in top and bottom slab, per design including #8 @12.
- Location of top and bottom reinforcement, referenced from center of gravity of section, slab steel section depth and assumed cover, etc.

Both negative (tension at the top) and positive (tension at the bottom) capacities are calculated at various sections along the length of the bridge by the *PSSECx* or similar computer program. The resistance factor for flexure, $\phi_{flexure}$ = 1.0, as we are dealing with extreme conditions corresponding to column overstrength.

(3) *Superstructure Shear Capacity*

SDC 5.4.3 specifies that the superstructure shear capacity is calculated according to AASHTO-CA BDS-8. As shear failure is brittle, the specified minimum material properties rather than expected material properties are used to calculate the shear capacity of the superstructure.

20.1.2.10 Joint Shear Design

20.1.2.10.1 General

(1) ***Principal Stresses***

In a ductility-based design approach for concrete structures, connections are key elements that must have adequate strength to maintain structural integrity under seismic loading. In moment resisting connections, the force transfer across the joint typically results in sudden changes in the magnitude and nature of moments, resulting in significant shear forces in the joint. Such shear forces inside the joint can be many times greater than the shear forces in individual components meeting at the joint.

SDC Section 7.4 requires that moment resisting connections between the superstructure and the column shall be designed to transfer the maximum forces produced when the column has reached its overstrength capacity, M_0^{col}, including the effects of overstrength shear V_0^{col}. Accordingly, *SDC* Section 7.4.2 requires all superstructure/column moment-resisting joints to be proportioned so that the principal stresses satisfy the following equations:

For principal tension, p_t: $p_t \le 12\sqrt{f_c'}$ (psi) (*SDC* 7.4.2-1)

For principal compression, p_c: $p_c \le 0.25 f_c'$ (psi) (*SDC* 7.4.2-2)

$$p_t = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \quad (SDC\ 7.4.2\text{-}3)$$

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \quad (SDC\ 7.4.2\text{-}4)$$

$$f_h = \frac{P_b}{B_{cap} D_s} \quad (SDC\ 7.4.2\text{-}5)$$

$$f_v = \frac{P_c}{A_{jh}} \quad (SDC\ 7.4.2\text{-}6)$$

$$v_{jv} = \frac{T_c}{A_{jv}} \quad (SDC\ 7.4.2\text{-}7)$$

$$A_{jh} = \left(D_c + D_s\right) B_{cap} \quad (SDC\ 7.4.2\text{-}8)$$

$$A_{jv} = l_{ac}(B_{cap}) \quad (SDC\ 7.4.2\text{-}9)$$

where:

f_h = average normal stress in the horizontal plane (ksi)

f_v = average normal stress in the vertical plane (ksi)

B_{cap} = bent cap width (in.)

D_c = cross sectional dimension of column in the direction of bending (in.)

D_s = depth of superstructure at the bent cap for integral joints (in.)

L_{ac} = length of column reinforcement embedded into the bent cap (in.)

P_c = column axial force including the effects of overturning (kip)

P_b = beam axial force at the center of the joint, including the effects of prestressing (kip)

T_c = column tensile force (defined as M_o^{col}/h) associated with the column overstrength plastic hinging moment, M_o^{col}. Alternatively, T_c may be obtained from the moment-curvature analysis of the cross section (kip)

h = distance from the center of gravity of the tensile force to the center of gravity of the compressive force of the column section (in.)

In the above equations, the value of f_h may be taken as zero unless prestressing is specifically designed to provide horizontal joint compression.

(2) *Minimum Bent Cap Width*

The minimum bent cap width required for adequate joint shear transfer shall be taken as:

$$B_{cap} = D_c + 24 \qquad (SDC\ 7.4.3\text{-}1)$$

(3) *Minimum Joint Shear Reinforcement*

SDC 7.4.5.1 specifies that, if the principal tensile stress, p_t is less than or equal to $3.5\sqrt{f_c'}$ (psi), only the minimum joint reinforcement is required, and the volumetric ratio of the transverse column reinforcement ($\rho_{s,\min}$) continued into the cap shall not be less than:

$$\rho_{s,\min} = \frac{3.5\sqrt{f_c'}}{f_{yh}} \quad \text{(psi)} \qquad (SDC\ 7.4.5.1\text{-}1)$$

If p_t is greater than $3.5\sqrt{f_c'}$, joint shear reinforcement shall be provided. The amount and type of joint shear reinforcement depend on whether the joint is classified as a "T" joint or a Knee Joint.

20.1.2.10.2 Joint Description

The following types of joints are considered as "T" joints for joint shear analysis (*SDC* Section 7.4.4):

- Interior joints of multi-column bents in the transverse direction
- All integral column-to-superstructure joints in the longitudinal direction
- Exterior column joints for box girder superstructures if the cap beam longitudinal reinforcement is fully beyond the exterior face of the column.

Any exterior column joint that satisfies the following equation shall be designed as a Knee joint. For Knee joints, it is also required that the main bent cap top and bottom bars be fully developed from the inside face of the column and extend as closely as possible to the outside face of the cap (see Figure *SDC* 7.4.4.2-1).

$$S < D_c \qquad (SDC\ 7.4.4.2\text{-}1)$$

where:

S = cap beam short stub length, defined as the distance from the exterior girder edge at soffit to the face of the column measured along the bent centerline (see Figure *SDC* 7.4.3-1),

D_c = diameter or cross section dimension of column

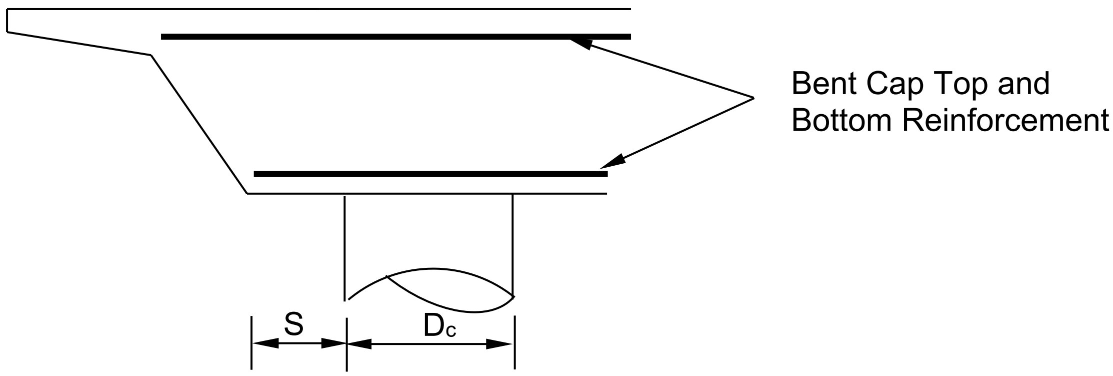


Figure *SDC* 7.4.4.2-1 Knee Joint Parameters

20.1.2.10.3 T Joint Shear Reinforcement

(1) *Vertical Stirrups in Joint Region*

Vertical stirrups or ties shall be placed transversely within a distance D_c extending from either side of the column centerline within the zones as indicated in SDC Figure 7.4.5.2-1. The required vertical stirrup area in each zone A_s^{jv} is given as

$$A_s^{jv} \geq 0.2 \times A_{st} \qquad (SDC\ 7.4.5.2\text{-}1)$$

Where:

A_{st} = total area of column longitudinal reinforcement anchored in the joint. For placement of the vertical stirrups see *SDC* Figures 7.4.5.2-1,2,3,4.

(2) *Horizontal Stirrups*

Horizontal stirrups or ties, A_s^{jh}, shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically with a 135-degree hook at not more than 18 inches.

$$A_s^{jh} = 0.1 \times A_{st} \qquad (SDC\ 7.4.5.2\text{-}2)$$

This horizontal reinforcement shall be placed within a distance D_c extending from either side of the column centerline. See *SDC* Figures 7.4.5.2-2,3,4.

(3) ***Horizontal Side Reinforcement***

The total longitudinal side face reinforcement in the bent cap shall satisfy:

$$A_s^{sf} \geq \max \begin{cases} 0.1 \times A_{cap}^{top} \\ \\ 0.1 \times A_{cap}^{bot} \end{cases} \qquad (SDC\ 7.4.5.2\text{-}3)$$

where:

A_{cap} = area of bent cap top or bottom flexural steel (in.^{2}).

The side reinforcement A_s^{sf} shall be placed near the side faces of the bent cap with a maximum spacing of 12 inches. Any side reinforcement placed to meet other requirements shall count towards meeting this requirement.

(4) ***J- Dowels***

For bents skewed more than 20^o, J- dowels hooked around the longitudinal top deck steel extend alternately 24 in. and 30 in. into the bent cap. The J-dowel reinforcement shall be equal to or greater than the area specified as:

$$A_s^{j-bar} \geq 0.08 A_{st} \qquad (SDC\ 7.4.5.2\text{-}4)$$

This reinforcement helps to prevent any potential delamination of concrete around deck top reinforcement. The J-dowels shall be placed within a rectangular region defined by the width of the bent cap and the distance D_c on either side of the centerline of the column.

(5) ***Transverse Reinforcement***

$$\rho_S^T = \frac{4A_b}{D's} \geq 0.4 \left[\frac{A_{st}}{l_{ac,provided}^2} \right] \qquad (SDC\ 7.4.5.2\text{-}5)$$

Transverse reinforcement in the joint region shall consist of hoops. The volumetric ratio of the hoops, ρ_S^T shall satisfy:

where:

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})

l_{ac} = actual length of column longitudinal reinforcement embedded into the bent cap (in.)

For interlocking cores, ρ_s^T shall be based on area of reinforcement A_{st} of each core.

All vertical column bars shall be extended as close as possible to the top bent cap reinforcement without interfering with post-tensioning duct configurations.

(6) ***Anchorage for Main Column Reinforcement***

The longitudinal column reinforcement shall be extended into cap beams and footings as close as possible to fully develop the compression strut mechanism in the joint. If the minimum joint shear reinforcement prescribed in *SDC* Equation 7.4.5.1-1 is met, and the column longitudinal reinforcement extension into the cap beam is confined by transverse hoops or spirals with the same volumetric ratio as that required at the top of the column, the anchorage for longitudinal column bars developed into the cap beam for seismic loads shall not be less than:

$$l_{ac} = 24d_{bl} \qquad (SDC\ 8.3.1.1\text{-}1)$$

For slab bridges, the development length are specified in *SDC* 9.2.

20.1.2.10.4 Knee Joint Shear Reinforcement

Knee joints may fail in either “opening” or “closing” modes (see Figure *SDC* C7.4.4.2-1). Therefore, both loading conditions must be evaluated. Refer to *SDC* Section C7.4.4.2 for the description of Knee joint failure modes.

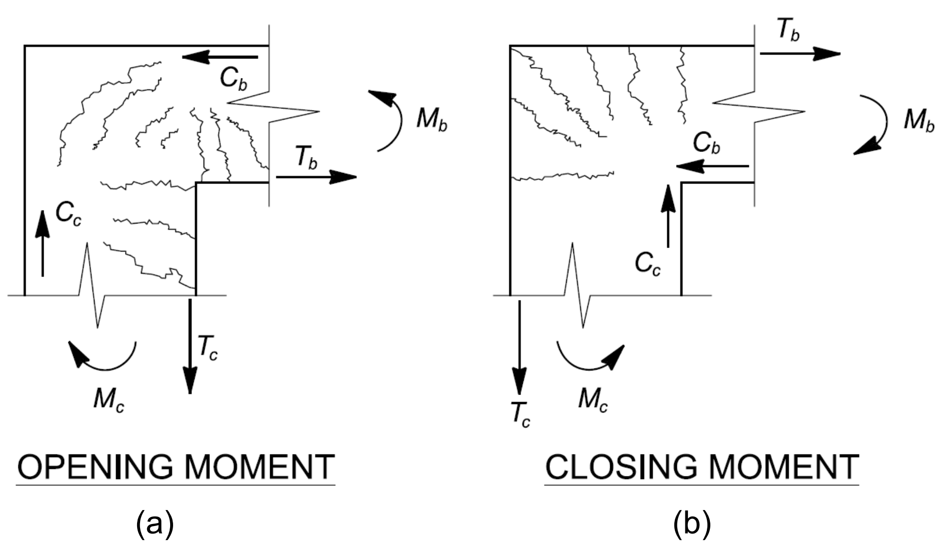


Figure *SDC* C7.4.4.2-1 Knee Joint Failure Modes

Two cases of Knee joints are identified as follows:

Case 1: $S < \frac{D_c}{2}$ (*SDC* 7.4.4.2-2)

Case 2: $\frac{D_c}{2} \leq S < D_c$ (*SDC* 7.4.4.2-3)

The following reinforcement is required for Knee joints.

A) *Bent Cap Top and Bottom Flexural Reinforcement - Use for both Cases 1 and 2*

The top and bottom reinforcement within the bent cap width used to meet this provision shall be in the form of continuous U-bars with minimum area:

$$A_s^{u-bar} \geq 0.33 A_{st}$$ (*SDC* 7.4.5.3-1)

where:

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})

The "U" bars may be combined with bent cap main top and bottom reinforcement using mechanical couplers. Splices in the "U" bars shall not be located within a distance, l_d, from the interior face of the column.

l_d = development length in tension of straight bars (in.)

B) *Vertical Stirrups in Joint Region - Use for both Cases 1 and 2*

Vertical stirrups or ties, A_s^{jv} as specified in *SDC* Equation 7.4.5.3-2, shall be placed transversely within each of regions 1, 2, and 3 of Figure *SDC* 7.4.5.3-1

$$A_s^{jv} \geq 0.2 \times A_{st}$$ (*SDC* 7.4.5.3-2)

The stirrups provided in the overlapping areas shown in Figure *SDC* 7.4.5.3-1 shall count towards meeting the requirements of both areas creating the overlap. These stirrups can be used to add shear capacity to the bent cap.

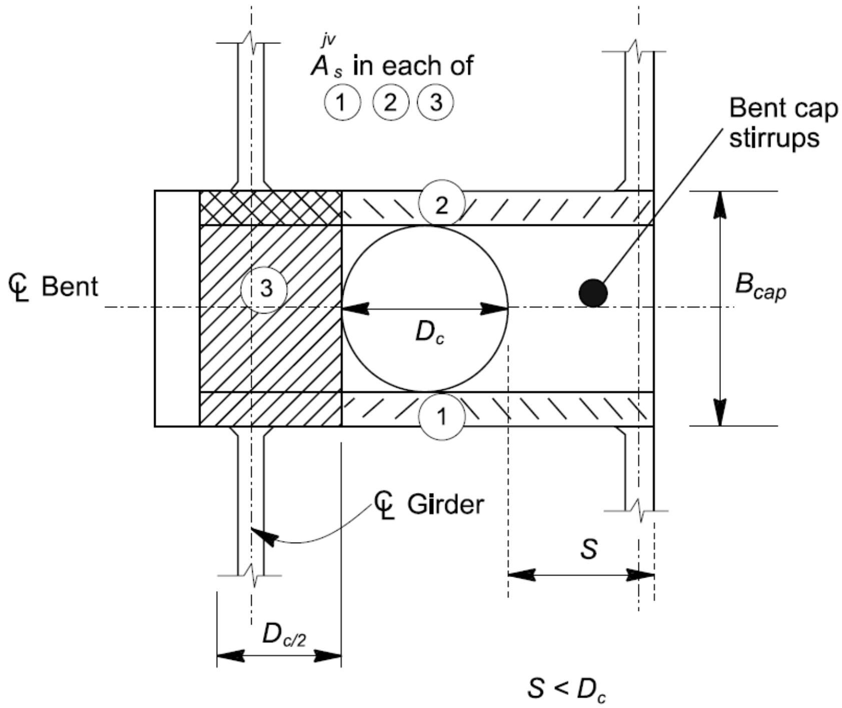


Figure *SDC* 7.4.5.3-1 Location of Knee Joint Vertical Shear Reinforcement (Plan View)

C) *Horizontal Stirrups - Use for both Cases 1 and 2*

Horizontal stirrups or ties, A_s^{jh}, as specified in *SDC* Equation 7.4.5.3-3, shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18 inches (see Figures *SDC* 7.4.5.3-2, 7.4.5.3-3, 7.4.5.3-4 and 7.4.5.3-5 for rebar placement).

$$A_s^{jh} \geq 0.1 \times A_{st} \qquad (SDC\ 7.4.5.3\text{-}3)$$

D) *Horizontal Side Reinforcement- Use for both Cases 1 and 2*

Longitudinal side face reinforcement, A_s^{sf}, shall be placed near the side faces of the bent cap with a maximum spacing of 12 inches. The required area of A_s^{sf} shall satisfy:

$$A_s^{sf} \geq \max \begin{cases} 0.1 \times A_{cap}^{top} \\ \quad or \\ 0.1 \times A_{cap}^{bot} \end{cases} \qquad (SDC\ 7.4.5.2\text{-}3)$$

where:

A_{cap}^{top} = area of bent cap top flexural steel (in.^{2})

A_{cap}^{bot} = area of bent cap bottom flexural steel (in.^{2})

The side reinforcement shall be in the form of "U" bars and shall be continuous over the exterior face of the Knee Joint. Splices in the U bars shall be located at least a distance l_d from the interior face of the column. Any side reinforcement placed to meet other requirements shall count towards meeting this requirement. Refer to *SDC* Figures 7.4.5.2-2&4 and 7.4.5.3-2,3,4&5 for placement details.

E) ***Horizontal Cap End Ties for Case 1 Only***

Horizontal ties, A_s^{jhc} shall be placed at the end of the bent cap, as shown in *SDC* Figures 7.4.5.3-2, 7.4.5.3-3, and 7.4.5.3-5. The required area of A_s^{jhc} shall satisfy:

$$A_s^{jhc} \geq 0.33 A_s^{u-bar} \quad (SDC\ 7.4.5.3\text{-}5)$$

This reinforcement shall be placed around the intersection of the bent cap horizontal side reinforcement and the continuous bent cap U-bar reinforcement and spaced at not more than 12 inches vertically and horizontally. The horizontal reinforcement shall extend through the column cage to the interior face of the column.

F) ***J-Dowels - Use for both Cases 1 and 2***

For bents skewed more than 20 degree, "J" bars (dowels) shall be provided, as shown in *SDC* Figures 7.4.5.3-3, and 7.4.5.3-4. The area of shall satisfy:

$$A_s^{j-bar} \geq 0.08 A_{st} \quad (SDC\ 7.4.5.3\text{-}6)$$

G) ***Transverse Reinforcement***

Transverse reinforcement in the joint region shall consist of hoops with a minimum reinforcement ratio as specified in *SDC* Equations 7.4.5.3-7 to 7.4.5.3-9.

For Case 1 Knee joint

$$\rho_s^{knee} = \frac{0.76 A_{st}}{D_c l_{ac,provided}} \quad (SDC\ 7.4.5.3\text{-}7)$$

For Case 2 Knee joint, Integral bent cap

$$\rho_s^{knee} = 0.4 \left[\frac{A_{st}}{l_{ac,provided}^2} \right] \quad (SDC\ 7.4.5.3\text{-}8)$$

For Case 2 Knee joint, Non-integral bent cap

$$\rho_s^{knee} = 0.6 \left[\frac{A_{st}}{l_{ac,provided}^2} \right] \quad (SDC\ 7.4.5.3\text{-}9)$$

where:

$l_{ac,provided}$ = actual length of column longitudinal reinforcement embedded into the bent cap (in.)

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})

D_c = diameter or depth of column in the direction of loading (in.)

The column transverse reinforcement extended into the bent cap may be used to satisfy this requirement. For interlocking cores, ρ_s shall be calculated on the basis of A_{st} and D_c of each core (for Case 1 Knee joints) and on area of reinforcement, A_{st} of each core (for Case 2 Knee joints). All vertical column bars shall be extended as close as possible to the top bent cap reinforcement.

20.1.2.11 Torsional Capacity

There is no history of damage to bent caps of Ordinary Standard Bridges from previous earthquakes attributable to torsional forces. Therefore, these bridges are not usually analyzed for torsional effects. However, non-standard bridge features (for example, superstructures supported on relatively long outrigger bents) may experience substantial torsional deformation and warping and should be designed to resist torsional forces.

20.1.2.12 Abutment Support Length Requirements

Sufficient seat width shall be provided to prevent the superstructure from unseating when the design seismic hazards occur. Per *SDC* Section 6.3.3, the support length normal to the centerline of the backwall as shown in Figure *SDC* 6.3.3-1, N_A shall satisfy:

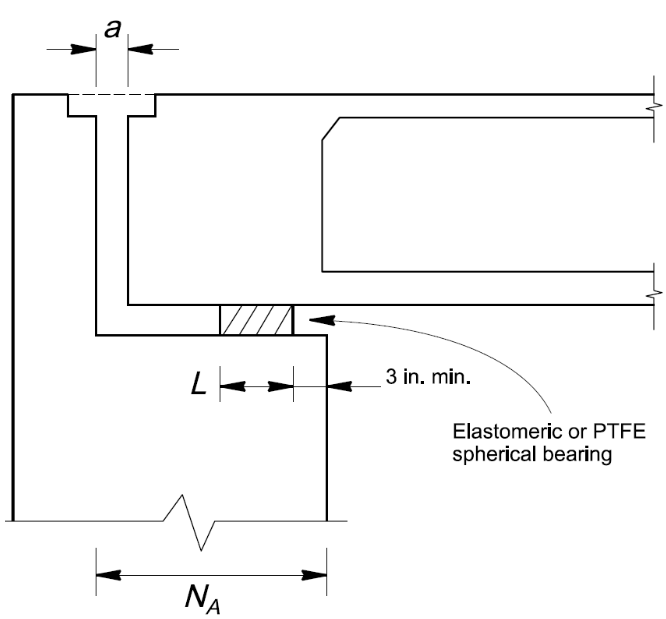


Figure *SDC* 6.3.3-1 Abutment Support Length Requirements

$$N_A \geq \begin{cases} \left(MR + \Delta_{eq} + L\right) \\ \dfrac{1}{3} D_s \\ 30\ (\text{inches}) \end{cases} \qquad (SDC\ 6.3.3\text{-}1)$$

where:

N_A = abutment support length as shown Figure *SDC* 6.3.3-1 (in.)

MR = movement range, i.e., total anticipated joint movement from the widest to the narrowest opening (in.) from the anticipated thermal movement, prestress shortening, creep, shrinkage, and the relative longitudinal displacement as shown in Figure *SDC* 6.3.3-1.

L = pad dimension along the bridge for elastomeric bearings (in.), or length of masonry plate for PTFE spherical bearings (in.)

D_s = depth of superstructure at the support (in.)

Δ_{eq} = displacement demand, Δ_D for the adjacent frame. Displacement of the abutment is assumed to be zero (in.)

20.1.2.13 Hinge Support Length Requirements

For adjacent frames with a ratio of fundamental periods of vibration for adjacent frames in the longitudinal directions greater than or equal to 0.7, as depicted in *SDC* Section 7.1.3 (Balanced Frame Geometry), *SDC* Section 7.2.3.2 requires that enough hinge seat width be provided to accommodate the anticipated thermal movement, prestress shortening, creep and shrinkage and the relative longitudinal earthquake displacement demand between the two frames - see Figure *SDC* 7.2.3.2-1. The minimum hinge seat width, measured normal to the centerline of the bent, N_H is given by:

$$N_H \geq \begin{cases} \left(MR + \Delta_{eq} + L_{mp}.\right) \\ \dfrac{1}{3} D_s \\ 30\ (\text{inches}) \end{cases} \qquad (SDC\ 7.2.3.2\text{-}1)$$

where:

$$\Delta_{eq} = \sqrt{\left(\Delta_{D1}\right)^2 + \left(\Delta_{D2}\right)^2 - 0.4\Delta_{D1}\Delta_{D2}} \qquad (SDC\ 7.2.3.3\text{-}2)$$

Δ_{eq} = relative longitudinal earthquake displacement demand base on Safety Evaluation Earthquake at an expansion joint (in.) see *SDC* Section 7.2.3.2.

$\Delta_D^{(i)}$ = the larger earthquake displacement demand for each frame calculated by either Nonlinear Time History Analysis (NTHA) or by Caltrans sponsored research (Desroches and Fenves, 1997) *SDC* Method 2 (in.)

L_{mp} = length of masonry plate for the bearing (in.)

N_H = support length normal to the centerline of bearing (in), as shown in Figure 7.2.3.2-1.

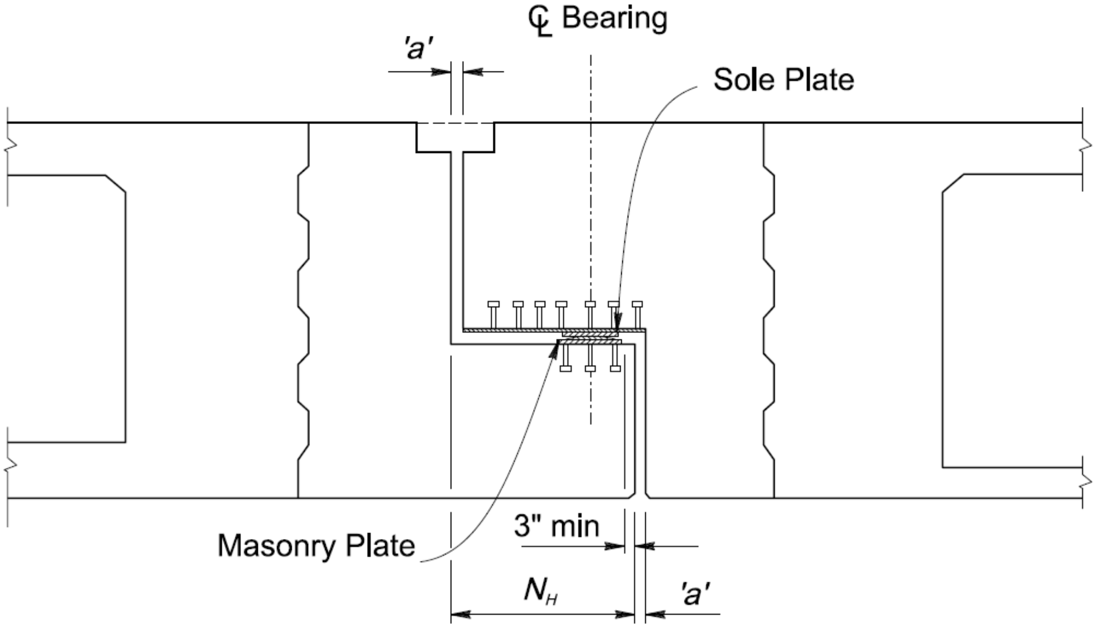


Figure *SDC* 7.2.3.3-1 Support Length Requirements at In-span Hinges

20.1.2.14 Abutment Shear Key Design

Shear keys in bridge abutments provide lateral restraints to bridge superstructures under normal service loads and moderate earthquake forces. In the event of a severe earthquake, shear keys should function as structural fuses to prevent the transmission of large seismic forces to the abutment piles.

20.1.2.14.1 General

According to *SDC* Section 6.3.4, abutment shear key force capacity, F_{sk} shall be determined as follows:

$$F_{sk} = \alpha(0.75V_{piles} + V_{ww}) \quad \text{For Abutment on piles} \qquad (SDC\ 6.3.4\text{-}1)$$

$$F_{sk} = \alpha\ P_{dl} \quad \text{For Abutment on Spread footing} \qquad (SDC\ 6.3.4\text{-}2)$$

$$0.5 \le \alpha \le 1 \qquad (SDC\ 6.3.4\text{-}3)$$

where:

V_{piles} = sum of lateral capacity geotechnical of the piles (kip)

V_{ww} = shear capacity of one wingwall (kip)

P_{dl} = superstructure dead load reaction at the abutment plus the weight of the abutment and its footing (kip)

α = adjustment factor that defines the range over which F_{sk} is allowed to vary

For abutments supported by a large number of piles, it is permitted to calculate the shear key capacity using the following equation, provided the value of F_{sk} is less than that furnished by *SDC* Equation 6.3.4-1:

$$F_{sk} = \alpha P_{dl}^{sup} \leq \alpha(0.75V_{piles} + V_{ww}) \quad (SDC\ 6.3.4\text{-}4)$$

$$F_{sk} = \alpha P_{dl}^{\text{sup}} \leq \alpha(0.5P_{dl}) \quad (SDC\ 6.3.4\text{-}5)$$

where:

P_{dl}^{sup} = superstructure dead load reaction at the abutment (kip)

For more explanation of the above terms, refer to the commentary section C 6.3.4 of *SDC*.

20.1.2.14.2 Abutment Shear Key Reinforcement

The *SDC* provides two methods for designing abutment shear key reinforcement: the Isolated key or the Monolithic (i,e., Non-isolated) key methods.

(1) *Vertical Shear Key Reinforcement*, A_{sk}

$$A_{sk}^{Iso} = \frac{F_{sk}}{1.8f_{ye}} \quad \text{Isolated shear key} \quad (SDC\ 6.3.5.1\text{-}1)$$

$$A_{sk}^{mono} = \frac{1}{1.4f_{ye}}\left(F_{sk} - 0.4 \times A_{cv}\right) \quad \text{Non-isolated shear key} \quad (SDC\ 6.3.5.2\text{-}1)$$

$$0.4A_{cv} < F_{sk} \leq \min\begin{pmatrix} 0.25f'_{ce}A_{cv} \\ 1.5A_{cv} \end{pmatrix} \quad (SDC\ 6.3.5.2\text{-}2)$$

$$A_{sk}^{mono} \geq \frac{0.05A_{cv}}{f_{ye}} \quad (SDC\ 6.3.5.2\text{-}3)$$

where:

A_{cv} = area of concrete engaged in interface shear transfer (in.^{2})

In the above equations, f_{ye} and f'_{ce} have units of ksi, A_{cv} and A_{sk} are in in2, and F_{sk} is in kip. See *SDC* Figure 6.3.5-1 and 6.3.5-2 for placement of shear key reinforcement for both methods.

***(2) Horizontal Reinforcement in the Stem Wall (Hanger Bars)**, Ash*

$$A_{sh} = (2.0)A^{Iso}_{sk(provided)}$$ Isolated shear key (*SDC* 6.3.5.1-2)

$$A_{sh} = \max\begin{cases}(2.0)A^{mono}_{sk(provided)} \\ \dfrac{F_{sk}}{f_{ye}}\end{cases}$$ Monolithic shear key (*SDC* 6.3.5.2-4)

where:

$A^{Iso}_{sk(provided)}$ = area of interface shear reinforcement provided for isolated shear key in *SDC* Equation 6.3.5.1-1(in.^{2})

$A^{mono}_{sk(provided)}$ = area of interface shear reinforcement provided for monolithic shear key in *SDC* Equation 6.3.5.2-1 (in.^{2})

For the isolated key design method, the vertical shear key reinforcement, A^{Iso}_{sk} should be positioned relative to the horizontal reinforcement in the stem wall (i,e., Hanger bars), A_{sh} to maintain a minimum length L_{min} given by (see *SDC* Figure 6.3.5-1):

$$L_{\min,hooked} = 0.6(a+b) + l_{dh}$$ (*SDC* C6.3.5.1-1)

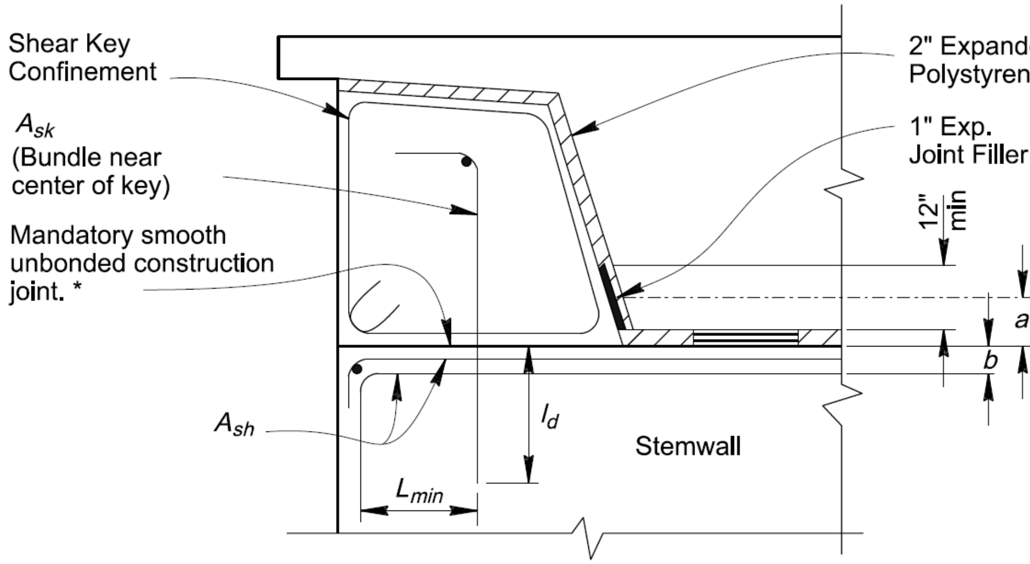
$$L_{\min,headed} = 0.6(a+b) + 3 \text{ in.}$$ (*SDC* C6.3.5.1-2)

where:

a = vertical distance from the location of the applied force on the shear key to the top surface of the stem wall, taken as one-half the vertical length of the expansion joint filler plus the pad thickness (in.) (see *SDC* Figure 6.3.5-1)

b = vertical distance from the top surface of the stem wall to the centroid of the lowest layer of shear key horizontal reinforcement (in.)

l_{dh} = development length in tension of standard hooked bars as specified in AASHTO-CA BDS-8 (in.)



* Smooth construction joint is required at the shear key interfaces with the stem wall and backwall to effectively isolate the key except for specifically designed reinforcement. These interfaces should be trowel-finished smooth before application of a bond breaker such as construction paper. Form oil shall not be used as a bond breaker for this purpose.

Figure 6.3.5.-1 Isolated Shear Key Reinforcement Details

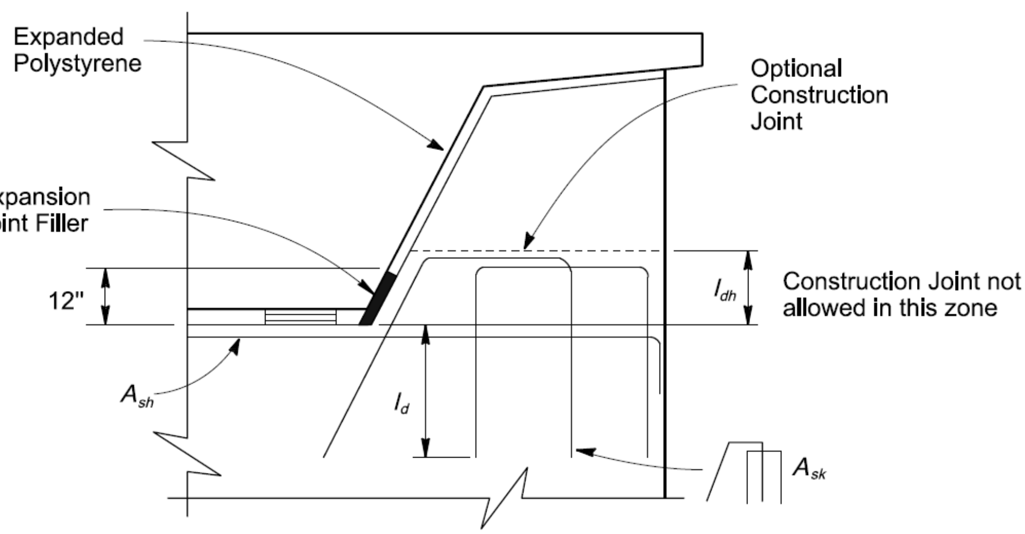


Figure 6.3.5-2 Monolithic Shear Key Reinforcement Details

NOTE:

Not all shear key bars shown

On high skews, use 2-inch expanded polystyrene with 1-inch expanded polystyrene over the 1-inch expansion joint filler to prevent binding on post-tensioned bridges.

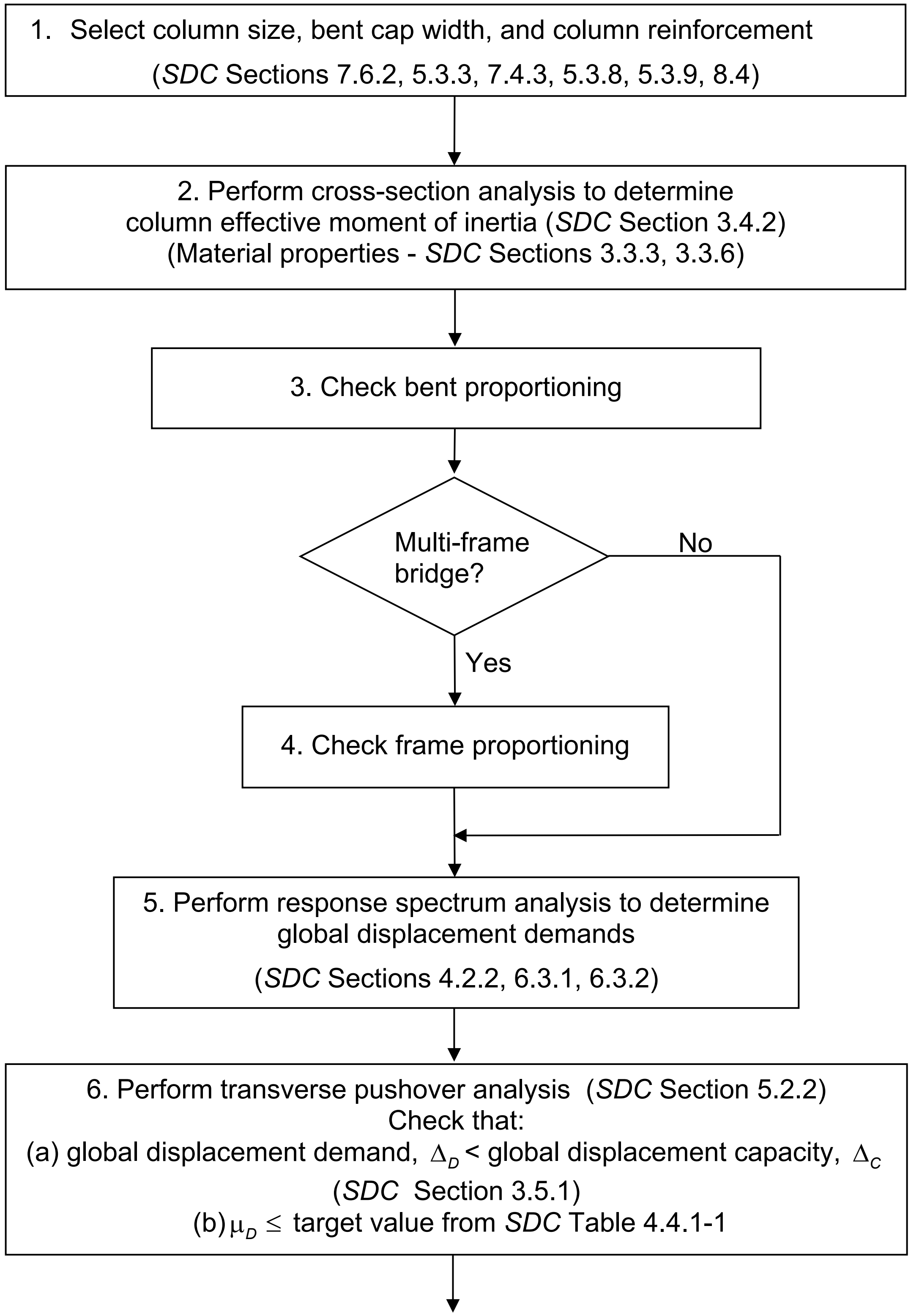
Figure SDC 6.3.5-1 and 6.3.5-2 Abutment Shear Key Reinforcement Details

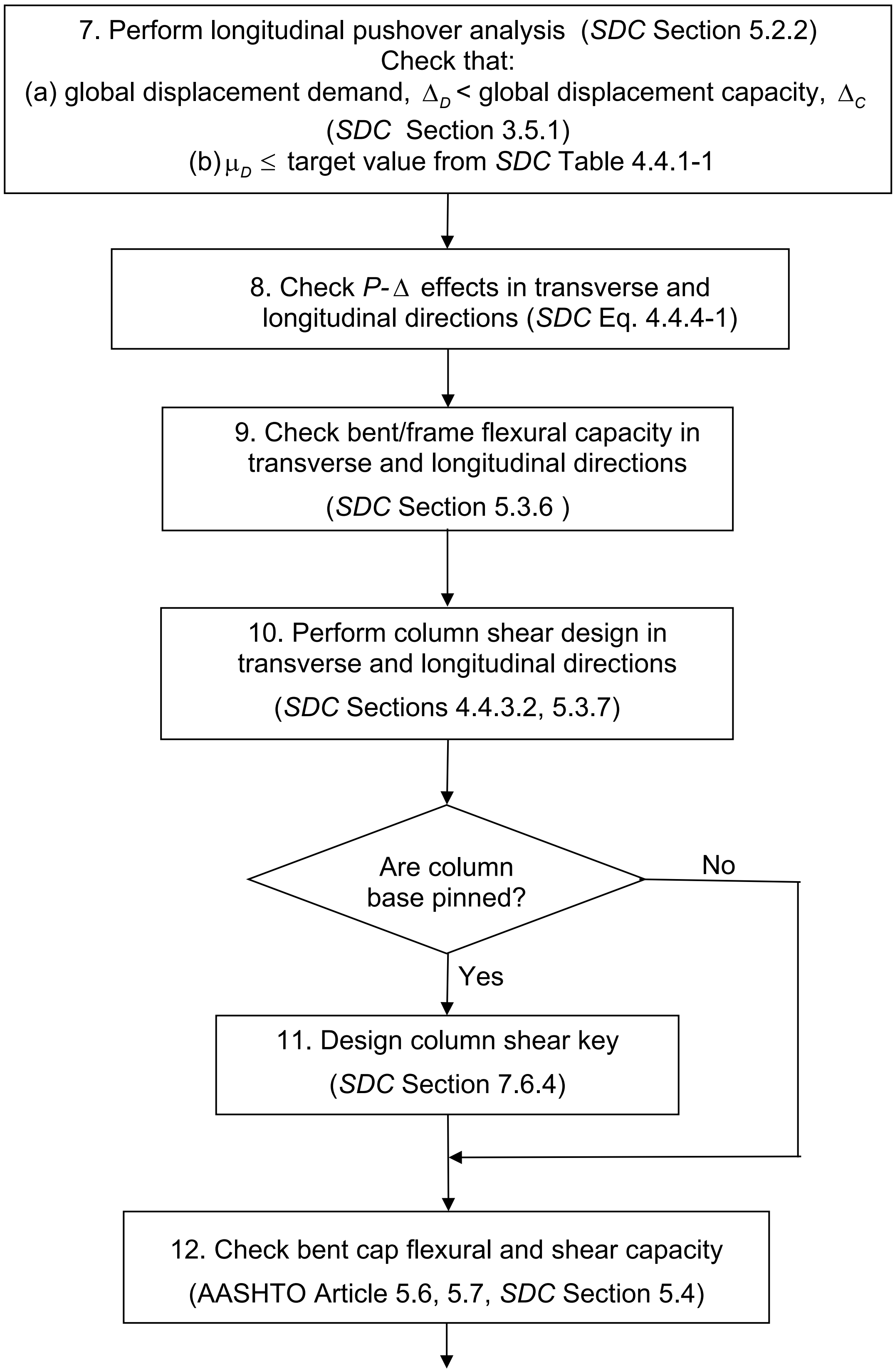
20.1.2.15 No-Splice Zone Requirements

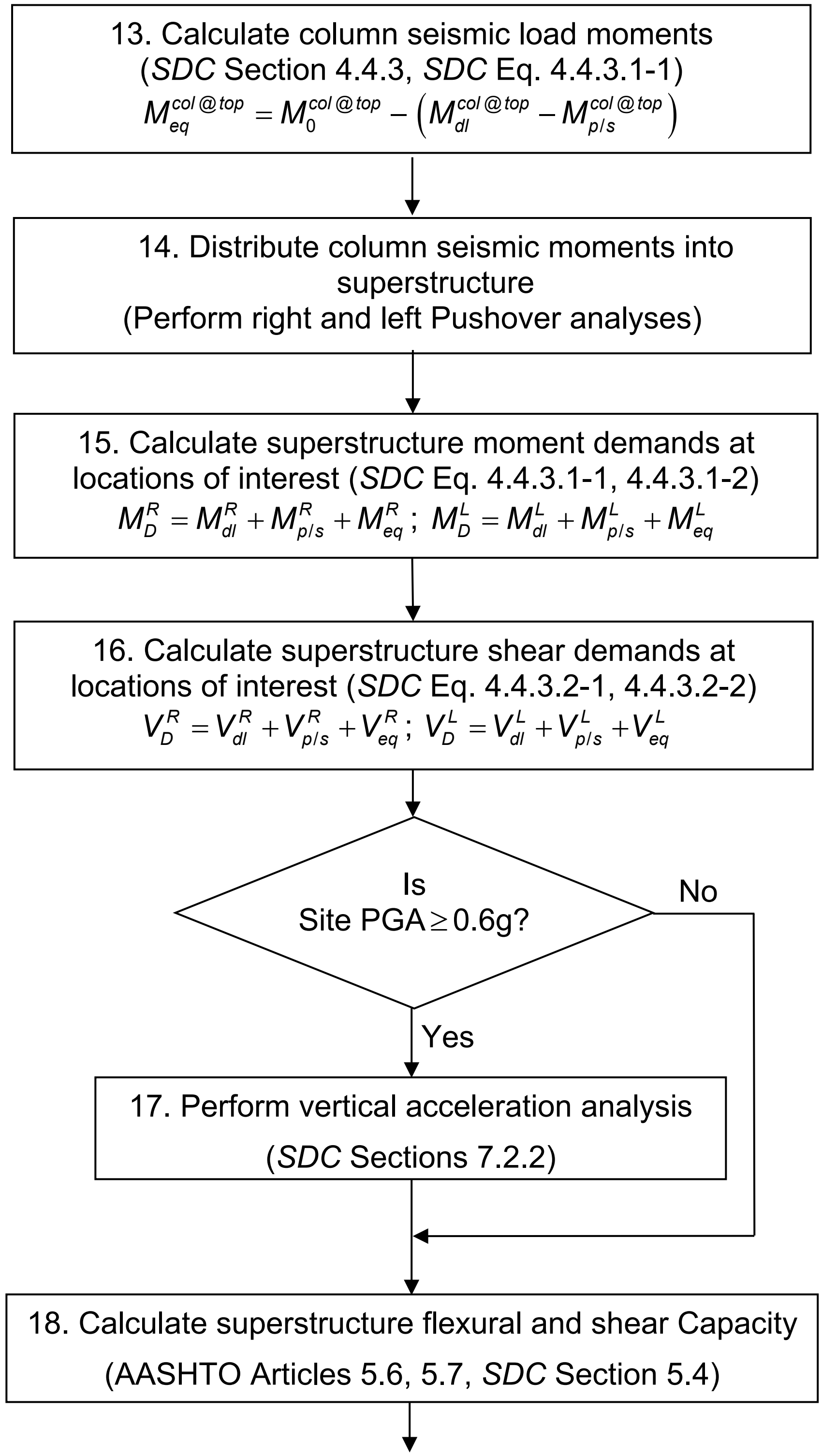
No splices in longitudinal column reinforcement are required in the plastic hinge regions (see *SDC* Section 5.3.2 of ductile members). These plastic hinge regions are called “No-Splice Zones,” and shall be detailed with enhanced lateral confinement and shown on the plans.

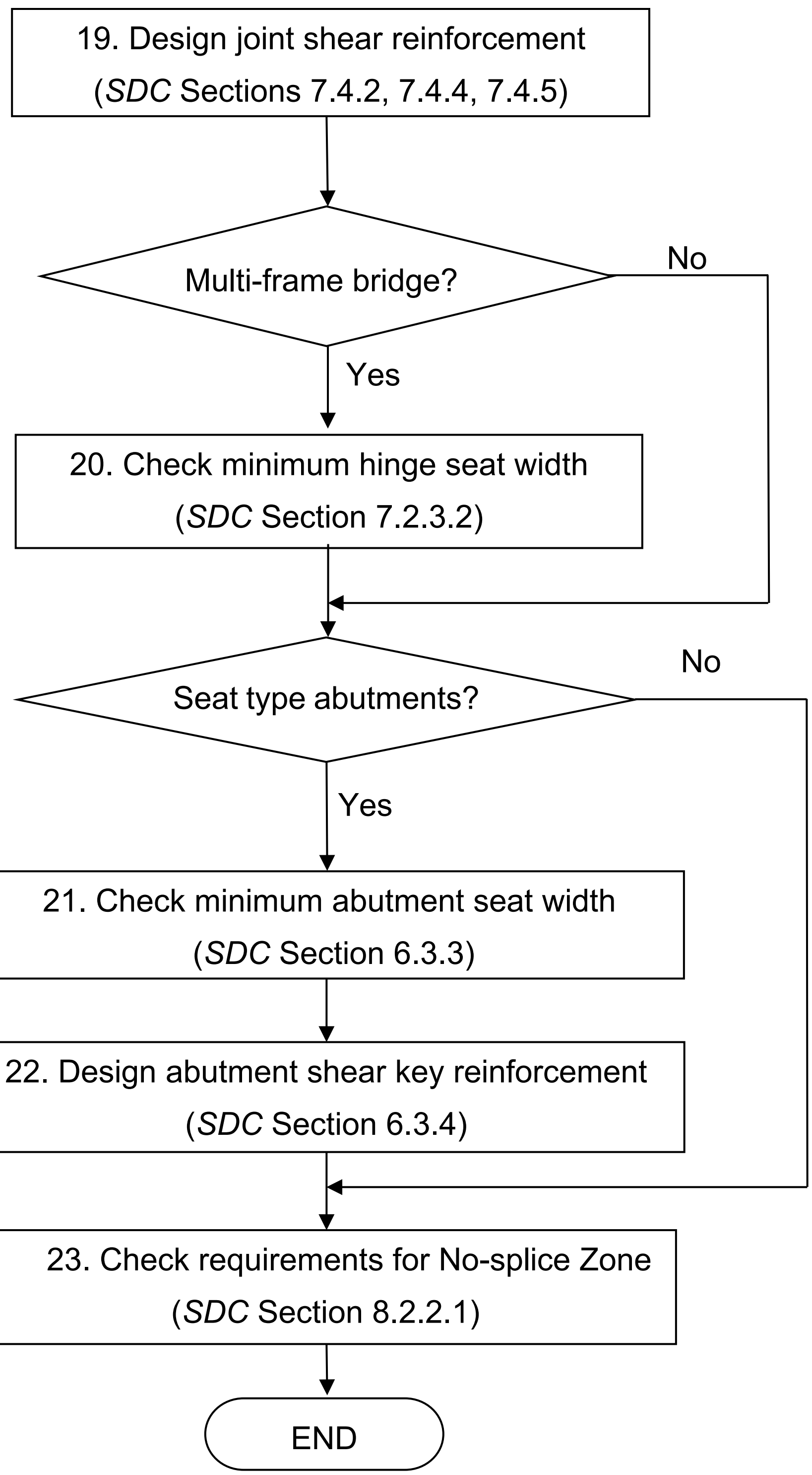
In general, for seismic critical elements, no splices in longitudinal rebars are required if the rebar cage is less than 60 ft. long. Transverse reinforcement in SCM’s shall be either ultimate butt-spliced hoops or continuous spiral. Transverse reinforcement shall be used over the entire length of the seismic critical member. Refer to *SDC* Tables 8.2.1-1 and 8.2.2.3-1 for more provisions for “No-Splice Zones” in ductile members.

20.1.2.16 Seismic Design Procedure Flowchart




7. Perform longitudinal pushover analysis (*SDC* Section 5.2.2)
Check that:
(a) global displacement demand, Δ_D < global displacement capacity, Δ_C (*SDC* Section 3.5.1)
(b) $\mu_D \leq$ target value from *SDC* Table 4.4.1-1
8. Check *P*-Δ effects in transverse and longitudinal directions (*SDC* Eq. 4.4.4-1)
9. Check bent/frame flexural capacity in transverse and longitudinal directions (*SDC* Section 5.3.6)
10. Perform column shear design in transverse and longitudinal directions (*SDC* Sections 4.4.3.2, 5.3.7)
Are column base pinned?
No
Yes
11. Design column shear key (*SDC* Section 7.6.4)
12. Check bent cap flexural and shear capacity (AASHTO Article 5.6, 5.7, *SDC* Section 5.4)


13. Calculate column seismic load moments
(*SDC* Section 4.4.3, *SDC* Eq. 4.4.3.1-1)
$M_{eq}^{col@top} = M_{0}^{col@top} - \left(M_{dl}^{col@top} - M_{p/s}^{col@top} \right)$
14. Distribute column seismic moments into superstructure
(Perform right and left Pushover analyses)
15. Calculate superstructure moment demands at locations of interest (*SDC* Eq. 4.4.3.1-1, 4.4.3.1-2)
$M_{D}^{R} = M_{dl}^{R} + M_{p/s}^{R} + M_{eq}^{R}$; $M_{D}^{L} = M_{dl}^{L} + M_{p/s}^{L} + M_{eq}^{L}$
16. Calculate superstructure shear demands at locations of interest (*SDC* Eq. 4.4.3.2-1, 4.4.3.2-2)
$V_{D}^{R} = V_{dl}^{R} + V_{p/s}^{R} + V_{eq}^{R}$; $V_{D}^{L} = V_{dl}^{L} + V_{p/s}^{L} + V_{eq}^{L}$
Is Site PGA $\geq$ 0.6g?
No
Yes
17. Perform vertical acceleration analysis
(*SDC* Sections 7.2.2)
18. Calculate superstructure flexural and shear Capacity
(AASHTO Articles 5.6, 5.7, *SDC* Section 5.4)


19. Design joint shear reinforcement
(SDC Sections 7.4.2, 7.4.4, 7.4.5)
Multi-frame bridge?
No
Yes
20. Check minimum hinge seat width
(SDC Section 7.2.3.2)
Seat type abutments?
No
Yes
21. Check minimum abutment seat width
(SDC Section 6.3.3)
22. Design abutment shear key reinforcement
(SDC Section 6.3.4)
23. Check requirements for No-splice Zone
(SDC Section 8.2.2.1)
END

20.1.3 DESIGN EXAMPLE - THREE-SPAN CONTINUOUS CAST-IN-PLACE CONCRETE BOX GIRDER BRIDGE

20.1.3.1 Bridge Data

The three-span prestress reinforced concrete box girder bridge shown in Figure 20.1.3-1 will be used to illustrate the procedures of seismic bridge design. The span lengths are 126 ft, 168 ft, and 118 ft. The column height varies from 44 ft at Bent 2 to 47 ft at Bent 3. Both bents have a skew angle of 20 degrees. The columns are pinned at the bottom. The bridge ends are supported on seat-type abutments.

Material Properties:

Concrete: $f'_c = 4$ ksi

Reinforcing steel: A706 Grade 60, f_y = 60 ksi, E_s = 29,000 ksi,
f_{ye} = 68 ksi, f_{ue} = 95 ksi,

Bridge Site Conditions:

This example bridge crosses a roadway and railroad tracks. All column footings are supported on piles. The ground motion at the bridge site is assumed to be:

Soil Profile: $V_{s30} = 700$ ft/sec

Magnitude: 8.0 ± 0.25

Peak Ground Acceleration: 0.58*g*

Figure 20.1.3-2 shows the assumed design spectrum. For more information on Design Spectrum development, refer to *SDC* Appendix B.

20.1.3.2 Design Requirements

Perform seismic analysis and design in accordance with Caltrans *SDC* Version 2.0 (Caltrans 2019).

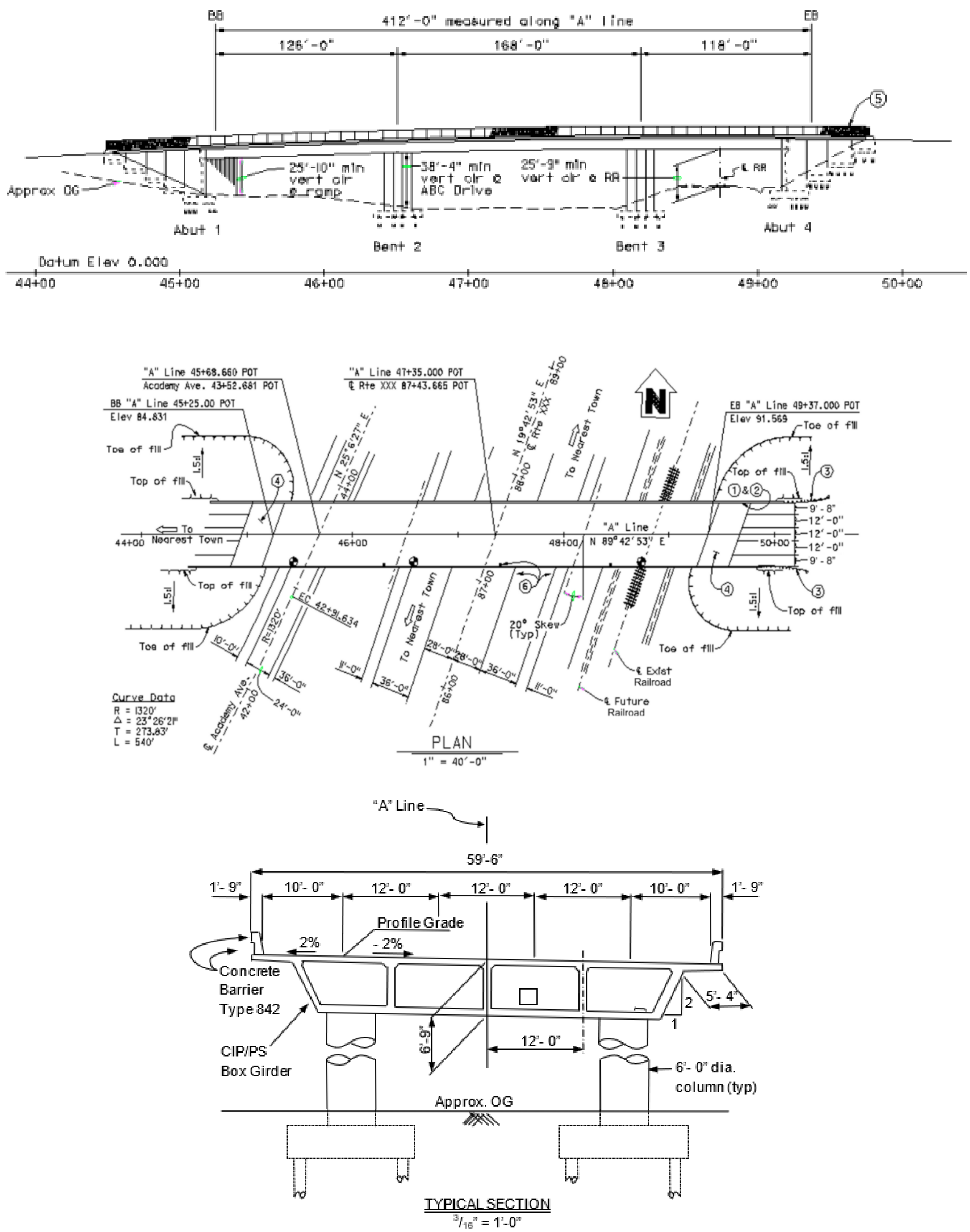


Figure 20.1.3-1. General Plan (Bridge Design Academy Prototype Bridge)

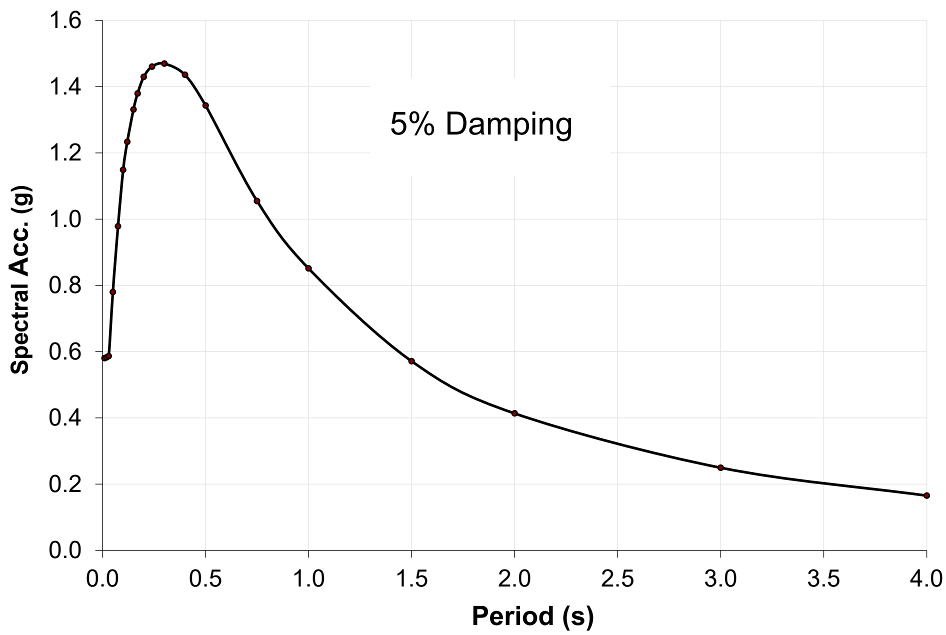


Figure 20.1.3-2. Design Acceleration Response Spectrum

20.1.3.3 Step 1 – Select Column Size, Bent Cap Width, and Column Reinforcement

20.1.3.3.1 Select Column Size

Given the superstructure depth, D_s = 6.75 ft, we select a circular column diameter, D_c = 6.00 ft.

So that:

$$0.7 < \left[\frac{D_c}{D_s} = 0.89\right] < 1.00 \qquad \text{OK} \qquad (SDC\ 7.6.2\text{-}1)$$

$$A_g = \frac{\pi}{4}(6 \times 12)^2 = 4{,}071.5.\ \text{in.}^2$$

20.1.3.3.2 Select Bent Cap Width

$$B_{cap} = D_c + 2 = 6 + 2 = 8 \text{ ft} \qquad (SDC\ 7.4.3\text{-}1)$$

20.1.3.3.3 Calculate Dead Load Axial Forces

As a first step toward calculating effective section properties of the column, the dead load axial force at the column top (the location of the potential plastic hinge) is calculated. These column axial forces are obtained from the *CTBridge* outputs. It should also be noted that these loads do not include the weight of the integral bent cap. The *CTBridge* model has the regular superstructure cross-section with flared the bottom slab instead of the solid cap section. In this example, the weight of the voided portion at the bent cap was added to the *CTBridge* results conservatively.

As read from the *CTBridge* output results, the column dead load axial forces without the bent cap weight are shown in Table 20.1.3-1:

Table 20.1.3-1 Column Dead Load Axial Forces without Bent Cap Weight

	Column 1	Column 2
Bent 2 *(P_{dl})* (kip)	1,529	1,529
Bent 3 (P_{dl}) (kip)	1,476	1,476

Bent Cap Weight:

$$= \text{Area of Voids } \times\left(\frac{8}{\cos 20^o}\right)(0.15) = 313.5\left(\frac{8}{\cos 20^o}\right)(0.15) = 400 \text{ kips}$$

Adding this bent cap weight and the total axial force (at the potential plastic hinge location) in each column becomes:

Table 20.1.3-2 Column Dead Load Axial Forces at Potential Plastic Hinge Region

	Column 1	Column 2
Bent 2 (P_{dl}) (kip)	1,729	1,729
Bent 3 (P_{dl}) (kip)	1,676	1,676

20.1.3.3.4 Check Column Axial Load Limits

Use Column 1 or 2 of Bent 2 (the worst case):

$$\rho_{dl} = \frac{P_{dl}}{f_c' A_g} = \frac{1{,}729}{(4)(4{,}071.5)} = 0.106 < 0.15 \qquad \text{OK} \qquad (SDC\ 5.3.3\text{-}1)$$

Note: The axial load ratio due to the dead load and the overturning will be checked in Step 6.

20.1.3.3.5 Select Column Longitudinal Reinforcement

Try #14 bars for the longitudinal reinforcement, A_b = 2.25 in.^{2}, d_b = 1.693 in.

Try 26 #14 longitudinal bars, A_{st} = 26(2.25) = 58.5 in.^{2}

$$A_{(st,max)} = 0.04A_g = 0.04(4,071.5) = 162.86 \text{ in.}^2 \qquad (SDC\ 5.3.9.1\text{-}1)$$

$$A_{(st,\min)} = 0.01A_g = 0.01(4,071.5) = 40.72 \text{ in.}^2 \qquad (SDC\ 5.3.9.1\text{-}2)$$

$$A_{(st,min)} = 40.72 \text{ in.}^2 < A_{st} = 58.5 \text{ in.}^2 < A_{(st,max)} = 162.86 \text{ in.}^2 \quad \text{OK}$$

Assuming a concrete cover of 2 in. as specified in AASHTO-CA *BDS*-8 Table 5.10.1-1 for the minimum concrete cover (Caltrans 2019). Therefore, the diameter of the longitudinal reinforcement loop (through the centerline of longitudinal bars) is:

$$d_M = 72 - 2(2) - 2(0.87) - 2\left(\frac{1.88}{2}\right) = 64.37 \text{ in.}$$

The spacing of longitudinal bars is:

$$\frac{\pi d_M}{26} = \frac{\pi(64.37)}{26} = 7.7 \text{ in.} < 12 \text{ in.} \qquad \text{OK} \qquad (SDC\ 8.4.2)$$

$$7.7 \text{ in.} > \text{Largest of} \begin{Bmatrix} 1.5d_d = 1.5(1.693) = 2.54 \text{ in.} \\ 1.5(\text{Max. size of coarse aggregate}) \\ = 1.5(1.0) = 1.5 \text{ in.} \\ 1.5 \text{ in.} \end{Bmatrix} = 2.54 \text{ in.} \quad \text{OK}$$

(AASHTO 5.10.3.1.1)

Note: If the calculated spacing turns out to be smaller than that the minimum spacing allowed, then a smaller bar size should be used.

20.1.3.3.6 Select Column Transverse Reinforcement

Try #7 hoops @ 6 in. for the plastic hinge region, and #7 hoops @ 10 in. for outside the plastic hinge region. Check the maximum spacing of hoops - Inside the plastic hinge region:

$$6 \text{ in.} < \text{Smaller of} \begin{Bmatrix} 6d_b = (6)(1.693) = 10.2 \text{ in.} \\ 8 \text{ in.} \end{Bmatrix} = 8 \text{ in.} \quad \text{OK} \qquad (SDC\ 8.4.1.1)$$

Check the maximum spacing of hoops - Outside the plastic hinge region:

$$10\ \text{in.} = \text{Smallest of} \begin{Bmatrix} \dfrac{D_c}{2} = \dfrac{72}{2} = 36\ \text{in.} \\ 9d_b = (9)(1.693) = 15.2\ \text{in.} \\ 10\ \text{in.} \end{Bmatrix} = 10\ \text{in.} \quad \text{OK} \qquad (\textit{SDC}\ 8.4.1.2)$$

The transverse reinforcement volumetric ratio, ρ_s for circular column is given by *SDC* C5.3.8.2:

$$\rho_s = \frac{4A_b}{D's} = \frac{4(0.6)}{(72 - 2\times 2 - 2\times 0.875/2)(6)} = 0.006$$

$$\rho_{dl} \cong 10\% \rightarrow \rho_s = \rho_{s,\,\min} = 0.006 \qquad \text{Say OK} \qquad (\text{See } \textit{SDC} \text{ Table 5.3.8.2-1})$$

20.1.3.4 Step 2 – Perform Cross-Section Analysis

The expected compressive strength of concrete is:

$$f'_{ce} = 1.3f'_c = (1.3)(4{,}000) = 5{,}200\ \text{psi} > 5{,}000\,\text{psi} \qquad \text{OK} \qquad (\textit{SDC}\ 3.3.6\text{-}4)$$

Other concrete properties used are listed in *SDC* Section 3.3.6. The following values are used as inputs to the *CSiBridge* Section-Designer:

Column Diameter = 72.0 in. Concrete cover = 2 in.

Main Reinforcement: #14 bars, total 26.

Lateral Reinforcement: #7 hoops @ 6 in. The program calculates the modulus of elasticity of concrete internally.

For Grade-60 A706 bar reinforcing steel expected material property is used (see *SDC* Table 3.3.3-1) and reduced ultimate tensile strain:

$$\varepsilon_{su}^{R} = \begin{cases} 0.09 & \text{Transverse steel} \\ 0.06 & \text{Logitudinal steel} \end{cases}$$

Select Input for the *CSiBridge* Section-Designer and the Moment-Curvature (M-ϕ) diagrams for Bents 2 & 3 Column are shown in Appendix 20.1.3-2.

Bent 2 Columns Axial Force, P_c = 1,729 kips.

Bent 3 Column Axial Force, P_c = 1,676 kips.

From M-ϕ analysis results, the effective moment of inertia, I_{eff} = 22.49 ft^4 for Bent 2 columns and for Bent 3 columns, I_{eff} = 22.35 ft^4 (See Appendices 20.1.3-2)

20.1.3.5 Step 3 – Check Bent Proportioning

20.1.3.5.1 Calculate Bent 2 Stiffness

$$E_c = 33\left(w_c\right)^{1.5}\sqrt{f'_{ce}}\,(\text{psi}) = 33\left(150\right)^{1.5}\sqrt{5{,}200} = 4{,}371{,}722 \text{ psi} = 4{,}372 \text{ ksi} \qquad (SDC\ 3.3.6\text{-}1)$$

$$K_2^e = (2)\frac{3EI_{eff}}{L^3} = (2)\frac{(3)(4{,}372)(22.49)(12)^4}{\left(44\times12\right)^3} = 83.11 \text{ kip/in.}$$

20.1.3.5.2 Calculate Bent 3 Stiffness

$$K_3^e = (2)\frac{3EI_{eff}}{L^3} = (2)\frac{(3)(4{,}372)(22.35)(12)^4}{\left(47\times12\right)^3} = 67.76 \text{ kip/in.}$$

20.1.3.5.3 Check Stiffness-to-Mass Ratios

Note: ignore half the Column Self weight for simplicity.

$$m_2 = \text{Total tributary mass at Bent 2} = (2)\frac{(2)(1{,}729)}{(32.2)(12)} = 8.95 \text{ kip-s}^2\text{/in.}$$

$$m_3 = \text{Total tributary mass at Bent 3} = (2)\frac{(2)(1{,}676)}{(32.2)(12)} = 8.67 \text{ kip-s}^2\text{/in.}$$

$$0.75 < \frac{\left(\dfrac{k_i^e}{m_i}\right)}{\left(\dfrac{k_j^2}{m_j}\right)} = \frac{\left(\dfrac{67.76}{8.67}\right)}{\left(\dfrac{83.11}{8.95}\right)} = 0.84 < 1.33 \qquad \text{OK} \qquad (SDC\ 7.1.2\text{-}2)$$

It is seen that the effective stiffness-to-mass ratio between adjacent bents within a frame is satisfied.

20.1.3.6 Step 4 – Check Frame Proportioning

Since this is a single-frame bridge, this step does not apply.

20.1.3.7 Step 5 – Perform Response Spectrum Analysis

The *CSiBridge* 3D model of response spectra analysis is used to determine the global displacement demands by applying one-half of the effective abutment stiffness (K_{eff}) to each abutment location determined based on *SDC* 2.0 Section 6.3.1. The model is shown in Figure 20.1.3-3.

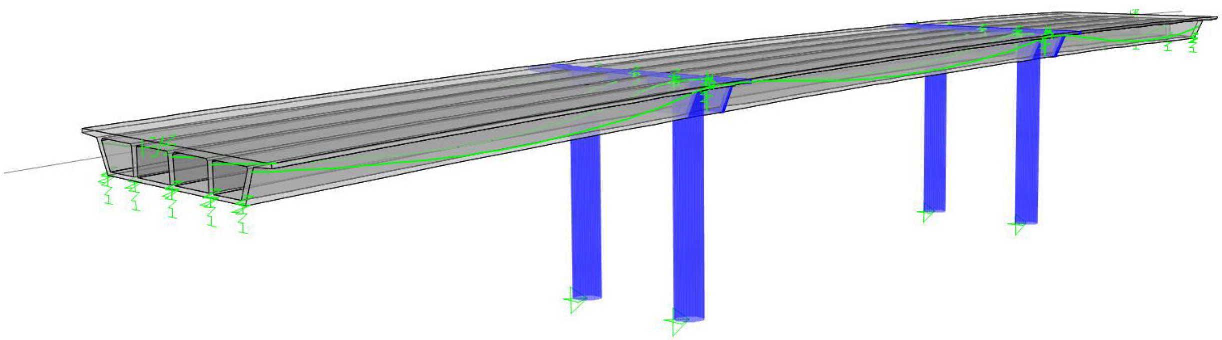


Figure 20.1.3-3. 3-D *CSiBridge* Model for Bridge Example

20.1.3.7.1 Calculate Abutment Longitudinal Stiffness

This bridge is supported on seat type abutments (see Figure 20.1.3-4 for effective abutment dimensions). The effective area is calculated as:

$$w_{bw} = \frac{49.83 + 43.08}{2} = 46.46 \text{ ft}$$

$$A_e = h_{bw} w_{bw} = (6.75)(46.46) = 313.6 \text{ ft}^2$$

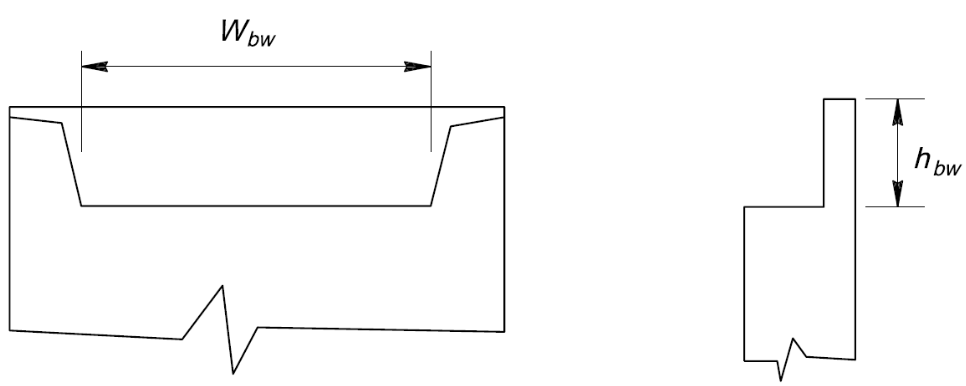


Figure 20.1.3-4. Effective Area of Seat Abutment

$$F_{abut} = w_{abut}\left(\frac{5.5h_{abut}^{2.5}}{1 + 2.37h_{abut}}\right)R_{sk} \qquad (SDC\ 6.3.1.2\text{-}4)$$

For θ = 20 degrees

$$R_{sk} = e^{-\frac{\theta}{45}} = 0.6412 \qquad (SDC\ 6.3.1.2\text{-}6)$$

For the seat type abutment, $w_{abut} = w_{bw}$ = 46.46 ft, $h_{abut} = h_{bw}$ = 6.75 ft.

$$F_{abut} = (46.46)\left(\frac{5.5(6.75)^{2.5}}{1+2.37(6.75)}\right)(0.6412) = 1{,}142 \text{ kips}$$

$$K_{abut} = w_{abut}(5.5h_{abut} + 20)R_{sk}$$
$$= (46.46)\left[5.5(6.75)+20\right](0.6412) = 1{,}702 \text{ kip/in.} \qquad (SDC\ 6.3.1.2\text{-}5)$$

$$\Delta_{abut} = \frac{F_{abut}}{K_{abut}} = \frac{1{,}142}{1{,}702} = 0.67 \text{ in.} \qquad \text{(see Figure 20.1.3-5)}$$

$$\Delta_{eff} = \Delta_{abut} + \Delta_{gap} = 0.67 + 2.60 = 3.27 \text{ in.} = 0.272 \text{ ft} \qquad (SDC\ 6.3.1.2\text{-}2)$$

See Appendix 20.1.3-9 for calculations for Δ_{gap}, the combined effect of the thermal movement and the anticipated shortening. The average contributory length is used in the calculation for Δ_{gap}.

$$K_{eff} = \frac{1{,}142}{3.27} = 349 \text{ kip/in.}$$

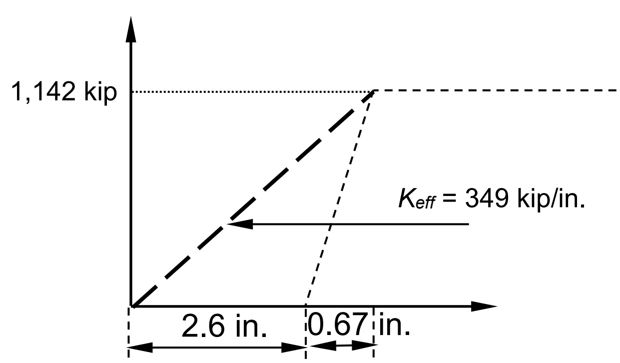


Figure 20.1.3-5. Nonlinear Abutment Model

This value is used as the initial abutment stiffness for the response spectrum analysis. The longitudinal displacement demand Δ_D = 8.4 in., is determined from the 3D *CSiBridge* response spectra analysis using the initial abutment stiffness.

$$R_A = \frac{\Delta_D}{\Delta_{eff}} = \frac{8.4}{3.27} = 2.569 \qquad (SDC\ 6.3.1.3\text{-}1)$$

Since $2 < R_A < 4$ (*SDC* 6.3.1.3-3)

$$\begin{aligned} K_{eff(1)}^{adj} &= K_{eff}\left[1-0.45\left(R_A-2\right)\right] = (349)\left[1-0.45(2.569-2)\right] \\ &= 260 \text{ kip/in.} = 3{,}120 \text{ kip/ft} \end{aligned} \qquad (SDC\ 6.3.1.3\text{-}3)$$

To obtain accurate lateral displacement demands the response spectra analysis is run using K_{eff}^{adj} and longitudinal displacement demand of Δ_D = 9.31 in. is obtained. Therefore, checking the displacement coefficient and revise K_{eff}^{adj} once again would get more accurate displacement demands.

$$R_A = \frac{\Delta_D}{\Delta_{eff}} = \frac{9.31}{3.27} = 2.847$$

$$K_{eff(2)}^{adj} = K_{eff(1)}^{adj}\left[1-0.45\left(R_A-2\right)\right] = (260)\left[1-0.45(2.847-2)\right] = 161 \text{ kip/in.} = 1{,}932 \text{ kip/ft}$$

20.1.3.7.2 Perform Response Spectra Analysis

From the response spectra analysis, using the $K_{eff(2)}^{adj}$, the longitudinal displacement demand is Δ_D = 10.61 in., as shown in Table 20.1.3-3 (see Appendix 20.1.3-8).

Table 20.1.3-3 Global Displacement Demand

Location	Transverse Displacement Demand (in.)	Longitudinal Displacement Demand (in.)
Bent 2	12.81	10.61
Bent 3	11.54	10.61

20.1.3.8 Step 6 – Perform Transverse Pushover Analysis

20.1.3.8.1 Establish Model

Figure 20.1.3-6 shows a schematic model of the frame in the transverse direction. Soil springs aren't included in the model as the soil above the footing is two feet or less.

The following values obtained from the *CSiBridge* Section-Designer output (see Appendix 20.1.3-2) are used as inputs in the *CSiBridge* program for pushover analysis.

Table 20.1.3-4 Bent 2 - Column Section Properties Under Axial Force

P_c (kip)	M_p (kip-ft)	I_{eff} (ft^4)	ϕ_Y (rad/in.)	ϕ_p (rad/in.)
1,729	13,232	22.49	0.0000779	0.0006104

ϕ_Y - Idealized yield curvature capacity (*CSiBridge* ver. 23.3.0)

ϕ_p - Idealized plastic curvature capacity is not reported on Section-Designer (*CSiBridge* ver. 23.3.0), instead the ultimate curvature capacity is reported, and the user has to calculate ϕ_p (see *SDC* C5.2.2-5).

I_{eff} - is labelled as I_{Crack} on the *CSiBridge* ver.23.3.0 Section-Designer (refer *SDC* 3.4.2).

The equivalent plastic hinge length, L_p, of the columns at Bent 2 and 3 shall be determined in accordance with *SDC* 5.3.4 Case A for this design example:

Bent 2:

L = 47.375 ft

$$L_p = 0.08L + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} \qquad \text{(SDC 5.3.4-1)}$$

$$L_p = 0.08(568.5) + 0.15(68)(1.693) = 62.75 \text{ in.} > 0.3(68)(1.693) = 34.54 \text{ in.}$$

Use $L_p = 62.75$ in.

Bent 3:

L = 50.375 ft

Similarly, $L_p = 65.63$ in.

As the frame is pushed toward the right, the resulting overturning moment causes redistribution of the axial forces in the columns. This overturning causes an additional axial force on the front-side column, which will experience the additional compression. The column on the back side experiences the same value in the tension, reducing the net axial load.

At the instant the first plastic hinge forms (in this case at the top of the front side column), the following Bent 2 displacement and corresponding lateral force values are obtained from the *CSiBridge* outputs (see Appendix 20.1.3-3):

$\Delta_Y^{col} = 8.84$ in. and corresponding lateral force is 653 kips

For this mechanism, the axial forces in the two columns as read from the *CSiBridge* analysis outputs are 820 kips and 2,638 kips, respectively (see Appendix 20.1.3-3).

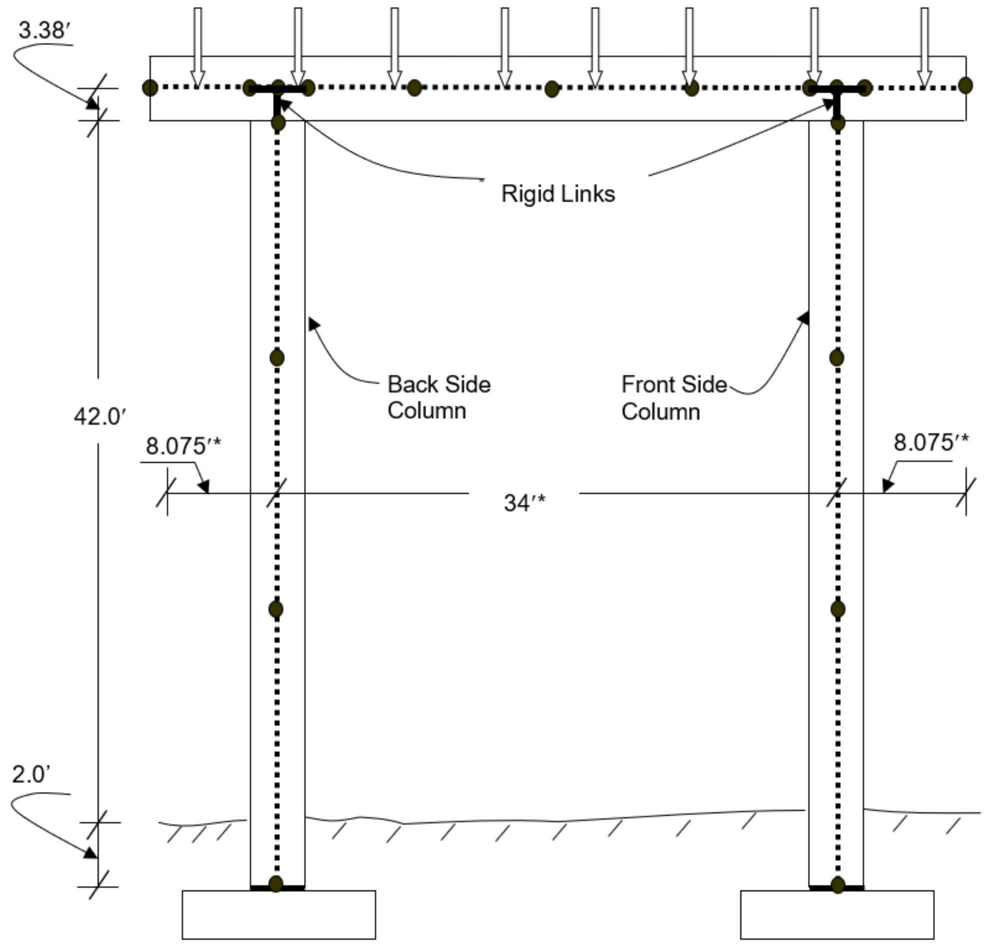


Figure 20.1.3-6. Transverse Pushover Analysis Model

These values can be quickly checked using simple hand calculations as described below:

$$M_{overturning} = (653)(47{,}375) = 30{,}936 \text{ kip-ft}$$

The axial compression force corresponding to $M_{overturning}$

$$\Delta P = \pm(653)(47{,}375) = 910 \text{ kip}$$

The axial force in the font side column is increased to 1,729 + 910 = 2,639 kips. The axial compression force in the back side column drops to 1,729 - 910 = 819 kips. These values compare well with the *CSiBridge*.

Column section properties corresponding to the updated axial forces (i.e. with overturning) are obtained from new *CSiBridge* Section-Designer runs and summarized in the Table 20.1.3-5 below (see Appendices 20.1.3-4 and 20.1.3-5 for select portions of the outputs for the front - and back-side columns, respectively).

Table 20.1.3-5 Column Section Properties of Updated Axial Forces

Column Type	P_c (kip)	M_p (kip-ft)	I_{eff} (ft^4)	ϕ_Y (rad/in.)	ϕ_p (rad/in.)
Back Side	820	11,911	20.65	0.0000765	0.000707
Front Side	2,638	14,418	24.95	0.0000765	0.000533

Note: that higher compression produces a higher value of M_p but a reduction in ϕ_p. This trend should be examined and is a reminder that M_p is not the only indicator of column performance.

With updated values of I_{eff}, the frame is pushed laterally, the front side column yields at the top at a displacement $\Delta_{Y(1)}^{col} = 8.09$ inches.

As the frame is pushed further, the already yielded front side column is able to undergo additional displacement because of its plastic hinge rotational capacity. As the bent is pushed further, the top of the back side column yields at a displacement, $\Delta_{Y(2)}^{col} = 10.13$ in. (see Appendix 20.1.3-6). At this point the effective bent stiffness approaches zero and will not attract any additional force if pushed further. The bent, however, will be able to undergo additional displacement until the rotational capacity of one of the hinges is reached. This rotational limit of any one plastic hinge describes a collapse mechanism. The force-displacement relationship is shown in Appendix 20.1.3-7.

The idealized yield Δ_Y^{col}, which was calculated previously based upon the assumption that the bent cap beam is rigid, is updated to 8.09 inches. The corresponding lateral force 629 kips.

20.1.3.8.2 Check Axial Load Limits due to Overturning

Per *SDC* 5.3.3, the axial load ratio due to the load and the overturning is checked as follows:

$$\rho_c = \frac{P_c}{f_c' A_g} = \frac{2,638}{(4)(4,071.5)} = 0.162 < 0.22 \text{ OK} \qquad (SDC\ 5.3.3\text{-}2)$$

20.1.3.8.3 Calculate Displacement Ductility Capacity

CSiBridge 2D static pushover analysis is performed to determine the transverse displacement capacities as shown in Table 20.1.3-6 (see Appendix 20.1.3-6 for Bent 2 *CSiBridge* output or Table 20.1.3-6 for both Bents).

Table 20.1.3-6 Transverse Displacement Capacity

Hinge Location	Yield Displacement (in.)	Plastic Displacement (in.)	Total Displacement Capacity (in.)
Bent 2	8.09	10.21	18.30
Bent 3	10.35	10.41	19.43

The plastic deformation is determined by subtracting the yield displacement form the total displacement capacity of the *CSiBridge* pushover results.

Check Transverse Displacement Ductility Demand:

Bent 2

$$\mu_D = \frac{\Delta_D}{\Delta_y} = \frac{12.81}{8.09} = 1.58 < 5 \qquad \text{OK} \qquad (SDC\ 4.4.1\text{-}1)$$

Also, the global displacement criteria are satisfied:

$$\Delta_D = 12.81 \text{ in.} < \Delta_C = 18.30 \text{ in.} \qquad \text{OK} \qquad (SDC\ 3.5.1)$$

Bent 3

$$\mu_D = \frac{\Delta_D}{\Delta_y} = \frac{11.54}{10.35} = 1.12 < 5 \qquad \text{OK}$$

Also, $\Delta_D = 11.54 \text{ in.} < \Delta_C = 19.43 \text{ in.}$ OK

20.1.3.9 Step 7 – Perform Longitudinal Pushover Analysis

From the longitudinal *CSiBridge* pushover analysis results (see Appendix 20.1.3-10 for displacement of the right push), the displacement capacities of Bent 2 and Bent 3 are shown in Table 20.1.3-7.

Table 20.1.3-7 Longitudinal Displacement Capacities

Location	Yield Displacement (Right Push) (in.)	Displacement Capacity (Right Push) (in.)	Yield Displacement (Left Push) (in.)	Displacement Capacity (Left Push) (in.)
Bent 2	8.74	26.63	8.40	26.29
Bent 3	9.40	29.51	9.48	29.88

The displacement capacities for both Bents 2 and 3 are approximately close as calculated for the right and left longitudinal bending for the case of gravity loading. This is because the longitudinal case has very little overturning to change the column axial loads.

This displacement demand is the same at Bents 2 and 3 because the superstructure constrains the bents to move together. This might not be the case when the bridge has significant foundation flexibility that can result from rotational and/or translational foundation movements.

Max $\mu_D = \frac{10.61}{8.74} = 1.21 < 5$ (Bent 2) OK (*SDC* 4.4.1-1)

Max $\mu_D = \frac{10.61}{9.40} = 1.13 < 5$ (Bent 3) OK (*SDC* 4.4.1-1)

The global displacement capacity to demand is checked as:

Δ_C =26.63 in. > Δ_D = 10.61 in., (Bent 2) OK

Δ_C =29.51 in. > Δ_D = 10.61 in., (Bent 3) OK

20.1.3.10 Step 8 – Check P - *Δ* Effects

20.1.3.10.1 Check Transverse Direction

We have relatively heavily loaded tall columns, but *P-Δ* effect is checked as:

Bent 2 Columns

P_{dl} = 1,729 kips, $M_p^{col} = 13{,}232$ kip–ft

The maximum seismic displacement Δ_D = 12.81 in.

$$\frac{P_{dl}\Delta_D}{M_p^{col}} = \frac{(1{,}729)(12.81)}{(13{,}232)(12)} = 0.14 < 0.25$$ OK. (*SDC* 4.4.4-1)

Bent 3 Columns

P_{dl} = 1,676 kips, $M_p^{col} = 13{,}161$ kip–ft

The maximum seismic displacement Δ_D = 11.54 in.

$$\frac{P_{dl}\Delta_D}{M_p^{col}} = \frac{(1{,}676)(11.54)}{(13{,}161)(12)} = 0.12 < 0.25$$ OK (*SDC* 4.4.4-1)

Note that the column sections meet the *P-Δ* requirements.

20.1.3.10.2 Check Longitudinal Direction

Bent 2 Columns

$$\frac{P_{dl}\Delta_D}{M_p^{col}} = \frac{(1,729)(10.61)}{(13,232)(12)} = 0.12 < 0.25 \qquad \text{OK} \qquad (SDC\ 4.4.4\text{-}1)$$

Bent 3 Columns

$$\frac{P_{dl}\Delta_D}{M_p^{col}} = \frac{(1,676)(10.61)}{(13,161)(12)} = 0.11 < 0.25 \qquad \text{OK} \qquad (SDC\ 4.4.4\text{-}1)$$

20.1.3.11 Step 9 – Check Bent Minimum Flexural Capacity

20.1.3.11.1 Check Transverse Direction

From the transverse pushover analysis shown in Appendix 20.1.3-7b of Bent 2, at 10% of the tributary weight applied as lateral load, the maximum moment demand obtained is 10,521 kip-ft. Also, the minimum plastic moment capacity of Bent 2 column (Back Side Column) is 11,911 kip-ft. Therefore, based on *SDC* section 5.3.6.1.

10% of tributary weight = 0.1(2)(1,729) = 346 kips

$M_{p,min}$ = 11,911 kip-ft (see Appendix 20.1.3-5)

$M_{D,max}$ = 10,521 kip-ft (see Appendix 20.1.3-7b)

$M_{p,min} > M_{D,max}$ OK (*SDC* 5.3.6.1)

20.1.3.11.2 Check Longitudinal Direction

Using *CSiBridge* longitudinal pushover analysis the maximum moment demand at ten percent of the tributary weight applied laterally is obtained as 10,694 kip-ft and is shown in Appendix 20.1.3-11a&b.

10% of tributary weight = 0.1(8,372/2) =419 kips

$M_{p,min}$ = 13,161 kip-ft (see Appendix 20.1.3-3)

$M_{D,max}$ = 10,694 kip-ft (see Appendix 20.1.3-11b)

$M_{p,min} > M_{D,max}$ OK (*SDC* 5.3.6.1)

20.1.3.12 Step 10 – Perform Column Shear Design

20.1.3.12.1 Check Transverse Direction

(1) *Bent 2*

$M_o^{col} = 1.2 M_p^{col} = (1.2)(14{,}418) = 17{,}302$ kip-ft (includes-overturning effects).

(*SDC* 4.4.2.1-1)

The shear demand associated with the overstrength moment is as:

$$V_o^{col} = \frac{M_o^{col}}{L_{col}} = \frac{17{,}302}{44} = 394 \text{ kips}$$

<u>Concrete Shear Capacity, V_c</u>

For #7 hoops @ 6 in.,

A_b = 0.6 in.[2], D' = 72 – 2 – 2 - 2(0.875/2) = 66.25 in., s = 6 in.

$$\rho_s = \frac{4A_b}{D's} = \frac{(4)(0.6)}{(66.25)(6)} = 0.00604 \qquad (SDC\ C5.3.8.2\text{-}1)$$

f_{yh} = 60 ksi

$\rho_s f_{yh}$ = 0.00614(60) = 0.36 ksi > 0.35 ksi

Use $\rho_s f_{yh}$ = 0.35 ksi

Using the maximum value of the displacement ductility demand, μ_d = 1.43 (see calculation for Bent 2 Transverse pushover analysis), the shear capacity factor F1 is calculated as:

$$F1 = \frac{\rho_s f_{yh}}{0.15} + 3.67 - \mu_d = \frac{0.35}{0.15} + 3.67 - 1.43 = 4.57 > 3 \qquad (SDC\ 5.3.7.2\text{-}5)$$

Use $F1$ = 3.0

$$F2 = 1 + \frac{P_c}{2{,}000 A_g} = 1 + \frac{820(1000)}{2{,}000(4{,}071.5)} = 1.10 < 1.5 \quad \text{OK} \qquad (SDC\ 5.3.7.2\text{-}6)$$

Use $F2$ = 1.10

It is seen from the equations for concrete shear capacity, that the plastic hinge region is more critical as the capacity will be lower in this region. Furthermore, the shear capacity is reduced when the axial load is decreased. The controlling shear capacity will be found in the Back Side column.

The nominal concrete shear capacity inside- and outside -the plastic hinge regions are equal due to the shear capacity factor $F1 = 3$.

$$v_c = (F1)(F2)\sqrt{f'_c} = (3)(1.1)\sqrt{4{,}000} = 209 \text{ psi} < 4\sqrt{4{,}000} = 253 \text{ psi} \quad \text{OK } (\textit{SDC } 5.3.7.2\text{-}3)$$

$$A_e = 0.8A_g = (0.8)(4{,}071.5) = 3{,}257 \text{ in.}^2 \qquad (\textit{SDC } 5.3.7.2\text{-}2)$$

$$\therefore\ V_c = v_c A_e = (209)(3{,}257) = 680{,}713 \text{ lb} = 681 \text{ kips} \qquad (\textit{SDC } 5.3.7.2\text{-}1)$$

<u>Transverse Reinforcement Shear Capacity, V_s</u>

$$V_s = \frac{n\pi A_b f_{yh} D'}{2s} = \frac{(1.0)\pi(0.6)(60)(66.87)}{2(6)} = 630 \text{ kips} \qquad (\textit{SDC } 5.3.7.3\text{-}1)$$

The maximum shear reinforcement requirement is checked as:

$$V_s = 630 \text{ kips} < 8\sqrt{f'_c}A_e = (8)\sqrt{4{,}000}(3{,}257)/1{,}000 = 1{,}648 \text{ kips} \quad \text{OK} \qquad (\textit{SDC } 5.3.7.4\text{-}1)$$

The minimum shear reinforcement requirement is checked as:

$$A_v = 0.6 \text{ in.}^2 > 0.025\left(\frac{D's}{f_{yh}}\right) = (0.025)\left(\frac{66.87(6)}{60}\right) = 0.17 \text{ in.}^2 \qquad \text{OK} \qquad (\textit{SDC } 5.3.7.5\text{-}1)$$

The shear capacity is:

$$\phi V_n = (1.0)(V_c + V_s) = (1.0)(681 + 630) = 1{,}311 \text{ kips} > V_o^{col} = 394 \text{ kips} \qquad \text{OK}$$

(2) *Bent 3*

$$M_o^{col} = 1.2M_p^{col} = (1.2)(14{,}458) = 17{,}350 \text{ kip-ft} \quad \text{(see Appendix 20.1.3-4) } (\textit{SDC } 4.4.2.1\text{-}1)$$

$$V_o^{col} = \frac{M_o^{col}}{L_{col}} = \frac{17{,}350}{47} = 370 \text{ kips}$$

The maximum column shear demand is 370 kips. Going through a similar calculation as was done for Bent 2 columns, we determine that:

$F1 = 3.0$, $P_c = 736$ kips for back side column $\rightarrow$ $F2 = 1.09$

$$V_c = v_c A_e = (207)(3{,}257)/1000 = 674 \text{ kips}$$

$$\phi V_n = (1.0)(V_c + V_s) = (1.0)(674 + 630) = 1{,}304 \text{ kips} > V_o^{col} = 370 \text{ kips} \qquad \text{OK}$$

20.1.3.12.2 Check Longitudinal Direction

(1) Bent 2

$$V_o^{col} = 1.2V_p = \left(\frac{1.2M_p}{L_{col}}\right) = \frac{(1.2)(13{,}232)}{44} = 361\text{ kips}$$

This corresponds to the maximum shear value of V_p = 310 kips/column obtained from the *CSiBridge* pushover analysis.

For μ_D = 1.04, $F1$ = 4.96 > 3. Use $F1$ = 3.

For dead load axial force, factor $F2$ = 1.21

v_c = 230 psi which gives V_c = 749 kips

V_s = 630 kips as calculated before.

$$\phi V_n = (1.0)(V_c + V_s) = (1.0)(749 + 630) = 1{,}379 \text{ kips} > V_o^{col} = 361 \text{ kips} \quad \text{OK}$$

(2) Bent 3

$$V_o^{col} = 1.2V_p = \left(\frac{1.2M_p}{L_{col}}\right) = \frac{(1.2)(13{,}161)}{47} = 336\text{ kips}$$

This corresponds to the maximum shear value of V_p = 290 kips/column obtained from the *CSiBridge* pushover analysis.

For μ_D = 1.09, factor $F1$ = 4.91 > 3. Use $F1$ = 3

For dead load axial force, factor $F2$ = 1.21

v_c = 230 psi which gives V_c = 749 kips

V_s = 630 kips as calculated earlier.

$$\phi V_n = (1.0)(V_c + V_s) = (1.0)(749 + 630) = 1{,}379 \text{ kips} > V_o^{col} = 336 \text{ kips} \quad \text{OK}$$

20.1.3.13 Step 11 – Design Column Shear Key

20.1.3.13.1 Determine Shear Key Reinforcement

Since the net axial force on both columns of Bent 2 is compressive, the area of interface shear key, required A_{sk} is given by:

$$A_{sk} = \frac{1.2\left(V_o^{col} - 0.25P\right)}{f_y} \qquad (\textit{SDC}\ 7.6.4\text{-}1)$$

P = 820 kips (column with the lowest axial load at Bent 2) –see Appendix 20.1.3-5

The shear force associated with the column overstrength moment is as:

V_o^{col} = shear force associated with the column overstrength moment

$$V_o^{col} = \begin{cases} 394 \text{ kips} & \text{For Bent 2} \\ 370 \text{ kips} & \text{For Bent 3} \end{cases}$$

See Step 10 – Perform Column Shear Design and Appendix 20.1.3-9.

Therefore, $V_o^{col} = 394$ kips

$$A_{sk} = \frac{1.2\left[394 - 0.25(820)\right]}{60} = 3.78 \text{ in.}^2$$

Provide 6#8 dowels in column key ($A_{sk,provided}$ =4.74 in.^{2} > 3.78 in.^{2} OK)

Dowel Cage diameter: Preferred spacing of #8 bars = 4.25 in. diameter of dowel cage = (6) (4.25)/π = 8.1 in. say 9 in. cage

20.1.3.13.2 Determine Concrete Area Engaged in Shear Transfer, A_{cv}

$$A_{cv} \geq \begin{cases} \dfrac{4V_o^{col}}{f'_c} = \dfrac{4(394)}{4} = 394 \text{ in.}^2 \\ 0.67V_o^{col} = 264 \text{ in.}^2 \end{cases} \qquad (SDC\ 7.6.4\text{-}3)$$

Per *SDC* Section 7.6.4, A_{cv} shall not be less than that required to meet the axial resistance requirements specified in Section 5 AASHTO-CA BDS Article 5.6.4.4 (AASHTO 2017).

$$P_n = \phi(0.85)\left[0.85f'_c\left(A_g - A_{st}\right) + f_y A_{st}\right] \qquad \text{(AASHTO 5.6.4.4-2)}$$

Using the largest axial load with overturning effects P = 2,638 kips (see Appendix 20.3-4) and ϕ = 1.0, we have:

$$P_n = (1.0)(0.85)\left[0.85(4)\left(A_g - 4.74\right) + (60)(4.74)\right] = 2{,}638 \text{ kips}$$

A_g = 833.9 in.^{2} > 394 in.^{2}

Therefore, $A_{cv,\ reqd,}$ = 834 in.^{2}

Diameter of $A_{cv} = \sqrt{\dfrac{(834)(4)}{\pi}} = 32.6$ in.

Use A_{cv} diameter = 33 in. (see Figure 20.1.3-6)

The minimum development length of epoxy-coated longitudinal reinforcement into the column, l_{ac}:

$$l_{ac} = 1.2(24d_{bl}) = (1.2)(23(1.0) = 29 \text{ in. Use 30 in.} \qquad (SDC\ 8.3.1.1\text{-}1)$$

The column shear key detail is shown in Figure 20.1.3-6.

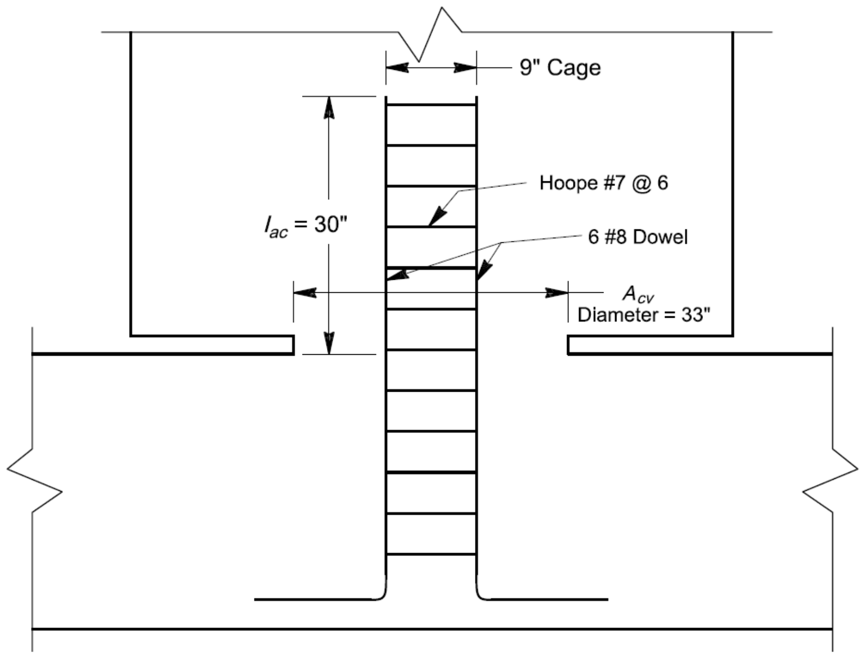


Figure 20.1.3-6 Column Shear Key

20.1.3.14 Step 12 – Check Bent Cap Flexural and Shear Capacity

20.1.3.14.1 Check Bent Cap Flexural Capacity

The capacity protected member design for strength limit states resulted in the following main reinforcement for the bent cap:

Top Reinforcement: 22 - #11 and #8 @12” top bars within the effective width of the section.

Bottom Reinforcement: 24 - #11 bottom bars within the effective width of the section.

Ignoring the side face reinforcement, the positive and negative flexural capacities of the bent cap are estimated to be M_{ne}^{+ve} = 20,672 kip-ft and M_{ne}^{-ve} = 24,644 kip-ft. Appendices 20.1.3-12 and 20.1.3-13 show these values, which are based on the expected material properties, see *SDC* Table 3.3.3-1.

The seismic flexural and shear demands in the bent cap are calculated corresponding to the column overstrength moment. These demands are obtained from a new *CSiBridge* pushover analysis (include dead loads and the lateral load, but the effect of the secondary prestressed moment at the bent due to the skewness is small and ignored) of Bent 2 with the column moment capacity taken as M_o^{col} . As shown in Appendix 20.1.3-14 (right pushover), bent cap moment demands are:

$M_D^{+ve} = 18{,}057 \text{ kip} - \text{ft} < \phi M_{ne}^{+ve} = 20{,}672$ kip-ft OK

$M_D^{-ve} = 22{,}225 \text{ kip} - \text{ft} < \phi M_{ne}^{-ve} = 24{,}644$ kip-ft OK

The negative moment demand reported above is taken at the center of the column and the positive moment at the face of the column. Bent 2 shear demand which is "$d_V = 0.72h$" away from the face of the column obtained from the above pushover analysis, V_D = 1734 kips.

20.1.3.14.2 Check Bent Cap Shear Capacity

Nominal shear resistance of the bent cap, V_n is the lesser of:

$$V_n = V_c + V_s + V_p \quad \text{(AASHTO 5.7.3.3-1)}$$

and

$$V_n = 0.25 f_c^{'} b_v d_v + V_p \quad \text{(AASHTO 5.7.3.3-2)}$$

where:

$$V_c = 0.0316\beta\sqrt{f_c^{'}} b_v d_v \quad \text{(AASHTO 5.7.3.3-3)}$$

$$V_s = \frac{A_v f_y d_v \left(\cot\theta + \cot\alpha\right)\sin\alpha}{s} \quad \text{(AASHTO C5.7.3.3-1)}$$

V_p = 0 (bent cap is not prestressed)

α = 90° (angle of inclination of transverse reinforcement to longitudinal axis)

b_v = effective web width = 8 ft = 96 in.

d_v = effective shear depth = distance between the resultants of the tensile and compressive forces due to flexure, not to be taken less than the greater of $0.9d_e$ or $0.72h$ (see AASHTO Article 5.7.2.8).

$0.72\ h = 0.72\ (81) = 58.3$ in.

Assuming clear distance from cap bottom to main bottom bars and cap top to main top bars = 5 in.

d_e = cap effective depth = 81-5-1.693/2 = 75.2 in.

$0.9d_e = 0.9(75.2) = 67.7$ in. > 58.3 in.

Therefore, $d_{v,min}$ = 67.7 in

Method 1 of AASHTO Article 5.7.3.4.1 (AASHTO 2017) is used to determine the values of β and θ (the bent cap section is non-prestressed, and the effect of any axial tension is assumed to be negligible).

Determine β *and* θ:

Using AASHTO BDS -CA (2017) Table B5.2-1 for sections with minimum amount of transverse reinforcement.

Shear Stress:

$$v_u = \frac{V_u}{\phi b_v d_v} = \frac{2{,}076}{1.0(96)(67.7)} = 0.319 \text{ ksi}$$

Shear Stress Factor:

$$\frac{\nu_u}{f'_c} = \frac{0.267}{4} = 0.067$$

Determine ε_x at mid-depth:

$$\varepsilon_x = \left(\frac{\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})} \right) \quad \text{(AASHTO B5.2-3)}$$

There is no prestressing force and axial force in the bent cap, the above equation reduces to:

$$\varepsilon_x = \left(\frac{\frac{|M_u|}{d_v} + 0.5|V_u|\cot\theta}{2(E_sA_s)} \right)$$

For 24-#11,

As = area of fully developed steel on flexural tension side of the member = 37.44 in^2

Assuming 0.5(cot θ) = 1 and absolute values of M_u = 1, 102 kip-ft and V_u= 2,076 kips (See BDP 5.6, Section 5.6.6.3.4 5.6-40 for M_u and V_u values).

$$\varepsilon_x = \left(\frac{\frac{|1102 \times 12|}{67.7} + 2{,}076}{2(29{,}000 \times 37.44)} \right) = 1.046(10)^{-3}$$

From Table B5.2-1: β = 2.18 and θ = 36.7°

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v = 0.0316(2.18)(\sqrt{4})(96)(67.7) = 895 \text{ kips}$$

Assuming 6-legged, #6 stirrups @ 7 in. C-C. transverse reinforcement (see Figure 20.1.3-19).

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{6(0.44)(60)(67.7)(\cot 36.7)}{7} = 2{,}055 \text{ kips}$$

$$V_n = V_c + V_s = 895 + 2{,}055 = 2{,}950 \text{ kips}$$

$$V_n = 0.25 f'_c b_v d_v = 0.25(4)(96)(67.7) = 6{,}499 \text{ kips} > 2{,}950 \text{ kips}$$

$$V_n = 2{,}950 \text{ kips}$$

$$\phi V_n = 1.0(2{,}950) = 2{,}950 \text{ kips} > V_D = 1{,}734 \text{ kips} \qquad \text{OK}$$

20.1.3.15 Step 13 – Calculate Column Seismic Load Moments

20.1.3.15.1 Determine Dead Load, Additional Dead Load, and Prestress Secondary Moments at Column Tops/Deck Soffit

For this bridge, the top of bent support results from the *CTBridge* (Table 20.1.3-8) will need to be transformed to the consistent planar coordinate system (i.e., the plane formed by the centerline of the bridge and the vertical axis) to ensure consistency with *CSiBridge* results and to account for the bridge skew. To do so, the following coordinate transformation (see Figure 20.1.3-7) will be applied to the top of column moments obtained from the *CTBridge*.

Table 20.1.3-8 Top of Bent Column Moments (kip-ft) from *CTBridge*

Bent	Skew (Degree)	DL			ADL			Sec. PS		
		M_z	M_y	M_{long}	M_z	M_y	M_{long}	M_z	M_y	M_{long}
2	20	-1824	664	-1941	-220	80	-234	-501	182	-533
3	20	1948	-709	2073	234	-85	249	535	-195	569

It is noted that the above values are for both columns in each bent.

(1) Moment at Column Top – Bent 2

Dead load and additional dead load moments (Figure 20.1.3-8)

Column moment at the base,

$M_{dl}^{col-bottom} = 0$ kip-ft (*CTBridge* Output)

Column moment at the deck soffit,

$$M_{dl}^{col-top} = (-1{,}941) + (-234) = -2175 \text{ kip-ft}$$

Secondary prestress moments (Figure 20.1.3-9)

Column moment at the base,

$$M_{p/s}^{col-bottom} = 0 \text{ kip-ft} \quad (\textit{CTBridge} \text{ Output})$$

Column moment at the deck soffit,

$$M_{p/s}^{col-top} = -533 \text{ kip-ft}$$

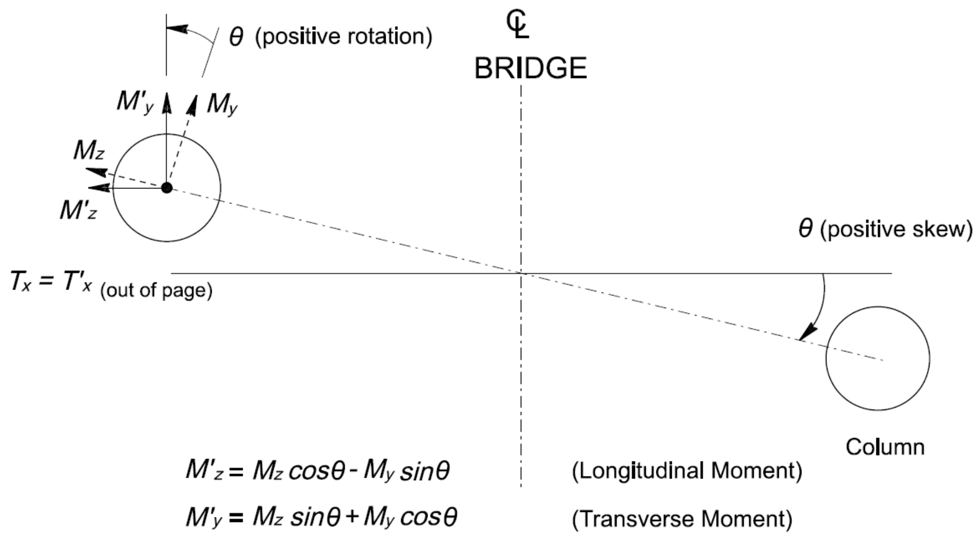


Figure 20.1.3-7 Coordinate Transformation from Skewed to Unskewed Configuration

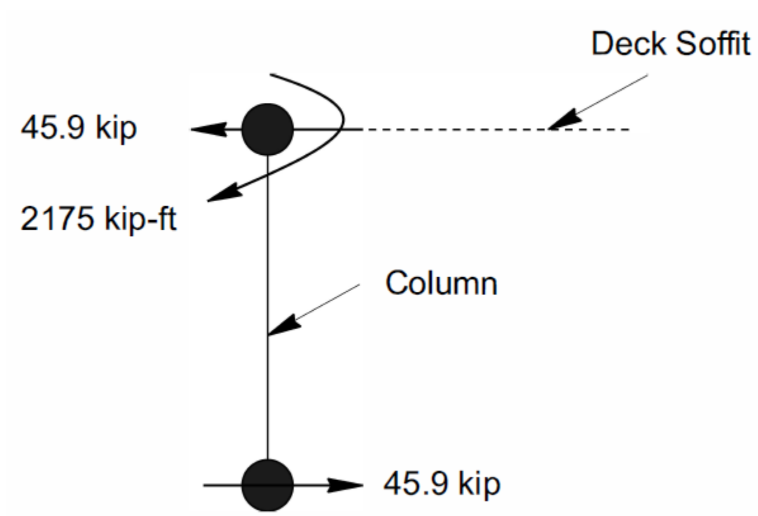


Figure 20.1.3-8 Free Body Diagram Showing Equilibrium of Dead Loading at Bent 2

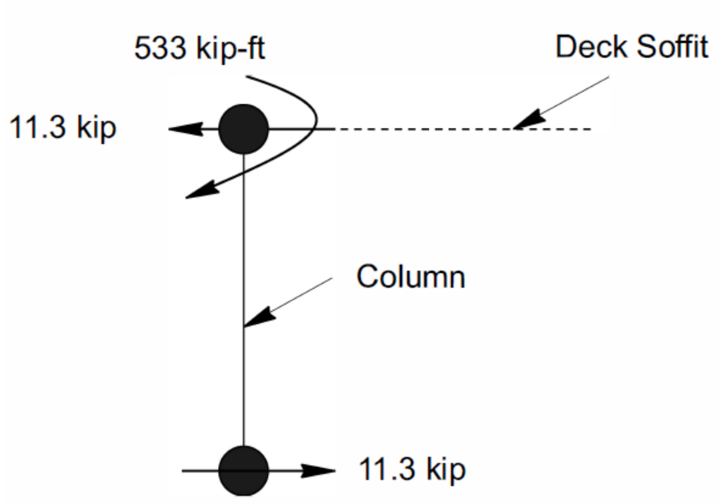


Figure 20.1.3-9 Free Body Diagram Showing Equilibrium of Secondary Prestress Forces at Bent 2

(2) Moment at Column Top - Bent 3

Dead load and additional dead load moments

Column moment at the base,

$$M_{dl}^{col-top} = 0 \text{ kip-ft} \quad (\textit{CTBridge} \text{ Output})$$

Column moment at the deck soffit,

$$M_{dl}^{col-top} = \{(+2073) + (+249)\} = +2322 \text{ kip-ft}$$

Secondary prestress moments

Column moment at the base,

$$M_{p/s}^{col-bottom} = 0 \text{kip-ft} \quad (\textit{CTBridge} \text{ Output})$$

Column Moment at the deck soffit,

$$M_{p/s}^{col-top} = +569 \text{kip-ft}$$

20.1.3.15.2 Determine Earthquake Moments in the Superstructure

(1) Dead Load and Additional Dead Load Moments

CTBridge output lists these moments at every 1/10th point of the span length and at the face of supports (see Table 20.1.3-9).

(2) Secondary Prestress Moments

CTBridge output lists these moments at every 1/10th point of the span length and at the face of supports (see Table 20.1.3-9).

Table 20.1.3-9 Dead Load and Secondary Prestress Moments from *CTBridge* Output

	Location		Whole Super Structure Width			Per Girder		
	x/L	*x*	M_{DL} (kip-ft)	M_{ADL} (kip-ft)	$M_{P/S}$ (kip-ft)	M_{DL} (kip-ft)	M_{ADL} (kip-ft)	$M_{P/S}$ (kip-ft)
Span 1	Support	1.5	1105	132	133	221	26	27
	0.1	12.6	8094	968	1198	1619	194	240
	0.2	25.2	13546	1625	2393	2709	325	479
	0.3	37.8	16411	1971	3587	3282	394	717
	0.4	50.4	16692	2006	4779	3338	401	956
	0.5	63	14391	1729	5971	2878	346	1194
	0.6	75.6	9507	1142	7161	1901	228	1432
	0.7	88.2	2041	243	8351	408	49	1670
	0.8	100.8	-8009	-967	9539	-1602	-193	1908
	0.9	113.4	-20660	-2488	10733	-4132	-498	2147
	Support	123	-32115	-3856	11352	-6423	-771	2270
Span 2	Support	129	-34003	-4083	11011	-6801	-817	2202
	0.1	142.8	-17074	-2059	11340	-3415	-412	2268
	0.2	159.6	-789	-97	11345	-158	-19	2269
	0.3	176.4	10905	1312	11348	2181	262	2270
	0.4	193.2	18009	2168	11350	3602	434	2270
	0.5	210	20525	2471	11352	4105	494	2270
	0.6	226.8	18452	2221	11351	3690	444	2270
	0.7	243.6	11790	1418	11353	2358	284	2271
	0.8	260.4	540	62	11356	108	12	2271
	0.9	277.2	-15301	-1846	11360	-3060	-369	2272
	Support	291	-31863	-3827	11048	-6373	-765	2210
Span 3	Support	297	-29961	-3599	11410	-5992	-720	2282
	0.1	305.8	-19981	-2407	10788	-3996	-481	2158
	0.2	317.6	-8677	-1048	9586	-1735	-210	1917
	0.3	329.4	341	38	8391	68	8	1678
	0.4	341.2	7094	851	7194	1419	170	1439
	0.5	353	11583	1392	5995	2317	278	1199
	0.6	364.8	13808	1659	4797	2762	332	959
	0.7	376.6	13769	1654	3598	2754	331	720
	0.8	388.4	11465	1375	2399	2293	275	480
	0.9	400.2	6892	824	1200	1378	165	240
	Support	410.5	1007	120	143	201	24	29

(1) Case 1 Earthquake Loading: Bridge moves from Abutment 1 towards Abutment 4

As shown in Figure 20.1.3-10, such loading results in positive moments in the columns according to the sign convention used here.

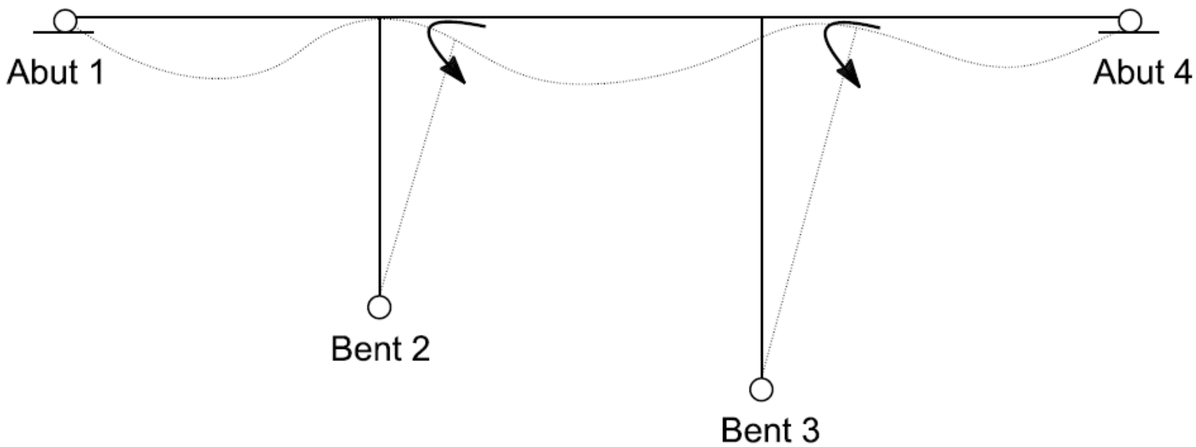


Figure 20.1.3-10 Seismic Loading Case "1" Producing Positive Moments in Columns

As calculated previously, the columns have already been "pre-loaded" by:

$$M_{dl}^{col-top} + M_{p/s}^{col-top} = \{(-2175) + (-533)\} = -2,708 \text{ kip-ft} \qquad \text{(Bent 2)}$$

$$M_{dl}^{col-top} + M_{p/s}^{col-top} = \{(+2322) + (+569)\} = +2,891 \text{ ki-ft} \qquad \text{(Bent 3)}$$

Column moment generated by seismic loading at column soffit is:

$$M_{eq}^{col-top} = 1.2M_{p}^{col-top} - \left(M_{dl}^{col} + M_{p/s}^{col-top}\right)$$

$$= 1.2(2)(13,232) - (-2,175 - 533) = +34,465 \text{ kip-ft} \qquad \text{(Bent 2)}$$

It should be noted that the secondary prestress moment is included because doing so results in increased seismic demand on the column and hence in the superstructure. Figure 20.1.3-11 schematically explains this superposition approach.

$$M_{eq}^{col-top} = 1.2(2)(13,161) - (2,322 + 0) = +29,265 \text{ kip-ft} \qquad \text{(Bent 3)}$$

It should be noted that for Bent 3, the effect of secondary prestress moments is not included because doing so results in increased seismic moments in the columns and hence in the superstructure.

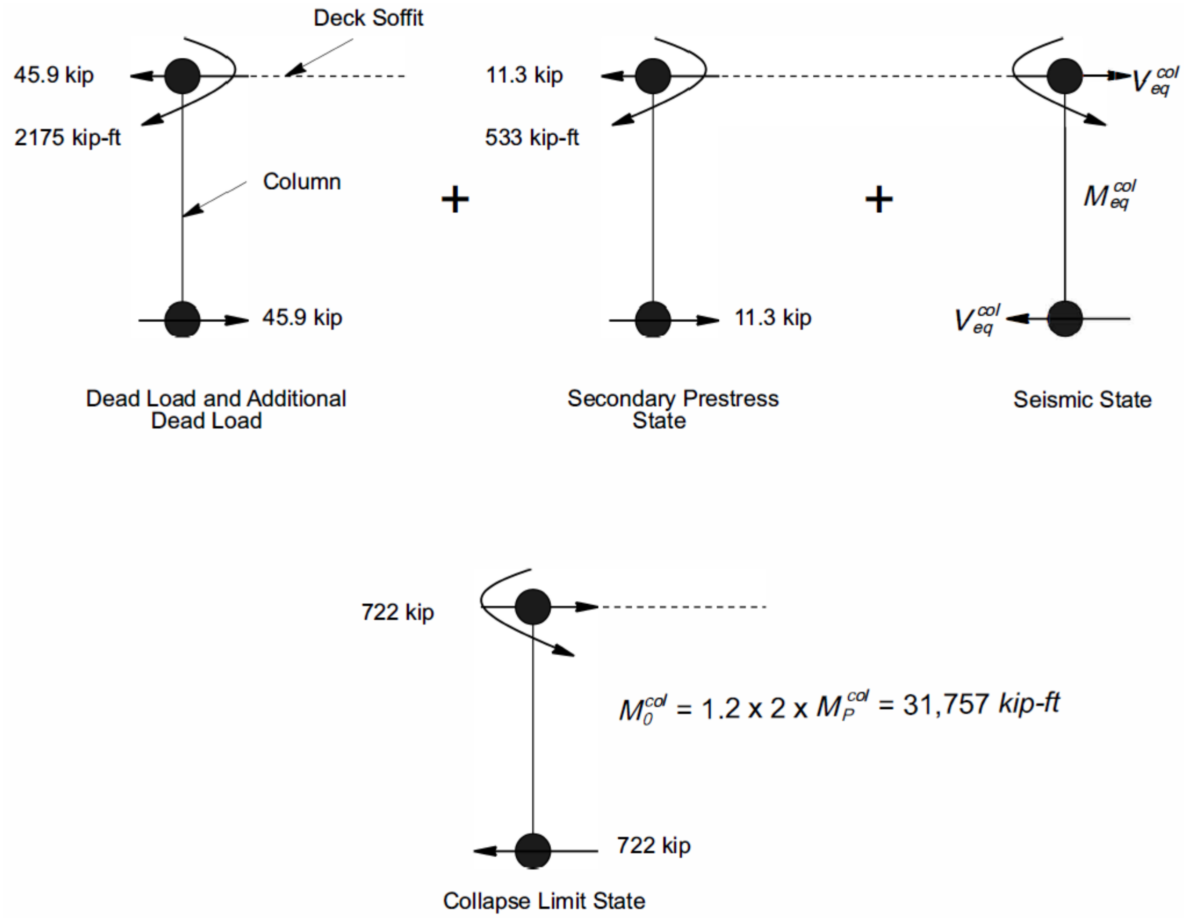


Figure 20.1.3-11 Superposition of Column Forces at Bent 2 for Loading Case “1”

(2) Case 2 Earthquake Loading: Bridge moves from Abutment 4 towards Abutment 1

As shown in Figure 20.1.3-12, such loading results in negative moments in the columns according to our sign convention.

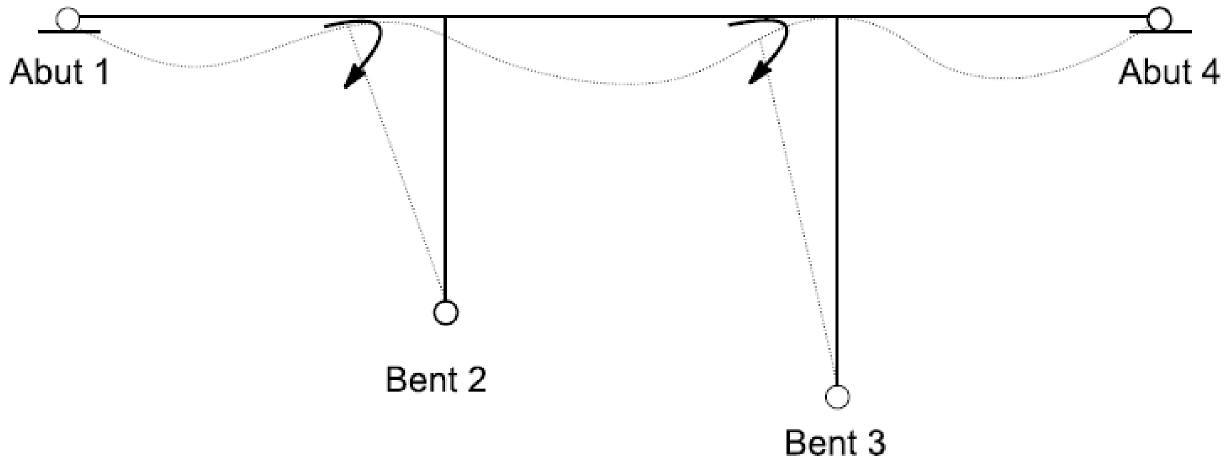


Figure 20.1.3-12 Seismic Loading Case "2" Producing Negative Moments in Columns

Bent 2

$$M_{eq}^{col-top} = 1.2M_{p}^{col} - \left(M_{dl}^{col} + M_{p/s}^{col}\right)$$
$$M_{eq}^{col-top} = 1.2(2)(-13{,}232) - (-2175 + 0) = -29{,}582 \text{ kip} - \text{ft}$$

Bent 3

$$M_{eq}^{col-top} = 1.2M_{p}^{col} - \left(M_{dl}^{col} + M_{p/s}^{col}\right)$$

$$M_{eq}^{col-top} = 1.2(2)(-13{,}161) - (2322 + 569) = -33{,}340 \text{ kip-ft}$$

Figure 20.1.3-13 schematically shows the free body diagram at Bent 2 for this seismic loading case.

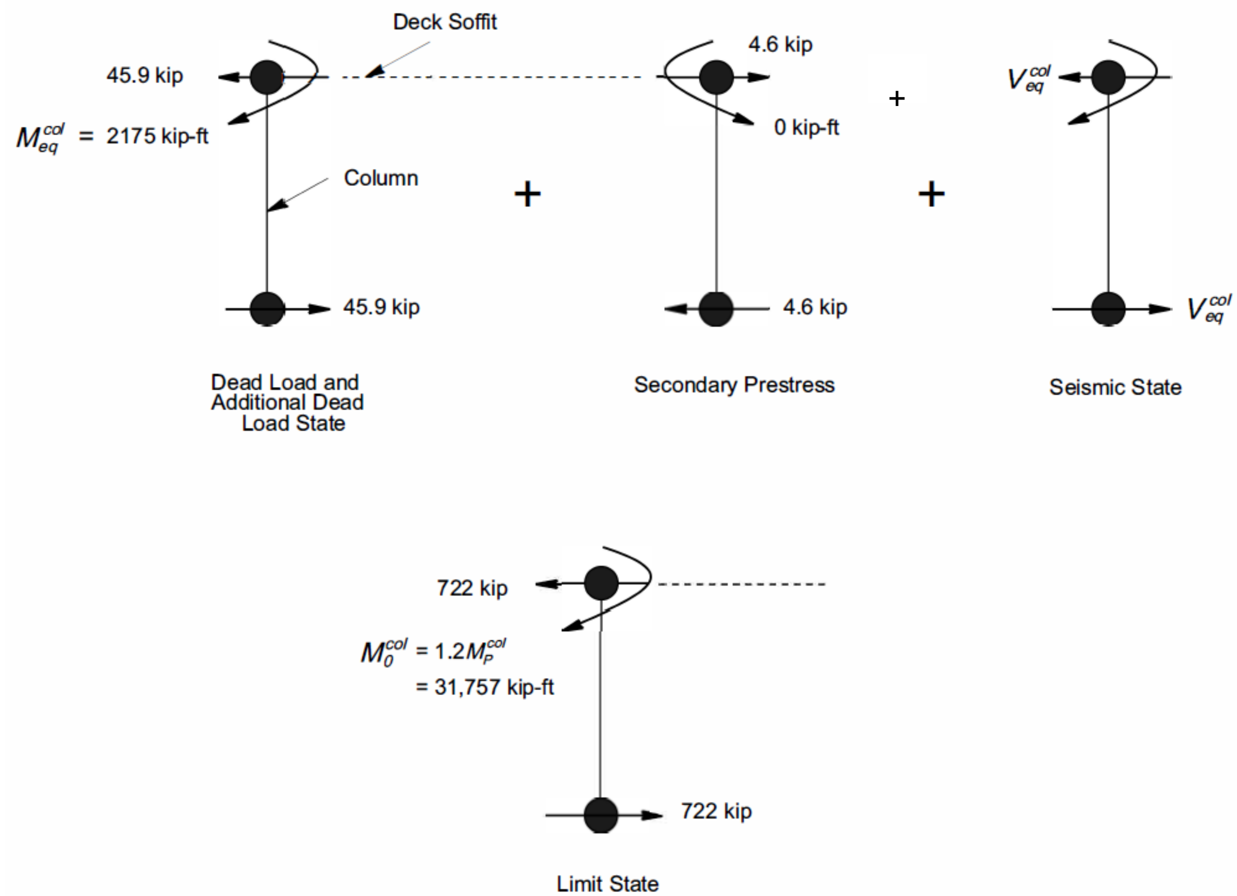


Figure 20.1.3-13 Superposition of Column Forces at Bent 2 for Loading Case “2”

20.1.3.16 Step 14 – Distribute Column Seismic Moments into Superstructure

The static non-linear “push-over” frame analysis on *CSiBridge* is used to distribute the column earthquake moments $M_{eq}^{col-top}$ into the superstructure.

Also, the superstructure dead loads are removed from the model and the distributed earthquake moments (see Table 20.1.3-10) are added with dead load and secondary prestressing moments obtained from *CTBridge* to get final moment demands on the superstructure (see Table 20.1.3-11). Appendix 20.1.3-15 shows portions of the output file for Case 1 (i.e., right push). Table 20.1.3-11 lists the distribution of earthquake moments in the superstructure as obtained from these pushover analyses.

20.1.3.17 Step 15 – Calculate Superstructure Seismic Moment Demands at Locations of Interest

Let us calculate the superstructure moment demand at the face of the cap on each side of the column.

*Example Calculation - **Bent 2**: Left and Right Faces of Bent Cap*

The effective section width is:

$B_{eff} = D_c + 2D_s = 6.00 + 2(6.75) = 19.50$ ft. (*SDC* 7.2.1.1-1)

Based on the column location and the girder spacing, it can easily be concluded that the girder aligned along the centerline of the bridge lies outside the effective width. Therefore, at the face of the bent cap, four girders are within the effective width. All five girders fall within the effective width for all the other tenths point locations. Note that the per-girder values used below have previously been listed in Table 20.1.3-9 and effective section width is used for capacity determination only.

Case 1

$$M_{dl}^{L} = \{(-6,423) + (-771)\}(5) = -35,970 \text{ kip-ft}$$

$$M_{dl}^{R} = \{(-6,801) + (-817)\}(5) = -38,090 \text{ kip-ft}$$

$$M_{p/s}^{L} = \{+2,270\}(5) = +11,350 \text{ kip-ft}$$

$$M_{p/s}^{R} = \{+2,202\}(5) = +11,010 \text{ kip-ft}$$

$$M_{eq}^{L} = -15,877 \text{ kip-ft (see Table 20.1.3-10)}$$

$$M_{eq}^{R} = +21,204 \text{ kip-ft (see Table 20.1.3-10)}$$

The superstructure moment demands are then calculated as:

$$M_{D}^{L} = M_{dl}^{L} + M_{p/s}^{L} + M_{eq}^{L} = (-35,970) + (11,350^{*}) + (-15,877) = -51,847 \text{ kip-ft}$$

$$M_{D}^{R} = M_{dl}^{R} + M_{p/s}^{R} + M_{eq}^{R} = (-38,090) + (11,010) + (21,204) = -5,876 \text{ kip-ft}$$

Table 20.1.3-8 lists these superstructures seismic moment demands.

Case 2

$$M_{eq}^{L} = +9,904\, kip-ft; \quad M_{eq}^{R} = -17,021 \text{ kip-ft}$$

$$M_{D}^{L} = (-35,970) + (11,350) + (9,904) = -14,716 \text{ kip-ft}$$

$$M_{D}^{R} = (-38,090) + (11,010^{*}) + (-17,021) = -55,111 \text{ kip-ft}$$

* - The prestressing secondary effect is ignored as doing so results in a conservatively

higher seismic demand in the superstructure.

Bent 3

Similarly, we obtain the following:

$$M_D^L = \begin{cases} -54{,}615 \text{ kip-ft} & \text{Case 1} \\ -10{,}042 \text{ kip-ft} & \text{Case 2} \end{cases}$$

$$M_D^R = \begin{cases} -4{,}945 \text{ kip-ft} & \text{Case 1} \\ -49{,}584 \text{ kip-ft} & \text{Case 2} \end{cases}$$

Seismic moment demands along the superstructure length have been summarized in the form of moment envelope values (see Table 20.1.3-11).

$$M_{positive} = M_{EQ,max} + M_{DL} + M_{ADL} + M_{p/s}^{*}$$

$$M_{negative} = M_{EQ,min} + M_{DL} + M_{ADL} + M_{p/s}^{*}$$

* Only include $M_{p/s}$ when it maximizes $M_{positive}$

** Only include $M_{p/s}$ when it minimizes $M_{negative}$

Table 20.1.3-10 Earthquake Moments from *CSiBridge* Output

Location			M_{EQ} (kip - ft) *CSiBridge* Output	
			Case - 1	Case - 2
Span 1	0	0	0	0
	Support	1.5	-194	121
	0.1	12.6	-1626	1015
	0.2	25.2	-3253	2029
	0.3	37.8	-4879	3044
	0.4	50.4	-6506	4058
	0.5	63	-8132	5073
	0.6	75.6	-9759	6087
	0.7	88.2	-11385	7102
	0.8	100.8	-13012	8116
	0.9	113.4	-14638	9131
	Support	123	-15877	9904
	1	126	-16264	10146
Span 2	0	0	21946	-17701
	Support	3	21204	-17021
	0.1	16.8	17793	-13893
	0.2	33.6	13640	-10085
	0.3	50.4	9486	-6277
	0.4	67.2	5333	-2469
	0.5	84	1180	1338
	0.6	100.8	-2973	5146
	0.7	117.6	-7127	8954
	0.8	134.4	-11280	12762
	0.9	151.2	-15433	16570
	Support	165	-18845	19698
	1	168	-19586	20378
Span 3	0	0	12424	-16443
	Support	3	12109	-16025
	0.1	11.8	11182	-14798
	0.2	23.6	9940	-13154
	0.3	35.4	8697	-11510
	0.4	47.2	7455	-9866
	0.5	59	6212	-8221
	0.6	70.8	4970	-6577
	0.7	82.6	3727	-4933
	0.8	94.4	2485	-3289
	0.9	106.2	1242	-1644
	Support	116.5	158	-209
	1	118	0	0

Table 20.1.3-11 Moment Demand Envelope

Location						Case 1	Case 2	Case 1		Case 2		Envelope	
			M_{DL}	M_{ADL}	$M_{P/S}$	M_{EQ}	M_{EQ}	$M_{positive}$	$M_{negative}$	$M_{positive}$	$M_{negative}$	$M_{positive}$	$M_{negative}$
	x/L	x (ft)	(kip-ft)	(kip-ft)	(kips-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)
Span 1	Support	1.5	1105	132	133	-194	121	1176	1043	1490	1357	1490	1043
	0.1	12.6	8094	968	1198	-1626	1015	8633	7435	11274	10076	11274	7435
	0.2	25.2	13546	1625	2393	-3253	2029	14311	11918	19593	17200	19593	11918
	0.3	37.8	16411	1971	3587	-4879	3044	17089	13502	25012	21425	25012	13502
	0.4	50.4	16692	2006	4779	-6506	4058	16972	12192	27536	22756	27536	12192
	0.5	63	14391	1729	5971	-8132	5073	13959	7988	27164	21193	27164	7988
	0.6	75.6	9507	1142	7161	-9759	6087	8051	890	23897	16736	23897	890
	0.7	88.2	2041	243	8351	-11385	7102	-750	-9102	17737	9386	17737	-9102
	0.8	100.8	-8009	-967	9539	-13012	8116	-12448	-21987	8680	-859	8680	-21987
	0.9	113.4	-20660	-2488	10733	-14638	9131	-27054	-37786	-3284	-14017	-3284	-37786
	Support	123	-32115	-3856	11352	-15877	9904	-40496	-51848	-14714	-26067	-14714	-51848
Span 2	Support	129	-34003	-4083	11011	21115	-17021	-5960	-16971	-44096	-55107	-5960	-55107
	0.1	142.8	-17074	-2059	11340	17292	-13893	9500	-1840	-21686	-33026	9500	-33026
	0.2	159.6	-789	-97	11345	12639	-10085	23098	11753	374	-10971	23098	-10971
	0.3	176.4	10905	1312	11348	7985	-6277	31550	20202	17287	5940	31550	5940
	0.4	193.2	18009	2168	11350	3332	-2469	34860	23509	29059	17708	34860	17708
	0.5	210	20525	2471	11352	-1053	1338	33296	21943	35687	24335	35687	21943
	0.6	226.8	18452	2221	11351	-4760	5146	27264	15914	37170	25820	37170	15914
	0.7	243.6	11790	1418	11353	-8466	8954	16095	4742	33516	22162	33516	4742
	0.8	260.4	540	62	11356	-12173	12762	-215	-11571	24720	13364	24720	-11571
	0.9	277.2	-15301	-1846	11360	-15880	16570	-21667	-33027	10782	-578	10782	-33027
	Support	291	-31863	-3827	11048	-18924	19698	-43567	-54615	-4945	-15993	-4945	-54615
Span 3	Support	297	-29961	-3599	11410	12109	-16025	-10042	-21451	-38175	-49584	-10042	-49584
	0.1	305.8	-19981	-2407	10788	11182	-14798	-418	-11206	-26399	-37187	-418	-37187
	0.2	317.6	-8677	-1048	9586	9940	-13154	9801	215	-13293	-22879	9801	-22879
	0.3	329.4	341	38	8391	8697	-11510	17467	9076	-2740	-11131	17467	-11131
	0.4	341.2	7094	851	7194	7455	-9866	22594	15400	5273	-1920	22594	-1920
	0.5	353	11583	1392	5995	6212	-8221	25182	19187	10749	4753	25182	4753
	0.6	364.8	13808	1659	4797	4970	-6577	25233	20437	13686	8890	25233	8890
	0.7	376.6	13769	1654	3598	3727	-4933	22748	19150	14087	10489	22748	10489
	0.8	388.4	11465	1375	2399	2485	-3289	17724	15325	11951	9552	17724	9552
	0.9	400.2	6892	824	1200	1242	-1644	10159	8958	7272	6072	10159	6072
	Support	410.5	1007	120	143	158	-209	1428	1285	1061	918	1428	918

20.1.3.18 Step 16 – Calculate Superstructure Seismic Shear Demands at Locations of Interest

Values of shear forces due to dead loads, additional dead loads, and the secondary prestress, as read from *CTBridge* output, are listed in Table 20.1.3-12.

Superstructure Seismic Shear Forces due to Seismic Moments, V_{eq}

Span 1, Case 1

Seismic Moment at Abutment 1, $M_{eq}^{(1)} = 0$ kip-ft

Seismic Moment at Bent 2 $M_{eq}^{(2)} = -16,264$ kip-ft

$$\text{Shear force in Span 1, } V_{eq} = \frac{\left(M_{eq}^{(2)} + M_{eq}^{(1)}\right)}{\text{Length of Span 1}} = \frac{(-16,264 + 0)}{126} = -129 \text{ kips}$$

Span 1, Case 2

Seismic Moment at Abutment 1, $M_{eq}^{(1)} = 0$ kip-ft

Seismic Moment at Bent 2, $M_{eq}^{(2)} = 9,960$ kip-ft

$$\text{Shear force in Span 1, } V_{eq} = \frac{\left(M_{eq}^{(2)} + M_{eq}^{(1)}\right)}{\text{Length of Span 1}} = \frac{(10,146 + 0)}{126} = 81 \text{ kips}$$

Similarly, the seismic shear forces for the remaining spans are calculated to be:

$$\textit{Span 2},\ V_{eq} = \begin{cases} -247 \text{ kips} & \text{Case 1} \\ +227 \text{ kips} & \text{Case 2} \end{cases}$$

$$\textit{Span 3},\ V_{eq} = \begin{cases} -109 \text{ kips} & \text{Case 1} \\ +139 \text{ kips} & \text{Case 2} \end{cases}$$

Table 20.1.3-13 lists these values. Table 20.1.3-14 lists the maximum shear demands summarized as a shear envelope.

$$V_{positive} = V_{EQ,max} + V_{DL} + V_{ADL} + V_{p/s}^{*}$$

$$V_{negative} = V_{EQ,min} + V_{DL} + V_{ADL} + V_{p/s}^{**}$$

$$V = \text{Greater of Absolute}\left(V_{positive}\right) \text{or Absolute}\left(V_{negative}\right)$$

* Only include V_{PS} when it maximizes $V_{positive}$

** Only include V_{PS} when it minimizes $V_{negative}$

Table 20.1.3-12 Dead Load and Secondary Prestress Shears Forces from *CTBridge* Output

	Location		Whole Super Structure Width			Per Girder		
	x/L	*x*	V_{DL} (kips)	V_{ADL} (kips)	$V_{P/S}$ (kips)	V_{DL} (kips)	V_{ADL} (kips)	$V_{P/S}$ (kips)
Span 1	Support	1.5	723	86	99	145	17	20
	0.1	12.6	535	65	97	107	13	19
	0.2	25.2	330	40	97	66	8	19
	0.3	37.8	125	15	97	25	3	19
	0.4	50.4	-80	-10	96	-16	-2	19
	0.5	63	-285	-34	96	-57	-7	19
	0.6	75.6	-490	-59	96	-98	-12	19
	0.7	88.2	-695	-84	96	-139	-17	19
	0.8	100.8	-900	-108	96	-180	-22	19
	0.9	113.4	-1108	-133	87	-222	-27	17
	Support	123	-1278	-152	85	-256	-30	17
Span 2	Support	129	1347	160	-10	269	32	-2
	0.1	142.8	1106	133	-2	221	27	0
	0.2	159.6	833	100	2	167	20	0
	0.3	176.4	559	67	2	112	13	0
	0.4	193.2	286	35	2	57	7	0
	0.5	210	13	2	2	3	0	0
	0.6	226.8	-260	-31	2	-52	-6	0
	0.7	243.6	-533	-64	2	-107	-13	0
	0.8	260.4	-806	-97	2	-161	-19	0
	0.9	277.2	-1079	-130	-4	-216	-26	-1
	Support	291	-1321	-157	-8	-264	-31	-2
Span 3	Support	297	1212	144	-122	242	29	-24
	0.1	305.8	1056	127	-106	211	25	-21
	0.2	317.6	860	104	-97	172	21	-19
	0.3	329.4	668	81	-97	134	16	-19
	0.4	341.2	476	57	-97	95	11	-19
	0.5	353	285	34	-99	57	7	-20
	0.6	364.8	93	11	-99	19	2	-20
	0.7	376.6	-99	-12	-99	-20	-2	-20
	0.8	388.4	-291	-35	-99	-58	-7	-20
	0.9	400.2	-484	-58	-99	-97	-12	-20
	Support	411	-659	-79	-99	-132	-16	-20

Table 20.1.3-13 Earthquake Shear Forces from *CSiBridge* Output

Location			V_{EQ} (kips) *CSiBridge* Output	
			Case - 1	Case - 2
Span 1	0	0	-129	81
	Support	1.5	-129	81
	0.1	12.6	-129	81
	0.2	25.2	-129	81
	0.3	37.8	-129	81
	0.4	50.4	-129	81
	0.5	63	-129	81
	0.6	75.6	-129	81
	0.7	88.2	-129	81
	0.8	100.8	-129	81
	0.9	113.4	-129	81
	Support	123	-129	81
	1	126	-129	81
Span 2	0	0	-247	227
	Support	3	-247	227
	0.1	16.8	-247	227
	0.2	33.6	-247	227
	0.3	50.4	-247	227
	0.4	67.2	-247	227
	0.5	84	-247	227
	0.6	100.8	-247	227
	0.7	117.6	-247	227
	0.8	134.4	-247	227
	0.9	151.2	-247	227
	Support	165	-247	227
	1	168	-247	227
Span 3	0	0	-109	139
	Support	3	-109	139
	0.1	11.8	-109	139
	0.2	23.6	-109	139
	0.3	35.4	-109	139
	0.4	47.2	-109	139
	0.5	59	-109	139
	0.6	70.8	-109	139
	0.7	82.6	-109	139
	0.8	94.4	-109	139
	0.9	106.2	-109	139
	Support	116.5	-109	139
	1	118	-109	139

Table 20.1.3-14 Shear Demand Envelope

Location						Case 1	Case 2	Case 1		Case 2		Envelope		
			V_{DL}	V_{ADL}	$V_{P/S}$	V_{EQ}	V_{EQ}	$V_{positiv}$	$V_{negative}$	$V_{positive}$	$V_{negative}$	$V_{positive}$	$V_{negative}$	V_{max}
	x/L	x (ft)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)
Span 1	Support	1.5	723	86	99	-129	81	779	890	989	890	989	890	989
	0.1	12.6	535	65	97	-129	81	568	680	778	680	778	680	778
	0.2	25.2	330	40	97	-129	81	338	450	547	450	547	450	547
	0.3	37.8	125	15	97	-129	81	108	220	318	220	318	220	318
	0.4	50.4	-80	-10	96	-129	81	-122	-9	87	-9	87	-9	87
	0.5	63	-285	-34	96	-129	81	-352	-239	-142	-239	-142	-239	239
	0.6	75.6	-490	-59	96	-129	81	-582	-468	-372	-468	-372	-468	468
	0.7	88.2	-695	-84	96	-129	81	-811	-698	-602	-698	-602	-698	698
	0.8	100.8	-900	-108	96	-129	81	-1041	-928	-832	-928	-832	-928	928
	0.9	113.4	-1108	-133	87	-129	81	-1283	-1161	-1074	-1161	-1074	-1161	1161
	Support	123	-1278	-152	85	-129	81	-1474	-1350	-1265	-1350	-1265	-1350	1350
Span 2	Support	129	1347	160	-10	-247	227	1250	1733	1724	1733	1724	1733	1733
	0.1	142.8	1106	133	-2	-247	227	990	1466	1464	1466	1464	1466	1466
	0.2	159.6	833	100	2	-247	227	688	1159	1162	1159	1162	1159	1162
	0.3	176.4	559	67	2	-247	227	382	853	855	853	855	853	855
	0.4	193.2	286	35	2	-247	227	76	547	549	547	549	547	549
	0.5	210	13	2	2	-247	227	-230	241	243	241	243	241	243
	0.6	226.8	-260	-31	2	-247	227	-536	-65	-62	-65	-62	-65	65
	0.7	243.6	-533	-64	2	-247	227	-842	-371	-368	-371	-368	-371	371
	0.8	260.4	-806	-97	2	-247	227	-1148	-676	-674	-676	-674	-676	676
	0.9	277.2	-1079	-130	-4	-247	227	-1461	-983	-987	-983	-987	-983	987
	Support	291	-1321	-157	-8	-247	227	-1734	-1251	-1260	-1251	-1260	-1251	1260
Span 3	Support	297	1212	144	-122	-109	139	1126	1496	1374	1496	1374	1496	1496
	0.1	305.8	1056	127	-106	-109	139	968	1322	1216	1322	1216	1322	1322
	0.2	317.6	860	104	-97	-109	139	758	1103	1006	1103	1006	1103	1103
	0.3	329.4	668	81	-97	-109	139	543	888	791	888	791	888	888
	0.4	341.2	476	57	-97	-109	139	328	673	576	673	576	673	673
	0.5	353	285	34	-99	-109	139	112	458	360	458	360	458	458
	0.6	364.8	93	11	-99	-109	139	-104	243	144	243	144	243	243
	0.7	376.6	-99	-12	-99	-109	139	-319	28	-71	28	-71	28	71
	0.8	388.4	-291	-35	-99	-109	139	-534	-187	-286	-187	-286	-187	286
	0.9	400.2	-484	-58	-99	-109	139	-750	-403	-502	-403	-502	-403	502
	Support	410.5	-659	-79	-99	-109	139	-945	-598	-697	-598	-697	-598	697

20.1.3.19 Step 17 – Perform Vertical Acceleration Analysis

Since the site PGA = 0.58g < 0.6g, vertical acceleration analysis is not required.

20.1.3.20 Step 18 – Calculate Superstructure Flexural and Shear Capacity

20.1.3.20.1 Superstructure Flexural Capacity

Table 20.1.3-15 lists the data that will be used to calculate the flexural section capacity using the computer program *CSiBridge* and the expected material properties as per *SCD* Tables 3.3.3-1 and Table 3.3.6-1. Symbols in Table 20.1.3-15 are shown in Figure 20.1.3-14. Appendix 20.1.3-16 lists the *CSiBridge* input for the superstructure section that lies just to the left face of Bent 2. The model is shown in Appendix 20.1.3-17(a). The *CSiBridge* selects output for negative capacity are shown in Appendix 20.1.3-17(b).

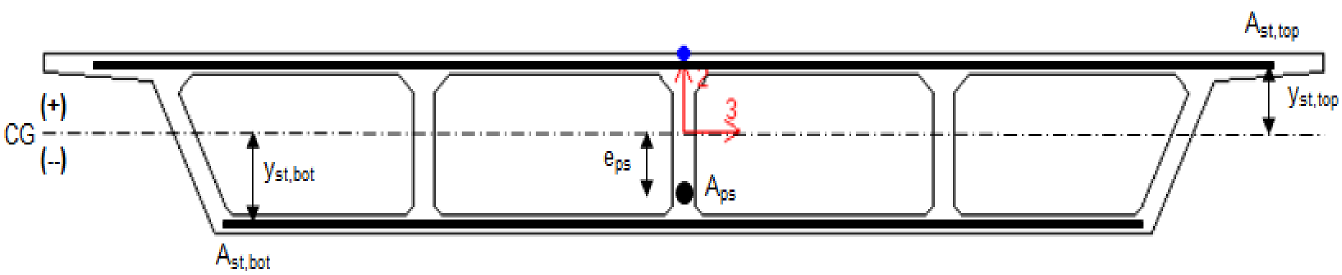


Figure 20.1.3-14 Typical Superstructure Cross Section

Section-Designer are used for every tenth point along the span to determine superstructure flexural capacities. Table 20.1.3-16 lists these capacities and also compares them with the maximum moment demands.

As can be seen from these results, the superstructure has sufficient flexural capacity to meet the anticipated seismic demands. Note that the flexural capacity determined from Section-Designer at the supports is multiplied by "4/5", due to the girder aligned along the centerline is the outside of the effective section width explained in Step 15.

SDC 5.4.2 specifies that the flexural capacity of the bent cap CPMs are taken as the nominal moment capacity based on expected material properties, M_{ne}, multiplied by the resistance factor of 1.0. M_{ne}, shall be determined by either M-ϕ analysis as specified in *SDC* Section 5.3.6.2, or nominal flexural resistance (at Strength limit state) as specified in AASHTO-CA BDS-8 using expected material properties. In this example, M_{ne} is determined using M-ϕ based on when either the concrete strain reaches 0.003 or the reinforcing steel strain reaches ε_{su}^{R} as derived from the steel stress strain model.

Table 20.1.3-15 Section Flexural Capacity Calculation Data

Location			No. Girders	No. Girders in Effective Section	Eccentricity e_{ps} (in.)	PS Force After All Losses (kips)	For Effective Section		Area of Top Mild Steel* $A_{st,top}$ (in.2)	Distance to Top Mild Steel $y_{st,top}$ (in.)	Area of Bottom Mild Steel* $A_{st,bot}$ (in.2)	Distance to Top Mild Steel $y_{st,bot}$ (in.)
	x/L	x (ft)					PS Force After All Losses (kips)	Area of PS A_{ps} (in.2)				
Span 1	Support	1.5	5	4	-1.54	7676	6141	38.28	8	31.8	6	-42.13
	0.1	12.6	5	5	-14.36	7748	7748	47.85	8	31.8	6	-42.13
	0.2	25.2	5	5	-24.84	7825	7825	47.85	8	31.8	6	-42.13
	0.3	37.8	5	5	-31.08	7895	7895	47.85	8	31.8	6	-42.13
	0.4	50.4	5	5	-33.16	7958	7958	47.85	8	31.8	6	-42.13
	0.5	63	5	5	-31.69	8006	8006	47.85	47.4	31.8	34.76	-42.13
	0.6	75.6	5	5	-27.29	8050	8050	47.85	47.4	31.8	34.76	-42.13
	0.7	88.2	5	5	-19.96	8091	8091	47.85	47.4	31.8	34.76	-42.13
	0.8	100.8	5	5	-9.69	8118	8118	47.85	47.4	31.8	34.76	-42.13
	0.9	113.4	5	5	5.39	8055	8055	47.85	47.4	31.8	34.76	-42.13
	Support	123	5	4	14.16	7978	6382	38.28	47.4	31.8	34.76	-42.13
Span 2	Support	129	5	4	14.29	7926	6341	38.28	47.4	31.8	34.76	-42.13
	0.1	142.8	5	5	2.89	7777	7777	47.85	47.4	31.8	34.76	-42.13
	0.2	159.6	5	5	-13.36	7737	7737	47.85	47.4	31.8	34.76	-42.13
	0.3	176.4	5	5	-24.36	7690	7690	47.85	47.4	31.8	34.76	-42.13
	0.4	193.2	5	5	-30.96	7639	7639	47.85	8	31.8	6	-42.13
	0.5	210	5	5	-33.16	7585	7585	47.85	8	31.8	6	-42.13
	0.6	226.8	5	5	-30.87	7584	7584	47.85	8	31.8	6	-42.13
	0.7	243.6	5	5	-24.01	7634	7634	47.85	47.4	31.8	34.76	-42.13
	0.8	260.4	5	5	-12.57	7679	7679	47.85	47.4	31.8	34.76	-42.13
	0.9	277.2	5	5	4.29	7715	7715	47.85	47.4	31.8	34.76	-42.13
	Support	291	5	4	16.03	7880	6304	38.28	47.4	31.8	34.76	-42.13
Span 3	Support	297	5	4	15.73	7940	6352	38.28	47.4	31.8	34.76	-42.13
	0.1	305.8	5	5	5.51	8011	8011	47.85	47.4	31.8	34.76	-42.13
	0.2	317.6	5	5	-12.57	8121	8121	47.85	47.4	31.8	34.76	-42.13
	0.3	329.4	5	5	-24.01	8097	8097	47.85	47.4	31.8	34.76	-42.13
	0.4	341.2	5	5	-30.87	8044	8044	47.85	47.4	31.8	34.76	-42.13
	0.5	353	5	5	-33.16	7982	7982	47.85	47.4	31.8	34.76	-42.13
	0.6	364.8	5	5	-31.83	7934	7934	47.85	8	31.8	6	-42.13
	0.7	376.6	5	5	-27.84	7882	7882	47.85	8	31.8	6	-42.13
	0.8	388.4	5	5	-21.19	7827	7827	47.85	8	31.8	6	-42.13
	0.9	400.2	5	5	-11.76	7769	7769	47.85	8	31.8	6	-42.13
	Support	410.5	5	4	-1.26	7717	6174	38.28	8	31.8	6	-42.13

Pjack = 9689 kips

* Area of mild steel based on minimum seismic requirement only

(Remaining limit state requirements need to be satisfied, Ast, top = 56.6 in.^{2} at right face of Bent 2)

Table 20.1.3-16 Section Flexural Capacity Calculation Data

Location			Moment Demand		Moment Capacity		*D*/*C* Ratio	
	x/L	*x*(ft)	$M_{positive}$ kip-ft	$M_{negative}$ kip-ft	$M_{positive}$ kip-ft	$M_{negative}$ kip-ft	Positive Moment	Negative Moment
Span 1	Support	1.5	1490	1043	27210	-32524	0.05	-0.03
	0.1	12.6	11274	7435	54687	-35101	0.21	-0.21
	0.2	25.2	19593	11918	65609	-23790	0.30	-0.50
	0.3	37.8	25012	13502	72360	-16696	0.35	0.00
	0.4	50.4	27536	12192	74396	-14686	0.37	0.00
	0.5	63	27164	7988	87925	-36693	0.31	0.00
	0.6	75.6	23897	890	83413	-41659	0.29	0.00
	0.7	88.2	17737	-9102	76048	-49618	0.23	0.00
	0.8	100.8	8680	-21987	65780	-60156	0.13	0.37
	0.9	113.4	-3284	-37786	30274	-76523	-0.11	0.49
	Support	123	-14714	-51848	29461	-59810	-0.50	0.87
Span 2	Support	129	-5960	-55107	29440	-59902	-0.20	0.92
	0.1	142.8	9500	-33026	52179	-74123	0.18	0.45
	0.2	159.6	23098	-10971	69537	-56535	0.33	0.19
	0.3	176.4	31550	5940	80585	-44800	0.39	0.00
	0.4	193.2	34860	17708	72078	-17416	0.48	0.00
	0.5	210	35687	21943	74405	-14947	0.48	0.00
	0.6	226.8	37170	15914	71987	-17505	0.52	0.00
	0.7	243.6	33516	4742	80229	-45177	0.42	0.00
	0.8	260.4	24720	-11571	77325	-48277	0.32	0.24
	0.9	277.2	10782	-33027	50614	-75335	0.21	0.44
	Support	291	-4945	-54615	28238	-61020	-0.18	0.90
Span 3	Support	297	-10042	-49584	28446	-60823	-0.35	0.82
	0.1	305.8	-418	-37187	49216	-76651	-0.01	0.49
	0.2	317.6	9801	-22879	68708	-57365	0.14	0.40
	0.3	329.4	17467	-11131	80035	-45020	0.22	0.00
	0.4	341.2	22594	-1920	87081	-37588	0.26	0.00
	0.5	353	25182	4753	89429	-35011	0.28	0.00
	0.6	364.8	25233	8890	72990	-16488	0.35	0.00
	0.7	376.6	22748	10489	68773	-20936	0.33	0.00
	0.8	388.4	17724	9552	66024	-23750	0.27	-0.40
	0.9	400.2	10159	6072	51944	-38330	0.20	-0.16
	Support	410.5	1428	918	27023	-33081	0.05	-0.03

20.1.3.20.2 Superstructure Shear Capacity

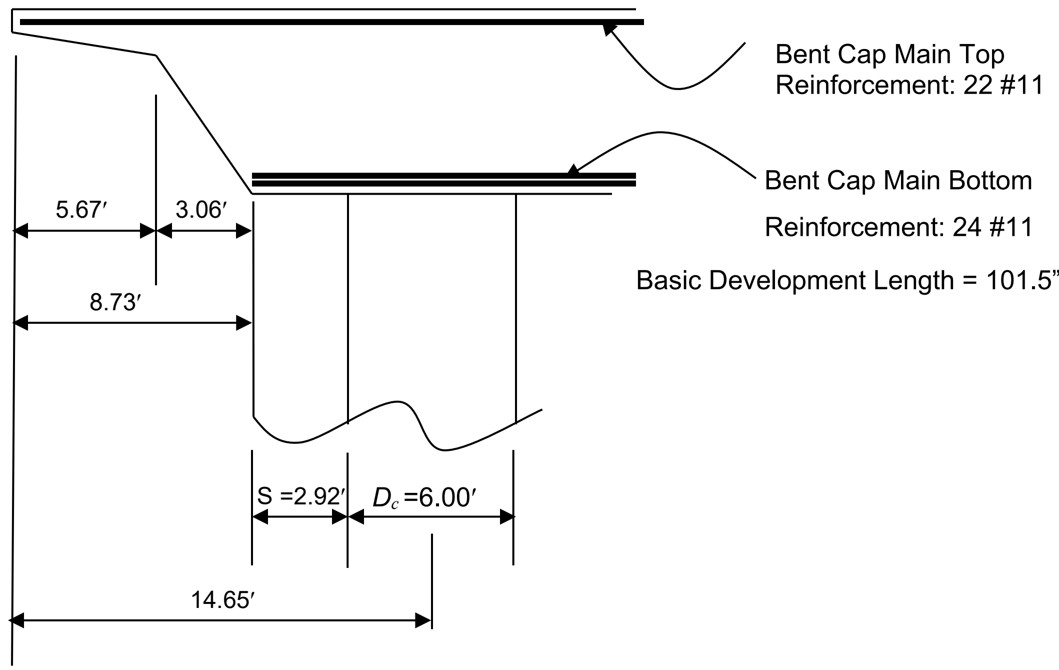
As shown in Table 20.1.3-17, seismic shear demands do not control as they are less than the demands from the controlling limit state (i.e. Strength I, Strength II, etc.) calculated using *CTBridge*. Therefore, the superstructure has sufficient shear capacity to resist seismic demands.

Table 20.1.3-17 Shear Demand vs. Capacity

	Location		Shear Demand	Shear Capacity (Strength Limit State)	*D/C Ratio*
			V_{EQ} (Kips)	ϕV_n (Kips)	D/C
Span 1	Support	1.5	989	2933	0.34
	0.1	12.6	778	2295	0.34
	0.2	25.2	547	1680	0.33
	0.3	37.8	318	1105	0.29
	0.4	50.4	87	635	0.14
	0.5	63	239	842	0.28
	0.6	75.6	468	1399	0.33
	0.7	88.2	698	2281	0.31
	0.8	100.8	928	2861	0.32
	0.9	113.4	1161	3519	0.33
	Support	123	1350	4050	0.33
Span 2	Support	3	1733	4372	0.40
	0.1	16.8	1466	3663	0.40
	0.2	33.6	1162	2866	0.41
	0.3	50.4	855	2083	0.41
	0.4	67.2	549	1345	0.41
	0.5	84	243	675	0.36
	0.6	100.8	65	1043	0.06
	0.7	117.6	371	1839	0.20
	0.8	134.4	676	2889	0.23
	0.9	151.2	987	3695	0.27
	Support	165	1260	4374	0.29
Span 3	Support	3	1496	3061	0.49
	0.1	11.8	1322	3283	0.40
	0.2	23.6	1103	2696	0.41
	0.3	35.4	888	2188	0.41
	0.4	47.2	673	1650	0.41
	0.5	59	458	1120	0.41
	0.6	70.8	243	621	0.39
	0.7	82.6	71	976	0.07
	0.8	94.4	286	1446	0.20
	0.9	106.2	502	2209	0.23
	Support	116.5	697	2762	0.25

20.1.3.21 Step 19 – Design Joint Shear Reinforcement

Figure 20.1.3-15 shows the bent cap-to-column joint.



Dimensions along skew direction

Figure 20.1.3-15 Bent Cap-to-Column Joint

Cap beam short stub length, S = 14.65 – 8.73 – 3.0 = 2.92 ft < D_c = 6 ft (*SDC* 7.4.4.2-1). Therefore, the joint will be designed as a knee joint in the transverse direction and a T joint in the longitudinal direction.

20.3.21.1 Design Transverse Direction (Knee Joint Design)

$$S = 2.92\text{ft} < \frac{6.00}{2} = 3.0\,\text{ft}$$

Therefore, the joint is classified as Case 1 Knee joint. (*SDC* 7.4.4.2-2)

(1) Closing Failure Mode - Bent 2 Knee Joint

Given: Superstructure depth, D_s = 6.75 ft

Column diameter, D_c = 6 ft, Concrete cover = 2 in.

Column reinforcement:

Main reinforcement anchored into cap beam: #14 bars, total 26 giving A_{st} = 58.50 in.^{2}

Transverse reinforcement: #7 hoops spaced at 6 in. c/c.

Column main reinforcement embedment length into the bent cap, $l_{ac,provided} = 66$ in.

From the Section-Designer analysis of Bent 2 with overturning effects (see Appendix 20.1.3-4):

Column plastic moment, M_p = 14,418 kip-ft

Column axial force (including the effect of overturning), P_c = 2, 638 kips

Cap Beam main reinforcement: top: #11 bars, total 22 and bottom: #11 bars, total 24.

Calculate principal stresses, p_t and p_c

Vertical Shear Stress, v_{jv}

$$T_c = M_o^{col}/h = 1.2 \times M_p/h = 1.2 \times 14,418/5.73 = 3,020 \text{ kips} \qquad (\textit{SDC} \text{ Section 7.4.2})$$

$$A_{jv} = l_{ac,provided}\left(B_{cap}\right) = (66)(96) = 6,336 \text{ in.}^2 \qquad (\textit{SDC} \text{ 7.4.2-9})$$

$$v_{jv} = \frac{T_c}{A_{jv}} = \frac{3,020}{6336} = 0.477 \text{ ksi} \qquad (\textit{SDC} \text{ 7.4.2-7})$$

Normal Stress (Vertical), f_v

$$f_v = \frac{P_c}{A_{jh}} = \frac{P_c}{(D_c + D_s)B_{cap}} = \frac{2,638}{(6.00 + 6.75)(8.00)(144)} = 0.18 \text{ ksi} \qquad (\textit{SDC} \text{ 7.4.2-6})$$

Normal Stress (Horizontal)

Assume P_b = 0 since no prestressing is specifically designed to provide horizontal joint compression. Therefore, horizontal normal stress, $f_h = P_b/B_{cap} \times D_s = 0$.

$$p_t = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} = \frac{(0.00 + 0.180)}{2} - \sqrt{\left(\frac{0.00 - 0.180}{2}\right)^2 + 0.477^2}$$

$$p_t = -0.395 \text{ ksi (- for joint in tension)} \qquad (\textit{SDC} \text{ 7.4.2 -3})$$

$$p_c = \frac{(0.00 + 0.18)}{2} + \sqrt{\left(\frac{0.00 - 0.18}{2}\right)^2 + 0.46^2} = 0.575 \text{ ksi}$$ (+ for joint in compression) (*SDC* 7.4.2-4)

Check Joint Size Adequacy

Principal compression, $p_c = 0.575 \text{ ksi} \le \left[0.25f'_c = 0.25 \times 4.0 = 0.1 \text{ ksi}\right]$ OK

(*SDC* 7.4.2-2)

Principal tension, $p_t = 0.395 \text{ ksi} \le \left[12\sqrt{f'_c} = 12 \times \sqrt{4000}/1000 = 0.76 \text{ ksi}\right]$ OK

(*SDC* 7.4.2-1)

Check the Need for Additional Joint Reinforcement

Since $p_t = 0.395 \text{ ksi} > \left[3.5\sqrt{f'_c} = 3.5 \times \sqrt{4000}/1000 = 0.221 \text{ ksi}\right]$, additional joint reinforcement is required (see *SDC* Section 7.4.5.1).

Similar calculations can be performed for Bent 3.

(2) Opening Failure Mode - Bent 2 Knee Joint

From *Bent 2* push-over analysis results (see Appendix 20.1.3-3.II),

Column axial force (including the effect of overturning), P_c = 820 kips

Column plastic moment, M_p = 11,911 kip-ft*

* These values were obtained from *Section Designer* analysis of Bent 2 with overturning effects (see Appendix 20.1.3-5)

T_c = 1.2 (2,079) kips = 2,495 kips.

A_{jv} = 66 (96) = 6,336 in.^{2}

$$v_{jv} = \frac{2,495}{6,336} = 0.394 \text{ ksi}$$

$$f_v = \frac{P_c}{A_{jh}} = \frac{P_c}{(D_c + D_s) \times B_{cap}} = \frac{820}{(6.00 + 6.75) \times 8.00 \times 144} = 0.056 \text{ ksi}$$

f_h = 0 (since P_b = 0)

$$p_t = \frac{(0.00 + 0.056)}{2} - \sqrt{\left(\frac{0.00 - 0.056}{2}\right)^2 + 0.394^2} = -0.367 \text{ ksi}$$

$$p_c = \frac{(0.00 + 0.056)}{2} + \sqrt{\left(\frac{0.00 - 0.056}{2}\right)^2 + 0.394^2} = 0.423 \text{ ksi}$$

Check Joint Size Adequacy

Principal compression, $p_c = 0.423 \text{ ksi} < \left[0.25 \times 4.0 = 1.0 \text{ ksi}\right]$ OK

Principal tension, $[\, p_t = 0.367 \text{ ksi}\,] < [\, \frac{12\sqrt{4000}}{1000} = 0.760 \text{ ksi}\,]$ OK

Check the Need for Additional Joint Reinforcement

Since p_t = 0.367 ksi > [$3.5\sqrt{4000}/1000 =$ 0.221 ksi], additional joint reinforcement is required.

Based upon the joint stress condition evaluation for both closing and opening modes of failure, the joint needs additional joint reinforcement. Refer to Figure 20.1.3-16 for regions of the additional joint shear reinforcement.

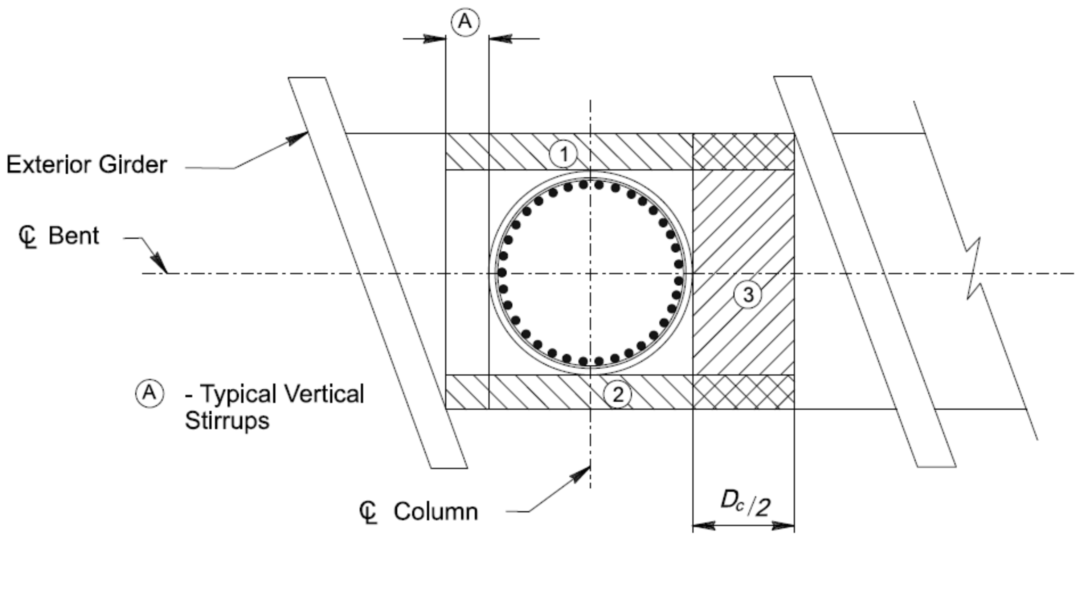


Figure 20.1.3-16 Regions of Additional Joint Shear Reinforcement

Check Joint Shear Requirement

Bent Cap Top and Bottom Flexural Reinforcement, A_s^{U-Bar} (Refer to Figure 20.1.3-17)

$A_s^{U-Bar}{}_{required}$ ≥ 0.33 A_{st} = 0.33 (58.5) = 19.3 in.^{2} (*SDC* 7.4.5.1-3)

The bent cap reinforcement based upon service and seismic loading consists of:

Top Reinforcement #11, total 22 bars giving A_{st} = 34.32 in.^{2}

Bottom Reinforcement #11, total 24 bars giving A_{st} = 37.44 in.^{2}

$A_s^{U-Bar}{}_{provided}$ = 12 (1.56) = 18.72 in.^{2} (within 4 % of 19.3 in.^{2}) Say OK

See Figure 20.1.3-17 for the rebar layout.

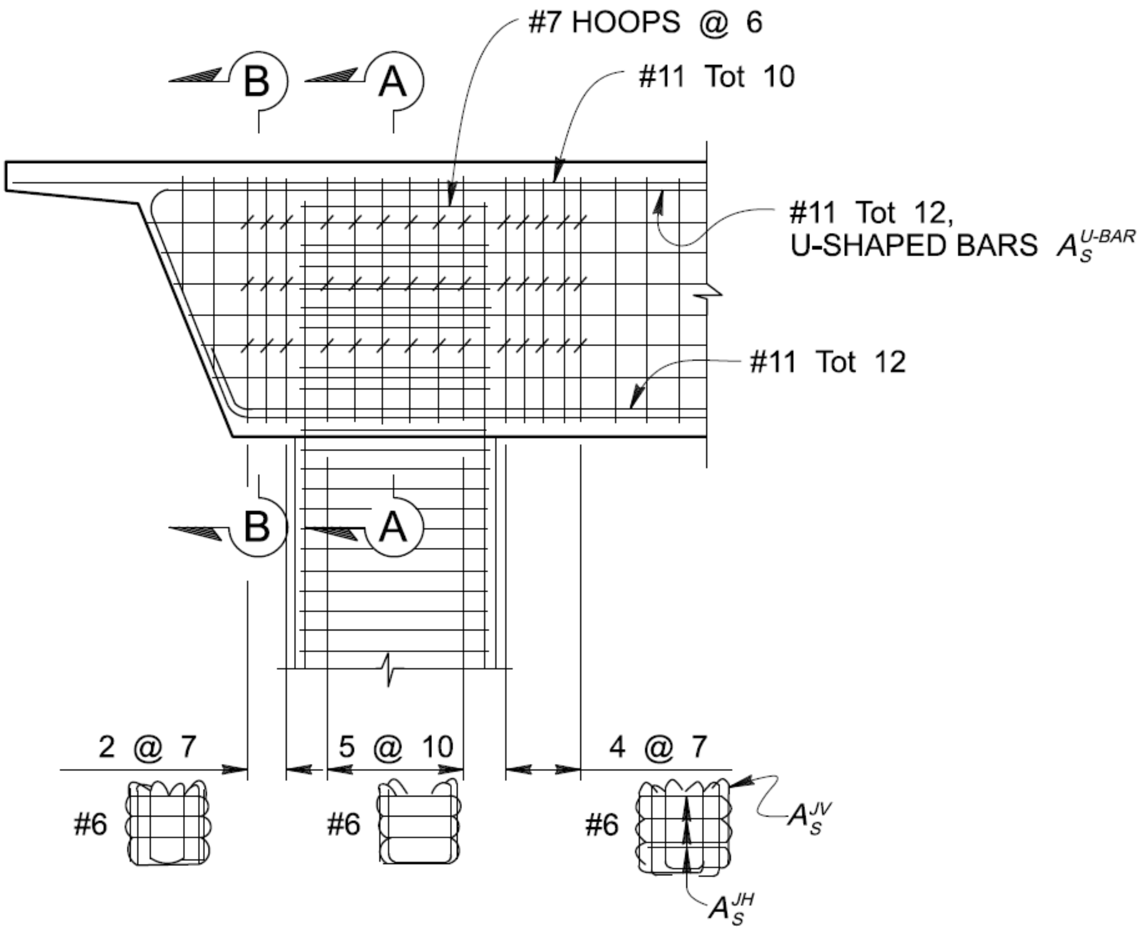


Figure 20.1.3-17 Location of Joint Shear Reinforcement (Elevation View)

Vertical Stirrups in Joint Region:

$A_{s\ required}^{jv} \geq 0.2A_{st} \geq 0.20\ (58.5) = 11.7\ \text{in.}^2$ (*SDC* 7.4.5.1-2)

Provide 5 sets of 6-legged, #6 stirrups so that

$A_{s\ provided}^{jv} = (6\ \text{legs})(5\ \text{sets})(0.44) = 13.2\ \text{in.}^2 > 11.7\ \text{in.}^2$ OK

Place stirrups transversely within a distance D_c = 72 inches extending from either side of the column centerline. These vertical stirrups are shown in Figure 20.1.3-17 and also in Figure 20.1.3-18.

Horizontal Stirrups in Joint Region:

$A_{s\ required}^{jh} \geq 0.1\ A_{st} = 0.1\ (58.5) = 5.85\ \text{in.}^2$ (*SDC* 7.4.5.3-3)

As shown in Figure 20.1.3-18, provide 3-legged #6 stirrups, total 14 sets

$A_{s\ provided}^{jh} = (3\ legs)(14\ sets)(0.44) = 18.48\ \text{in.}^2 > 5.85\ \text{in.}^2$

Placed within a distance D_c = 72 in. extending from either side of the column centerline as shown in Figure 20.1.3-18.

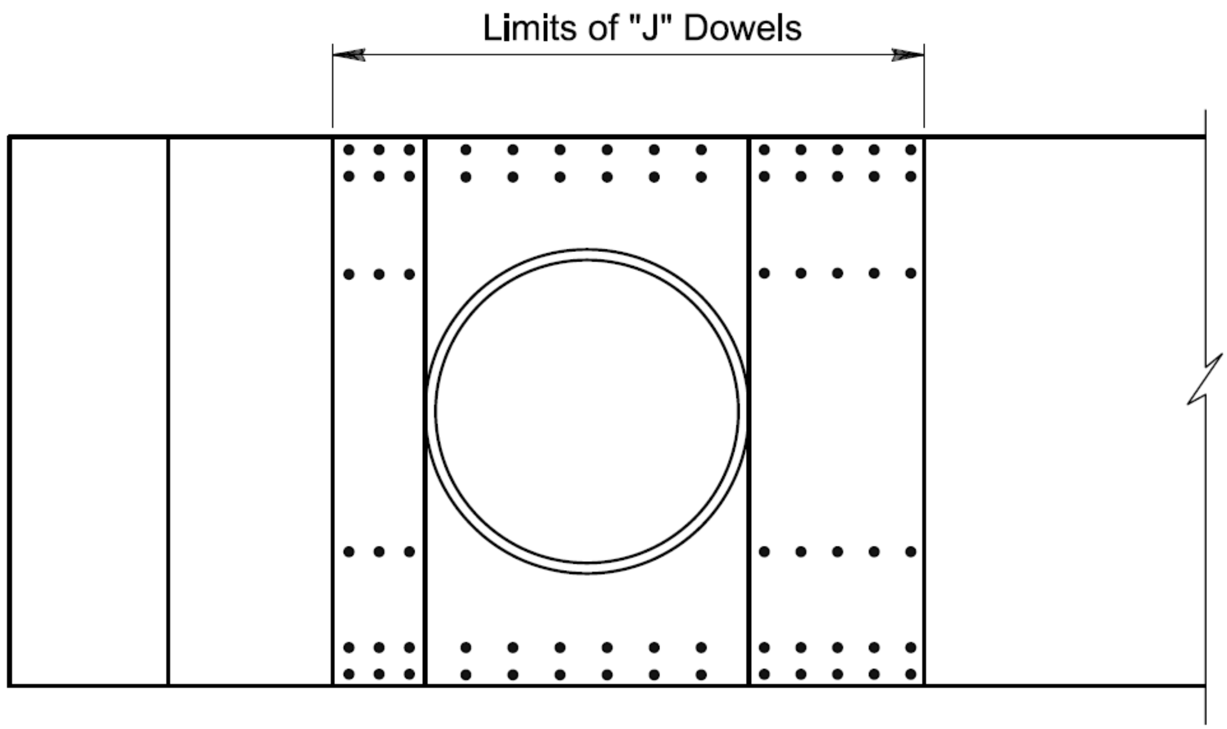


Figure 20.1.3-18 Location of Vertical Stirrups, A_s^{jv}

Horizontal Side Reinforcement:

$$A_s^{sf} \geq max\begin{cases} 0.1\times A_{cap}^{top} \\ or \\ 0.1\times A_{cap}^{bot} \end{cases} \qquad (SDC\ 7.4.5.3\text{-}4)$$

A_{cap}^{top} = 34.32 in.^{2}

A_{cap}^{bot} = 37.44 in.^{2}

$$A_s^{sf} \geq max\begin{cases} 0.1(34.32) = 3.43 \text{ in.}^2 \\ \text{or} \\ 0.1(37.44) = 3.74 \text{ in.}^2 \end{cases} \qquad \text{take } A_s^{sf} = 3.74 \text{ in.}^2$$

As shown in Figures 20.1.3-19 and 20.1.3-20, provide #6, total 5 (“U” shaped), giving:

$A_{s\ provided}^{sf}$ = (2 legs)(5 bars) (0.44)= 4.4 in.^{2} > 3.74 in.^{2}

Horizontal Cap End Ties:

$$A_s^{jhc} \geq 0.33 A_s^{u-bar} = 0.33(19.3) = 6.37\text{in.}^2 \qquad (SDC\ 7.4.5.3\text{-}5)$$

Provide #8, total 10 ($A^{jhc}_{s,provided} = 10(0.79) = 7.9\text{in.}^2 > 6.37\text{in.}^2$) OK

See *SDC* Figures 7.4.5.1-2, 7.4.5.1-3, and 7.4.5.1-5 for placement of A_s^{jhc}

J-Dowels

Strictly following *SDC*, there is no need for J-Dowels for this bridge. Let's provide it for educational purpose for this bridge.

$A_s^{j-bar} \geq 0.08$, A_{st} = 0.08 (58.5) = 4.68 in.^{2} (*SDC* 7.4.5.3-6)

Use 16, #5 J-Dowels.

$A_s^{j-bar}{}_{provided}$ = (16 bars)(0.31) = 4.96 in.^{2} > 4.68 in.^{2}

These dowels are placed within the rectangle defined by D_c on either side of the column centerline and the cap width. They are shown in Figures 20.1.3-19 and 20.1.3-20.

Check Transverse Reinforcement

The minimum reinforcement ratio of transverse reinforcement (hoops)

$$\rho_{s,required} = 0.76\left(\frac{A_{st}}{D_c l_{ac,provided}}\right) = 0.76\left(\frac{58.5}{72(66)}\right) = 0.00936$$ (*SDC* 7.4.5.3-7)

Column transverse reinforcement that extends into the joint region consists of #7 hoops at 6 in. spacing.

$$\rho_{s,\ provided} = \frac{7A_b}{D's} = \frac{7(0.60)}{\left(72 - 2(2) - 2\left[\frac{0.875}{2}\right]\right)(6)} = 0.0104 > 0.00936 \quad \text{OK}$$

Check Anchorage for Main Column Reinforcement

$l_{ac,\ required} = 24d_{bl}$ = 24 (1.69) = 40.6 in. < [$l_{ac,\ provided}$ = 66 in.] OK (*SDC* 8.3.1.1-1)

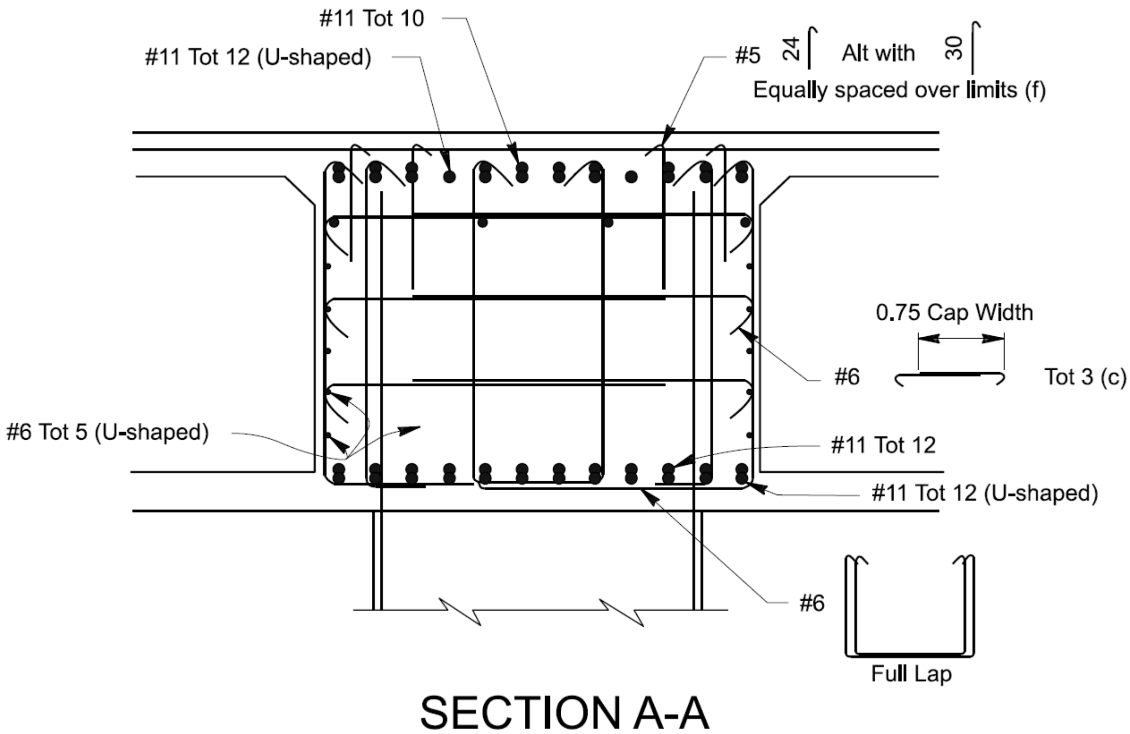


Figure 20.1.3-19 Joint Reinforcement Within the Column Region

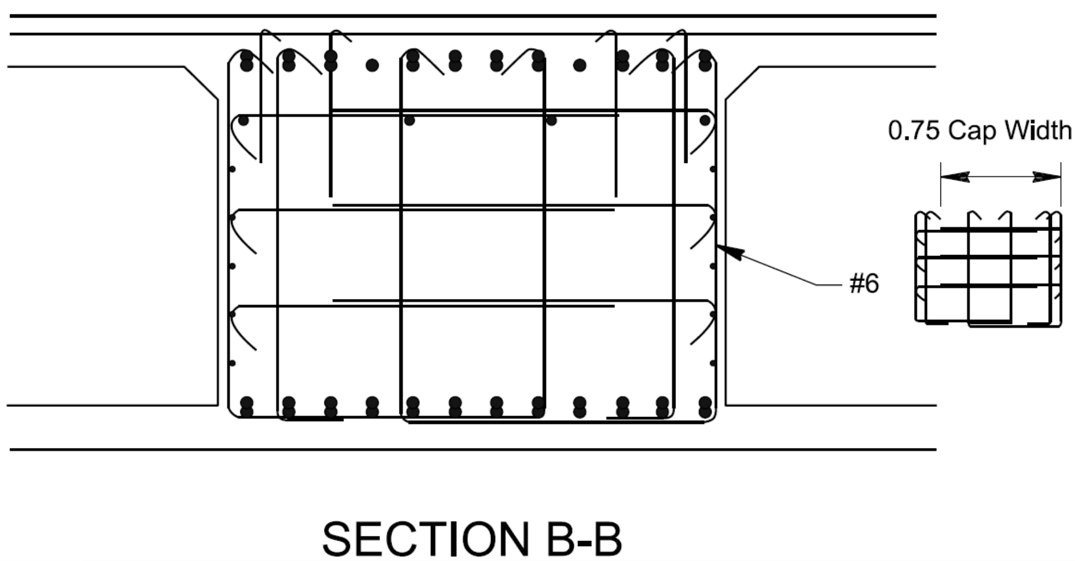


Figure 20.1.3-20 Joint Reinforcement Outside the Column Region

20.1.3.21.2 Design Longitudinal Direction (T-Joint)

Let us calculate joint stresses for the back side column, which will provide the higher value of principal tensile stress (generally more critical than the principal compressive stress).

Column plastic moment, M_p = 13,232 kip-ft*

Column axial force, P_c = 1,729 kips*

Cap beam main reinforcement: top: #11 bars, total 22 and bottom: #11 bars, total 24.

* These values were obtained from the *Section-Designer* analysis of Bent 2 columns without overturning effects (see Appendix 20.1.3-2).

Calculate Principal Stresses, p_t and p_c

$$T_c = 1.2(2310) = 2{,}772\,kips$$ using *Section-Designer* results

$$A_{jv} = l_{ac} \times B_{cap} = 66(96) = 6{,}336\,in^2$$

Vertical Shear Stress

$$v_{jv} = \frac{T_c}{A_{jv}} = \frac{2{,}772}{6{,}336} = 0.438\text{ ksi}$$

Normal Stress (Vertical)

$$f_v = \frac{P_c}{A_{jh}} = \frac{P_c}{(D_c + D_s)B_{cap}} = \frac{1{,}729}{(6.00 + 6.75)(8.00)(144)} = 0.118\text{ ksi}$$

Assume P_b = 0 since no prestressing is specifically designed to provide horizontal joint compression. Therefore, horizontal normal stress, $f_h = P_b / B_{cap} \times D_s = 0$.

$$p_t = \frac{(0.00 + 0.118)}{2} - \sqrt{\left(\frac{0.00 - 0.118}{2}\right)^2 + 0.438^2} = -0.383\text{ ksi} \quad \text{(i.e., tension)}$$

$$p_c = \frac{(0.00 + 0.118)}{2} + \sqrt{\left(\frac{0.00 - 0.115}{2}\right)^2 + 0.424^2} = 0.501\text{ ksi} \quad \text{(i.e., compression)}$$

Check Joint Size Adequacy

Principal compression, p_c = 0.501 ksi ≤ [0.25 (4.0) = 1 ksi] OK

Principal tension, p_t = 0.383 ksi ≤ [$12\sqrt{4000}/1000$ = 0.760 ksi] OK

Check the Need for Additional Joint Reinforcement

p_t = 0.383 ksi > [$3.5\sqrt{4000}/1000$ = 0.221 ksi], therefore additional joint reinforcement is required.

The horizontal stirrups, cap beam u-bar requirements, continuous cap side face reinforcement, J-dowels, minimum transverse reinforcement, and column reinforcement anchorage provided for transverse bending will also satisfy the joint shear requirements for longitudinal bending. The only additional joint reinforcement requirement that needs to be satisfied for longitudinal bending is to provide vertical stirrups in Regions 1 and 2 of Figure 20.1.3-16.

Vertical Stirrups in Joint Region – Regions 1 and 2 of Figure 20.1.3-16

$$A_{s\ required}^{jv} \geq 0.2\ A_{st} = 0.2\ (58.5) = 11.7\ \text{in.}^2$$

Provide: total 14 sets of 2-legged #6 stirrups or ties on each side of the column.

$$A_{s\ provided}^{jv} = (2\ \text{legs})(14\ \text{sets})(0.44) = 12.32\ \text{in.}^2 > 11.7\ \text{in.}^2 \qquad \text{OK}$$

As shown in Figures 20.1.3-17 and 20.1.3-18, these vertical stirrups and ties are placed transversely within a distance D_c extending from either side of the column centerline.

Note that in the overlapping portions of regions 1 and 2 with region 3, the outside two legs of the 6-legged vertical stirrups provided for transverse bending are also counted toward the two legs of the vertical stirrups required for the longitudinal bending.

20.1.3.22 Step 20 – Check Minimum Hinge Seat Width

This bridge is not a multi-frame bridge. Therefore, this step does not apply.

20.1.3.23 Step 21 – Check Minimum Abutment Seat Width

The abutment seat width shall satisfy *SDC* 6.3.3.

$$N_A \geq \begin{cases} MR + \Delta_{eq} + L \\ \dfrac{D_s}{3} \\ 30\ \text{in.} \end{cases} \qquad (\textit{SDC}\ 6.3.3\text{-}1)$$

The combined effect of $\Delta_{p/s}$, Δ_{cr+sh}, and Δ_{temp}, is calculated as 2.6 inches (see Joint Movement Calculation form - Appendix 20.1.3-9).

The maximum seismic displacement demand along the longitudinal direction of the bridge is calculated in a conservative way assuming that maximum longitudinal and transverse (along the bent line) demand displacements occur simultaneously, i.e.,

$$\Delta_{eq,longitudinal} = 12.81 + 10.61 \sin(20^\circ) = 16.44\ \text{in.}$$

$N_{A,\ required}$ (normal to centerline of bearing) = (16.44 + 2.6) cos (20°) + 4 = 21.89 in. < 30 in.

Provide abutment seat width N_A = 36 in. > 30 in. OK

20.1.3.24 Step 22 – Design Abutment Shear Key Reinforcement

Shear key force capacity, $F_{sk} = \alpha\left(0.75\,V_{piles} + V_{ww}\right)$ (*SDC* 6.3.4-1)

We shall assume the following information to be available from the abutment foundation and wingwall design:

14 piles for the abutment, and 40 k/pile as ultimate shear capacity of the pile based on Caltrans practice.

f_c' = 3.6 ksi

Wingwall thickness = 12 in.

Wingwall height to top of abutment footing = 14 ft (i.e., 6.75 ft Superstructure depth + 7.25 ft abutment stem height)

V_{piles} = 14 (40) = 560 kips

Using AASHTO Article 5.7.3.3, the shear capacity of one wingwall, V_{ww} may conservatively be estimated as follows:

Effective shear depth d_v = 0.72 (12 *in*) = 8.64 in.

Effective width $b_v = \left[6.75 + \frac{1}{3}(7.25)\right](12) = 110$ in.

$$V_{ww} = 0.0316\beta\sqrt{f_c'}\,b_v d_v = 0.0316(2)\sqrt{3.6}(110)(8.64) = 114 \text{ kips}$$

Assuming $\alpha = 0.75$, $F_{sk} = 0.75\left[0.75(560) + 114\right] = 401$ kips

We shall use the Isolated Shear Key Method for this example.

Vertical shear key reinforcement:

$$A_{sk}^{ISO} = \frac{F_{sk}}{1.8 f_{ye}} = \frac{401}{1.8(68)} = 3.28 \text{ in.}^2 \qquad (SDC\ 6.3.5.1\text{-}1)$$

Provide 8 #6 – bundle bars as shown in Figure *SDC* 6.3.5.1-1

($A_{sk,provided}$ = 3.52 in.^{2} > 3.28 in.^{2}) OK

Hanger bars,

$$A_{sh} = 2.0\, A_{sk(provided)}^{Iso} = 2\,(3.52) = 7.04 \text{ in.}^2 \qquad (SDC\ 6.3.5.1\text{-}2)$$

Provide 5 #11 hooked bars ($A_{sh,provided}$ = 7.8 in.^{2} > 7.04 in.^{2}) OK

Place the vertical shear key bars, A_{sk} at least L_{min} from the end of the lowest layer of the hanger bars, where:

$$L_{\min,hooked} = 0.6(a+b) + l_{dh} \qquad (SDC\ C6.3.5.1\text{-}1\text{-}3)$$

Assuming 3-inch-thick bearing pads and 12 in. vertical height of expansion joint filler (see *SDC* Figure 6.3.5-1),

a = (Bearing pad thickness + 6 in.) = 9 in.

Assuming 2 in cover and #4 distribution bars for the hanger bars,

b = 2 + 0.56 + 0.5 (1.63) = 3.4 in. (see *SDC* Figure 6.3.5-1 for definition of dimension "b")

$$l_{dh} = \frac{38 d_b}{\sqrt{f_c'}} = \frac{38(1.41)}{\sqrt{3.6}} = 28.2\text{in.}$$

$$L_{\min,hooked} = 0.6(a+b) + l_{dh} = 0.6(9+3.4) + 28.2 = 36\text{in.}$$

Place vertical shear key bars A_{sk} @ 40 in. from the hooked ends of the hanger bars A_{sh} as shown in Figure 20.1.3-21.

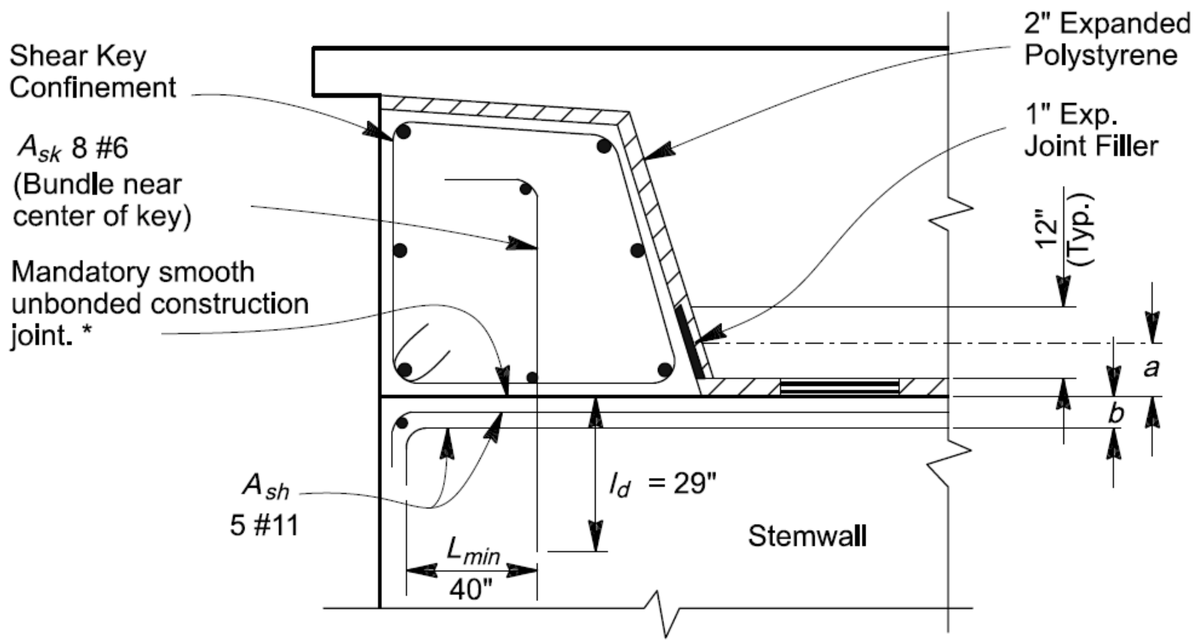


Figure 20.1.3-21 Isolated Shear key Reinforcement Details (modified *SDC* 2.0 Figure 6.3.5-1)

20.1.3.25 Step 23 – Check Requirements for No-splice Zone

SDC 8.2.2.1 specifies that the "No-Splice Zones" for SCMs shall correspond to the plastic hinge regions specified in Section 5.3.2, and "No-Splice Zones" shall be clearly identified on the plans. For this bridge, only columns have been designated as "seismic critical" elements.

The maximum length of column rebar can be estimated as

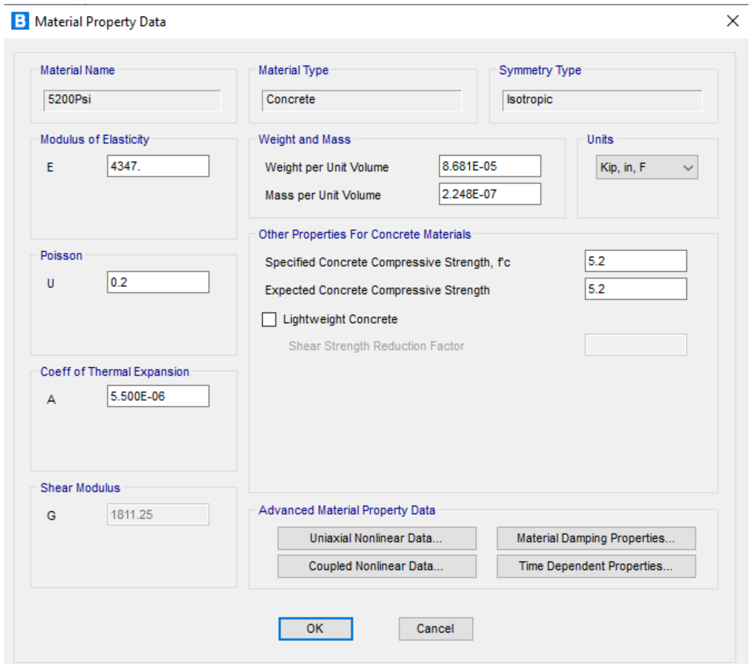
$$L_{max} = 47.00 + 5.5 = 52.5 \text{ ft} < 60.00 \text{ ft}$$

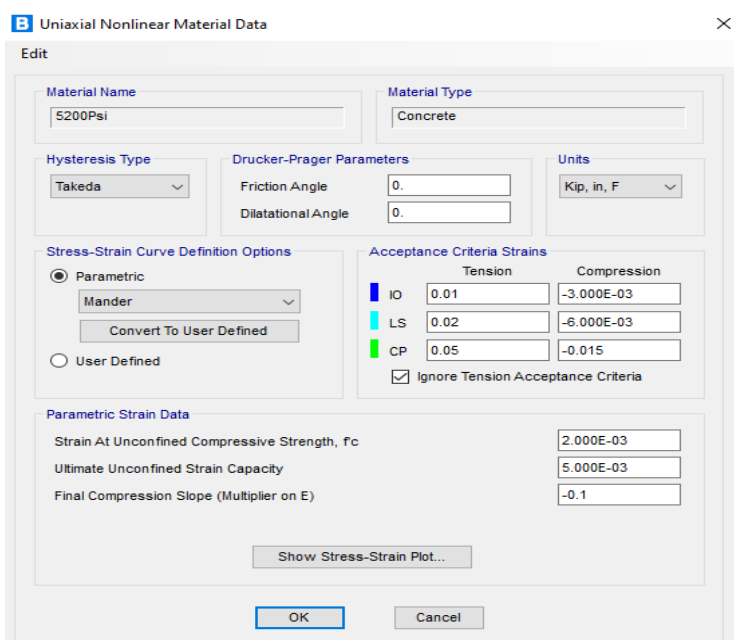
Therefore, we will specify on the plans that no splice is required for column main reinforcement. The superstructure rebars, however, can be spliced using the Service Splice.

APPENDICES

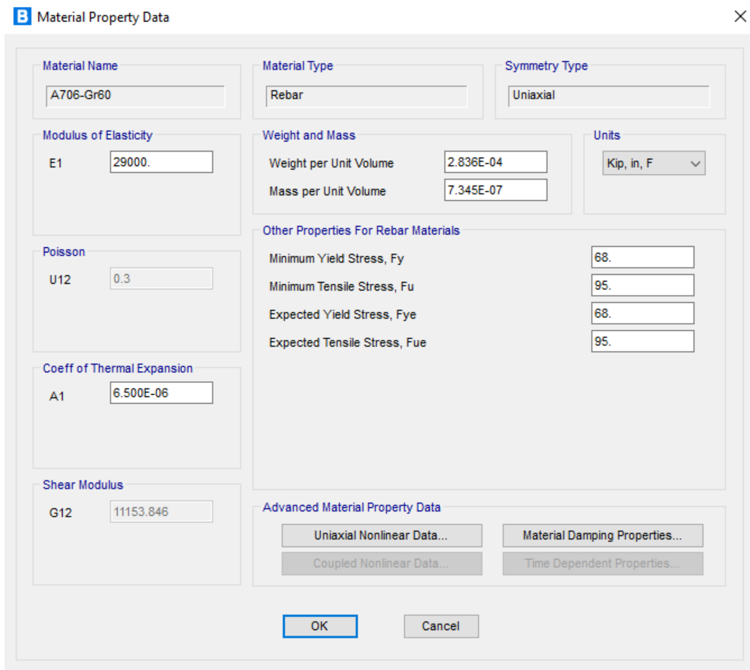
APPENDIX 20.1.3–1. Material Property Input to *CSiBridge*

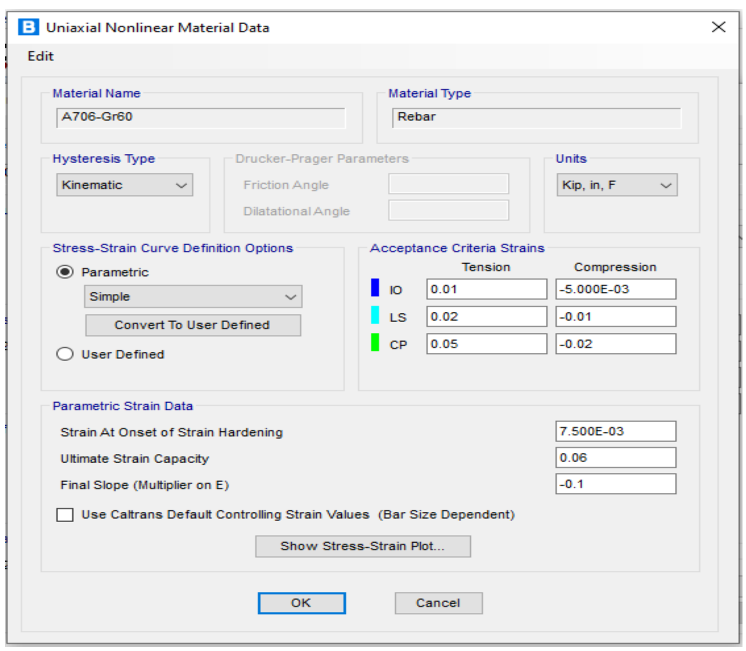
Concrete Type Information



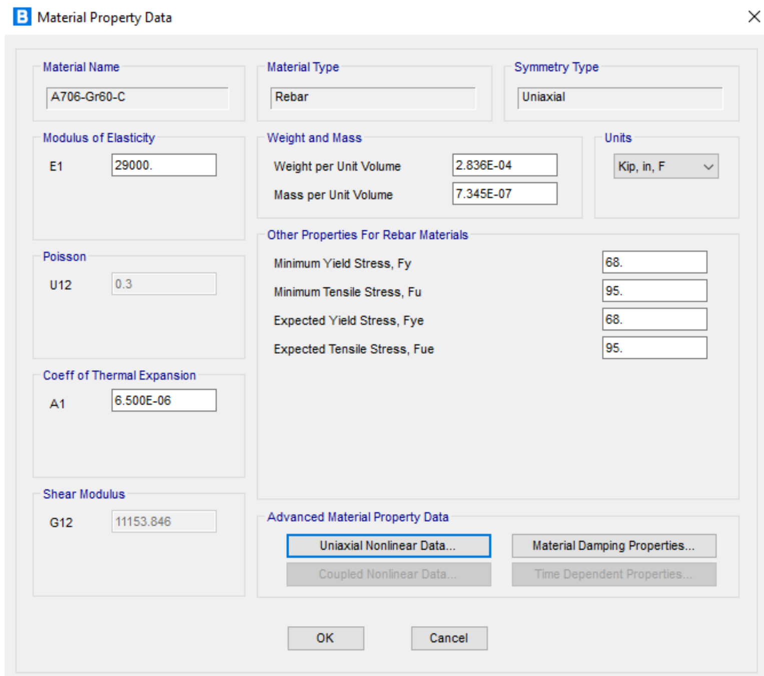


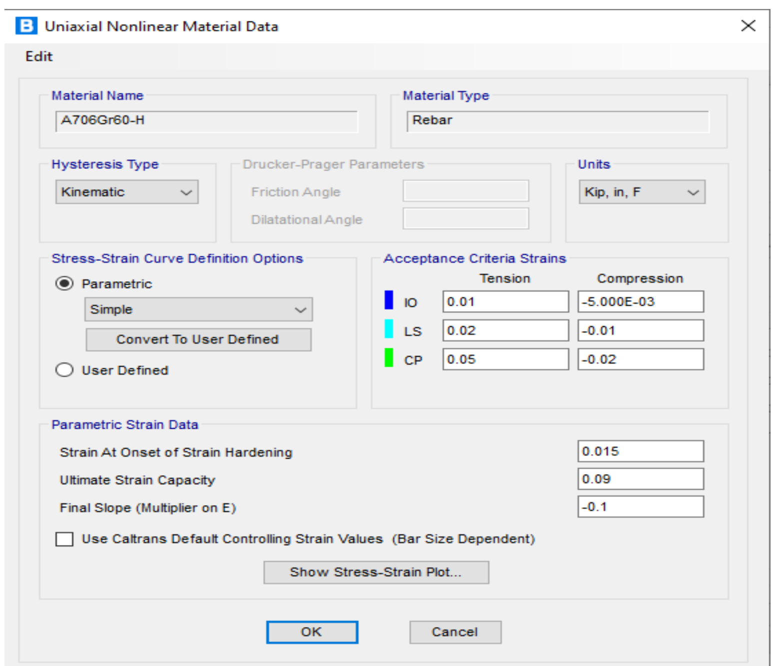
Steel Type Information- Main Bars





Steel Type Information – Hoop

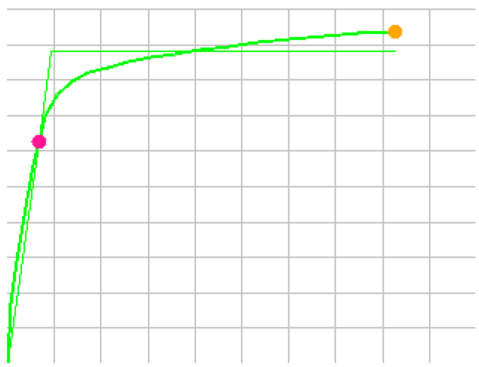




APPENDIX 20.1.3–2. *CSiBridge* Section-Designer Output for Bent 2 & 3 Columns

BENT 2: Section-Designer Output (no overturning effect)

MOMENT CURVATURE *(M-ϕ)* GRAPH - Kip, ft, F



Results For Exact-Integration

$\phi_{y(Initial)}$ = 6.631E-04
M_y = 9389.1683
$\phi_{y(Idealized)}$ = 9.345E-04
M_p = 13232
I_{crack} = 22.4914
$\phi_{concrete}$ = 8.259E-03
$M_{concrete}$ = 14076
ϕ_{steel} = N/A
M_{steel} = N/A

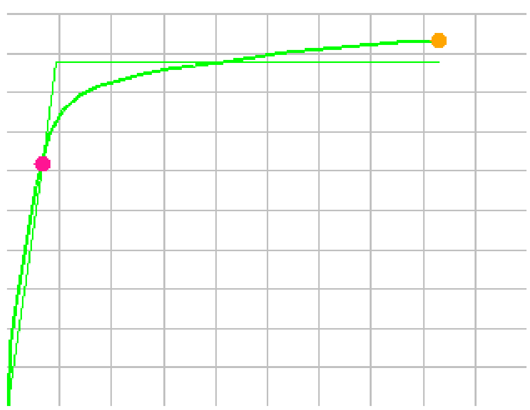
Conc. Strain	Neutral Axis	Steel Strain	Tendon Strain	Conc Comp	Steel Comp.	Steel Ten.	Prestress	Net Force	Curvature	Moment
-1.02E-04	0	-1.02E-04	0	-1556	-172.7	0	0	-1729	0	0
-3.11E-04	-1.0983	1.24E-04	0	-1582	-198.2	51	0	-1730	7.90E-05	2617
-5.40E-04	0.0995	5.47E-04	0	-1763	-270.2	304	0	-1730	1.98E-04	4382
-8.07E-04	0.5635	1.15E-03	0	-2069	-357.2	697	0	-1729	3.56E-04	6269
-1.13E-03	0.7856	1.91E-03	0	-2467	-469.6	1207	0	-1730	5.53E-04	8455
-1.50E-03	0.9313	2.85E-03	0	-2844	-595.8	1711	0	-1729	7.90E-04	10471
-1.87E-03	1.0817	4.00E-03	0	-3011	-699.6	1982	0	-1729	1.07E-03	11401
-2.25E-03	1.207	5.36E-03	0	-3091	-793.8	2156	0	-1729	1.38E-03	11945
-2.66E-03	1.3054	6.91E-03	0	-3131	-895.1	2295	0	-1731	1.74E-03	12317
-3.11E-03	1.3777	8.64E-03	0	-3164	-957.9	2393	0	-1729	2.13E-03	12538
-3.63E-03	1.4212	1.05E-02	0	-3233	-999.1	2502	0	-1729	2.57E-03	12782
-4.22E-03	1.4452	1.25E-02	0	-3287	-1052	2609	0	-1729	3.04E-03	12991
-4.93E-03	1.4486	0.0146	0	-3328	-1074	2671	0	-1730	3.56E-03	13079
-5.67E-03	1.4548	0.017	0	-3381	-1094	2746	0	-1729	4.11E-03	13252
-6.46E-03	1.4607	0.0194	0	-3434	-1116	2821	0	-1729	4.70E-03	13438
-7.30E-03	1.4656	0.0221	0	-3478	-1139	2888	0	-1729	5.34E-03	13587
-8.20E-03	1.4674	0.0249	0	-3520	-1166	2954	0	-1731	6.01E-03	13727
-9.17E-03	1.4678	0.0278	0	-3552	-1198	3020	0	-1729	6.72E-03	13858
-1.02E-02	1.4663	0.0309	0	-3578	-1233	3081	0	-1730	7.47E-03	13977
-1.14E-02	1.458	0.0341	0	-3619	-1243	3133	0	-1729	0.008259	14076

Note: units on the table - Kip, ft, F

BENT 3: Section-Designer Output (no overturning effect)

MOMENT CURVATURE *(M-ϕ)* GRAPH - Kip, ft, F

Axial Load = -1676
Moment Angle = 0



Results For Exact-Integration

$\phi_{y(Initial)}$ = 6.607E-04
M_y = 9296.6333
$\phi_{y(Idealized)}$ = 9.353E-04
M_p = 13161
I_{crack} = 22.3507
$\phi_{concrete}$ = 8.310E-03
$M_{concrete}$ = 14010
ϕ_{steel} = N/A
M_{steel} = N/A

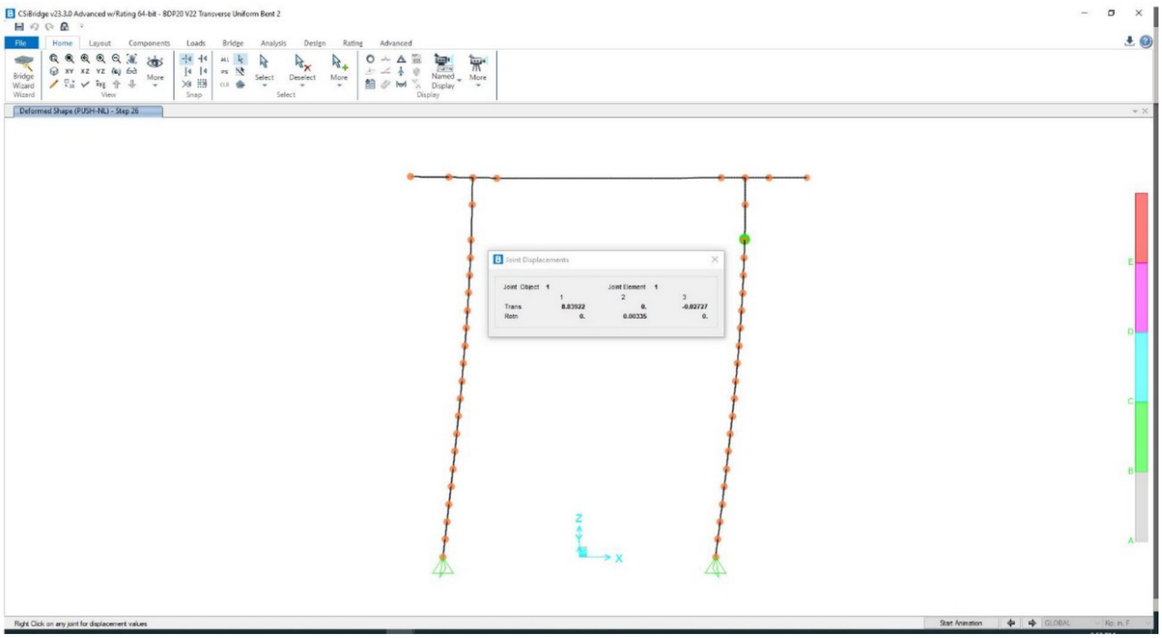
Conc. Strain	Neutral Axis	Steel Strain	Tendon Strain	Conc Comp	Steel Comp.	Steel Ten.	Prestress	Net Force	Curvature	Moment
-9.87E-05	0.00	-9.87E-05	0	-1508	-167.4543	0	0	-1676	0	2E-14
-3.08E-04	-1.04	1.30E-04	0	-1537	-194.2404	54.53	0	-1677	7.95E-05	3E+03
-5.36E-04	0.14	5.58E-04	0	-1722	-266.2962	311.9	0	-1677	0.0001988	4E+03
-8.04E-04	0.59	1.17E-03	0	-2033	-352.5532	709.4	0	-1676	0.0003578	6E+03
-1.13E-03	0.80	1.93E-03	0	-2435	-466.1136	1225	0	-1677	0.0005567	8E+03
-1.50E-03	0.95	2.88E-03	0	-2809	-591.0995	1724	0	-1676	0.0007952	10419
-1.86E-03	1.10	4.04E-03	0	-2974	-693.3007	1991	0	-1676	0.001074	11329
-2.24E-03	1.22	5.42E-03	0	-3053	-785.8834	2163	0	-1676	0.001392	11865
-2.65E-03	1.32	6.98E-03	0	-3094	-890.0533	2306	0	-1678	0.001749	12239
-3.10E-03	1.39	8.72E-03	0	-3126	-952.9766	2403	0	-1676	0.002147	12458
-3.62E-03	1.43	1.06E-02	0	-3196	-993.8605	2514	0	-1676	0.002584	12706
-4.21E-03	1.46	1.26E-02	0	-3248	-1045	2617	0	-1676	0.003062	12913
-4.91E-03	1.46	0.0148	0	-3289	-1069	2681	0	-1677	0.003578	13006
-5.66E-03	1.47	0.0171	0	-3343	-1090	2757	0	-1676	0.004135	13182
-6.45E-03	1.47	0.0196	0	-3397	-1111	2831	0	-1676	0.004732	13366
-7.29E-03	1.48	0.0223	0	-3442	-1134	2900	0	-1676	0.005368	13516
-8.20E-03	1.48	0.0251	0	-3483	-1160	2967	0	-1676	0.006044	13653
-9.17E-03	1.48	0.028	0	-3518	-1192	3033	0	-1676	0.006759	13787
-1.02E-02	1.48	0.0312	0	-3541	-1229	3096	0	-1674	0.007515	13903
-1.13E-02	1.47	0.0344	0	-3582	-1243	3149	0	-1676	0.00831	14010

Note: units on the table - Kip, ft, F

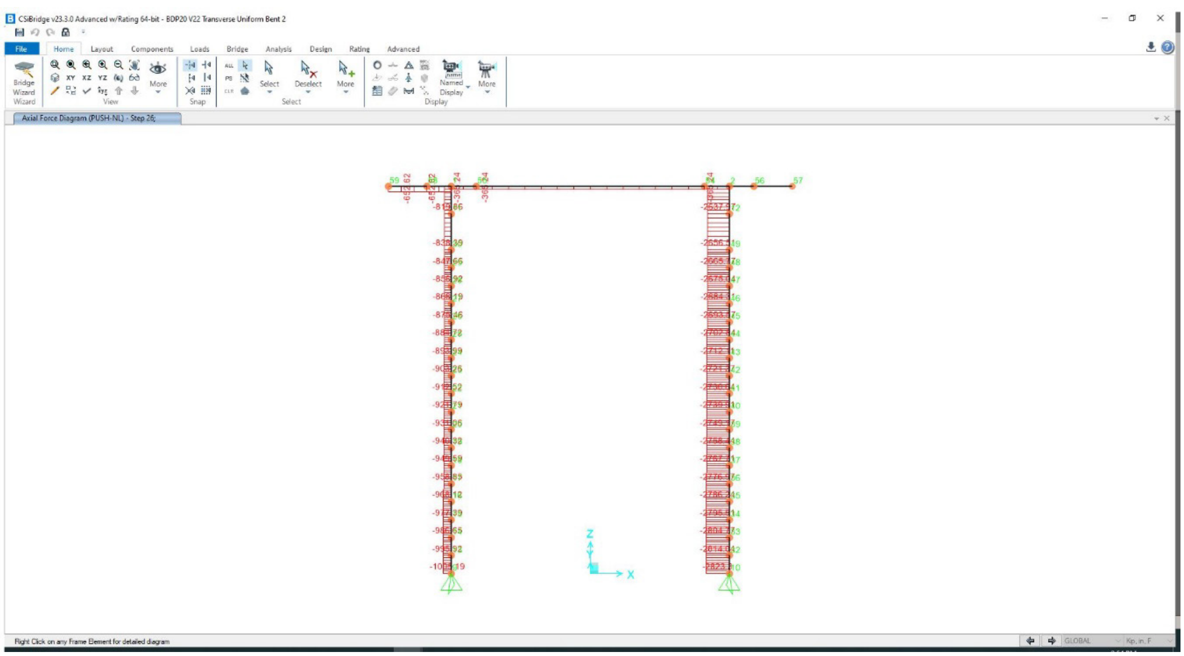
APPENDIX 20.1.3–3. Transverse Pushover Analysis of Bent 2 using *CSiBridge*

Uniform Column Section:

I. Yield Displacement When Front Side Column Yield



II. Axial Load Distribution When Front Side column yield

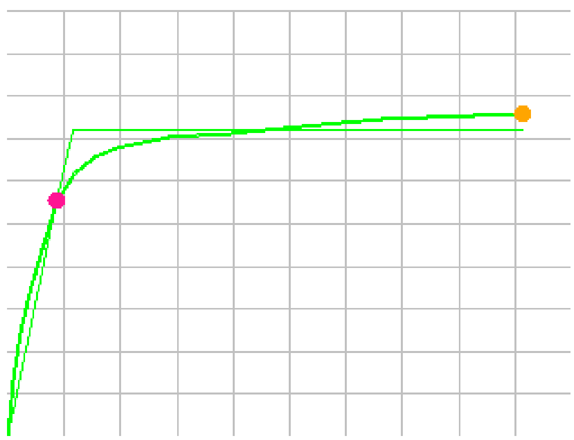


APPENDIX 20.1.3–4. *CSiBridge* Section-Designer Output, Front Side Column

Bent 2: Front Side Column (Include overturning effect)

MOMENT CURVATURE *(M-ϕ)* GRAPH - Kip, ft, F

Axial Load = -2638
Moment Angle = 0



Results For Exact-Integration

$\phi_{y(Initial)}$ = 7.047E-04
M_y = 11071
$\phi_{y(Idealized)}$ = 9.178E-04
M_p = 14418
I_{crack} = 24.9536
$\phi_{concrete}$ = 7.310E-03
$M_{concrete}$ = 15153
ϕ_{steel} = N/A
M_{steel} = N/A

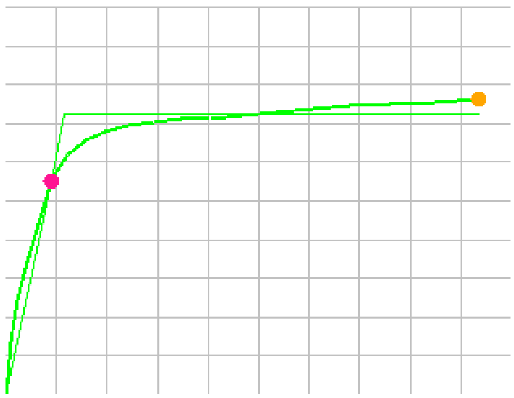
Conc. Strain	Neutral Axis	Steel Strain	Tendon Strain	Conc. Comp.	Steel Comp.	Steel Ten.	Prestress	Net Force	Curvature	Moment
-1.55E-04	0	-1.55E-04	0	-2374	-263.57	0	0	-2638	0	0
-3.52E-04	-2.2001	3.29E-05	0	-2379	-268.38	7.2921	0	-2640	7.00E-05	2743
-5.89E-04	-0.5325	3.74E-04	0	-2480	-337.85	179.8725	0	-2638	1.75E-04	4928
-8.52E-04	0.1268	8.80E-04	0	-2706	-423.77	491.4636	0	-2638	3.15E-04	6797
-1.16E-03	0.4624	1.53E-03	0	-3024	-530.52	914.6272	0	-2639	4.90E-04	8802
-1.53E-03	0.6524	2.32E-03	0	-3413	-654.6	1428.819	0	-2639	7.00E-04	11042
-1.90E-03	0.8175	3.29E-03	0	-3637	-781.99	1781.025	0	-2638	9.44E-04	12397
-2.29E-03	0.9612	4.45E-03	0	-3732	-898.86	1990.679	0	-2640	1.22E-03	13125
-2.71E-03	1.0756	5.77E-03	0	-3776	-1001	2138.738	0	-2638	1.54E-03	13589
-3.17E-03	1.1551	7.23E-03	0	-3835	-1051	2246.767	0	-2640	1.89E-03	13855
-3.68E-03	1.2164	8.84E-03	0	-3878	-1086	2325.46	0	-2639	2.27E-03	14056
-4.26E-03	1.2499	1.06E-02	0	-3930	-1106	2397.464	0	-2638	2.69E-03	14188
-4.96E-03	1.258	0.0124	0	-3980	-1134	2475.996	0	-2638	3.15E-03	14249
-5.70E-03	1.2678	0.0143	0	-4043	-1163	2568.284	0	-2638	3.64E-03	14407
-6.46E-03	1.2813	0.0164	0	-4084	-1191	2636.276	0	-2639	4.16E-03	14581
-7.27E-03	1.2928	0.0187	0	-4116	-1220	2699.496	0	-2637	4.72E-03	14756
-8.17E-03	1.2961	0.0211	0	-4162	-1224	2747.595	0	-2639	5.32E-03	14874
-9.14E-03	1.2966	0.0236	0	-4203	-1227	2791.851	0	-2638	5.95E-03	14974
-1.02E-02	1.2948	0.0262	0	-4237	-1235	2834.318	0	-2638	6.61E-03	15068
-1.13E-02	1.2908	0.029	0	-4266	-1245	2874.38	0	-2636	0.00731	15153

Note: units on the table - Kip, ft, F

Bent 3: Front Side Column (Include overturning effect)

MOMENT CURVATURE (*M-ϕ*) GRAPH - Kip, ft, F

Axial Load = -2616
Moment Angle = 0



Results For Exact-Integration

$\phi_{y(Initial)}$ = 7.070E-04
M_y = 11004
$\phi_{y(Idealized)}$ = 9.288E-04
M_p = 14458
I_{crack} = 24.7241
$\phi_{concrete}$ = 7.486E-03
$M_{concrete}$ = 15216
ϕ_{steel} = N/A
M_{steel} = N/A

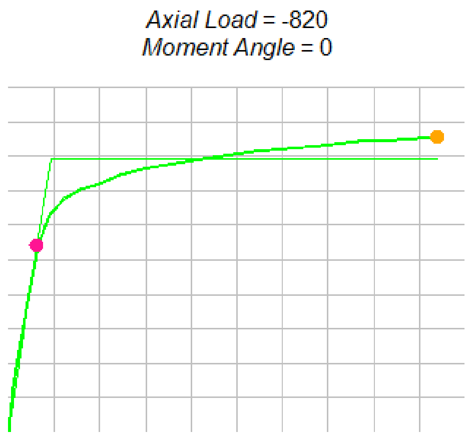
Conc. Strain	Neutral Axis	Steel Strain	Tendon Strain	Conc. Comp.	Steel Comp.	Steel Ten.	Prestress	Net Force	Curvature	Moment
-1.56E-04	0	-1.56E-04	0	-2351	-265	0	0	-2616	0	0
-3.57E-04	-2.1538	3.70E-05	0	-2354	-271	8.7643	0	-2616	7.16E-05	2765
-5.99E-04	-0.5083	3.87E-04	0	-2462	-342	187.59	0	-2616	1.79E-04	4947
-8.68E-04	0.1401	9.06E-04	0	-2693	-431	507.23	0	-2616	3.22E-04	6829
-1.19E-03	0.4669	1.57E-03	0	-3015	-541	938.74	0	-2618	5.02E-04	8850
-1.56E-03	0.6523	2.38E-03	0	-3406	-670	1458.5	0	-2618	7.16E-04	11103
-1.95E-03	0.8203	3.38E-03	0	-3613	-799	1795.8	0	-2616	9.67E-04	12390
-2.34E-03	0.965	4.56E-03	0	-3704	-917	2003.6	0	-2618	1.25E-03	13116
-2.76E-03	1.0803	5.91E-03	0	-3750	-1014	2148	0	-2616	1.58E-03	13576
-3.24E-03	1.1602	7.41E-03	0	-3819	-1058	2259.2	0	-2617	1.93E-03	13857
-3.75E-03	1.2245	9.07E-03	0	-3862	-1090	2335.4	0	-2617	2.33E-03	14078
-4.33E-03	1.2629	1.08E-02	0	-3930	-1105	2417.4	0	-2618	2.76E-03	14268
-5.06E-03	1.2632	0.0127	0	-3973	-1137	2493.5	0	-2616	3.22E-03	14277
-5.82E-03	1.2711	0.0147	0	-4036	-1167	2586.8	0	-2616	3.73E-03	14428
-6.60E-03	1.2857	0.0169	0	-4071	-1195	2649.4	0	-2617	4.26E-03	14599
-7.43E-03	1.2976	0.0192	0	-4106	-1224	2712.2	0	-2617	4.84E-03	14776
-8.34E-03	1.3021	0.0216	0	-4154	-1224	2761.4	0	-2616	5.44E-03	14895
-9.31E-03	1.3041	0.0242	0	-4196	-1228	2808.4	0	-2616	6.09E-03	15007
-1.04E-02	1.3041	0.0269	0	-4234	-1237	2854.7	0	-2616	6.77E-03	15116
-1.15E-02	1.3022	0.0297	0	-4267	-1246	2897.2	0	-2616	0.007486	15216

Note: units on the table - Kip, ft, F

APPENDIX 20.1.3–5. *CSiBridge* Section-Designer Output, Back Side Column

Bent 2:Back Side Column (Include overturning effect)

MOMENT CURVATURE *(M-ϕ)* GRAPH - Kip, ft, F



Results For Exact-Integration

$\phi_{y(Initial)}$ = 6.246E-04
M_y = 8105.6795
$\phi_{y(Idealized)}$ = 9.178E-04
M_p = 11911
I_{crack} = 20.6146
$\phi_{concrete}$ = 9.405E-03
$M_{concrete}$ = 12862
ϕ_{steel} = N/A
M_{steel} = N/A

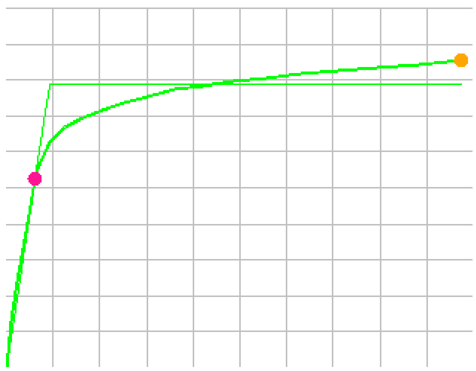
Conc. Strain	Neutral Axis	Steel Strain	Tendon Strain	Conc. Comp.	Steel Comp.	Steel Ten.	Pres tress	Net Force	Curvature	Moment
-4.82E-05	0.00E+00	-4.82E-05	0	-736.9	-81.8	0	0	-819	0.00E+00	6E-15
-2.50E-04	5.99E-02	2.46E-04	0	-831	-125.87	135	0	-822	9.00E-05	2E+03
-4.71E-04	7.42E-01	7.68E-04	0	-1104	-197.47	480.5	0	-821	0.000225	4E+03
-7.49E-04	9.84E-01	1.48E-03	0	-1497	-291.46	967.3	0	-821	0.000405	6E+03
-1.10E-03	1.09E+00	2.37E-03	0	-1978	-411.4	1569	0	-821	0.00063	8E+03
-1.45E-03	1.22E+00	3.50E-03	0	-2256	-508.34	1944	0	-820	0.0009	9495.94
-1.81E-03	1.35E+00	4.88E-03	0	-2396	-601.08	2177	0	-820	0.001215	10163
-2.18E-03	1.45E+00	6.49E-03	0	-2470	-693.63	2342	0	-821	0.001575	10566
-2.56E-03	1.5388	8.33E-03	0	-2493	-782.27	2455	0	-820	0.00198	10833
-3.02E-03	1.5905	1.04E-02	0	-2554	-858.06	2590	0	-821	0.00243	11164
-3.53E-03	1.6282	1.26E-02	0	-2605	-894.88	2679	0	-820	0.002925	11415
-4.12E-03	1.6454	0.015	0	-2659	-931.48	2770	0	-821	0.003465	11645
-4.82E-03	1.6435	0.0175	0	-2694	-987.43	2862	0	-820	0.00405	11829
-5.59E-03	1.6384	0.0202	0	-2762	-1007	2949	0	-821	0.00468	12013
-6.42E-03	1.6352	0.0231	0	-2831	-1022	3033	0	-820	0.005355	12183
-7.30E-03	1.631	0.0261	0	-2896	-1039	3114	0	-820	0.006075	12345
-8.22E-03	1.6315	0.0294	0	-2932	-1054	3166	0	-820	0.00684	12487
-9.20E-03	1.6313	0.0329	0	-2959	-1072	3211	0	-821	0.00765	12617
-1.02E-02	1.6295	0.0366	0	-2980	-1096	3256	0	-820	0.008505	12741
-1.13E-02	1.6271	0.0404	0	-2995	-1127	3301	0	-820	0.009405	12862

Note: units on the table - Kip, ft, F

Bent 3: Back Side Column (Include overturning effect)

MOMENT CURVATURE *(M-ϕ)* GRAPH - Kip, ft, F

Axial Load = -736
Moment Angle = 0



Results For Exact-Integration

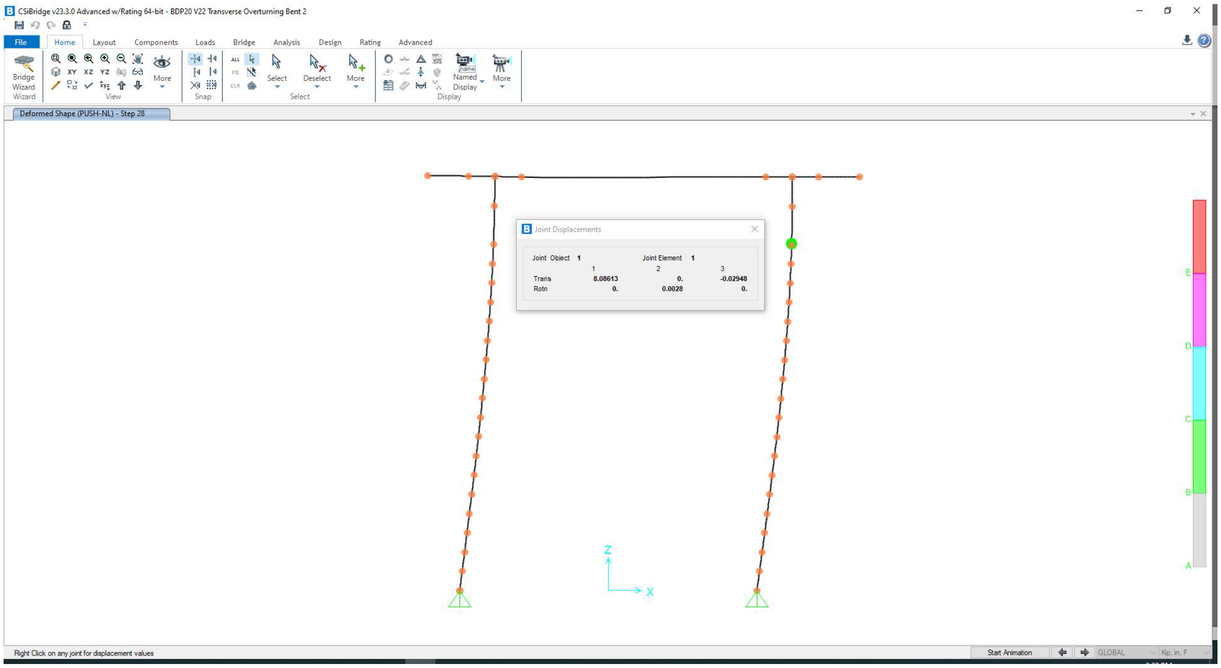
$\phi_{y(Initial)}$ = 6.218E-04
M_y = 7880.2928
$\phi_{y(Idealized)}$ = 9.330E-04
M_p = 11823
I_{crack} = 20.129
$\phi_{concrete}$ = 9.735E-03
$M_{concrete}$ = 12809
ϕ_{steel} = N/A
M_{steel} = N/A

Conc. Strain	Neutral Axis	Steel Strain	Tendon Strain	Conc. Comp .	Steel Comp.	Steel Ten.	Prestress	Net Force	Curvature	Moment
-4.38E-05	0	-4.38E-05	0	-660	-74	0	0	-733.95	0	0
-2.47E-04	0.1837	2.66E-04	0	-765	-121	150	0	-736.21	9.32E-05	1974
-4.73E-04	0.803	8.09E-04	0	-1054	-195	512	0	-736.28	2.33E-04	3620
-7.61E-04	1.0178	1.55E-03	0	-1461	-292	1016	0	-736.63	4.19E-04	5728
-1.13E-03	1.1085	2.46E-03	0	-1946	-416	1625	0	-737.13	6.52E-04	8202
-1.48E-03	1.2442	3.65E-03	0	-2205	-512	1981	0	-736.18	9.32E-04	9430
-1.84E-03	1.3719	5.08E-03	0	-2330	-606	2201	0	-736.18	1.26E-03	10038
-2.22E-03	1.4744	6.76E-03	0	-2404	-699	2366	0	-736.12	1.63E-03	10438
-2.61E-03	1.5601	8.67E-03	0	-2431	-787	2482	0	-736.71	2.05E-03	10726
-3.07E-03	1.6126	1.08E-02	0	-2496	-856	2615	0	-736.17	2.52E-03	11065
-3.58E-03	1.6501	1.31E-02	0	-2555	-887	2705	0	-737.85	3.03E-03	11339
-4.17E-03	1.6714	1.56E-02	0	-2622	-918	2804	0	-736.47	3.59E-03	11612
-4.91E-03	1.6627	0.0182	0	-2654	-980	2897	0	-736.25	4.19E-03	11776
-5.71E-03	1.6545	0.0209	0	-2719	-1,000	2982	0	-736.33	4.84E-03	11933
-6.56E-03	1.65	0.0239	0	-2789	-1015	3068	0	-736.17	5.54E-03	12103
-7.45E-03	1.6484	0.0272	0	-2844	-1029	3135	0	-737.58	6.29E-03	12266
-8.37E-03	1.6505	0.0306	0	-2878	-1041	3182	0	-737.15	7.08E-03	12407
-9.36E-03	1.651	0.0342	0	-2908	-1058	3230	0	-736.42	7.92E-03	12546
-1.04E-02	1.6504	0.038	0	-2932	-1079	3275	0	-736.15	8.80E-03	12677
-1.15E-02	1.6488	0.042	0	-2951	-1107	3321	0	-736.86	0.00974	12809

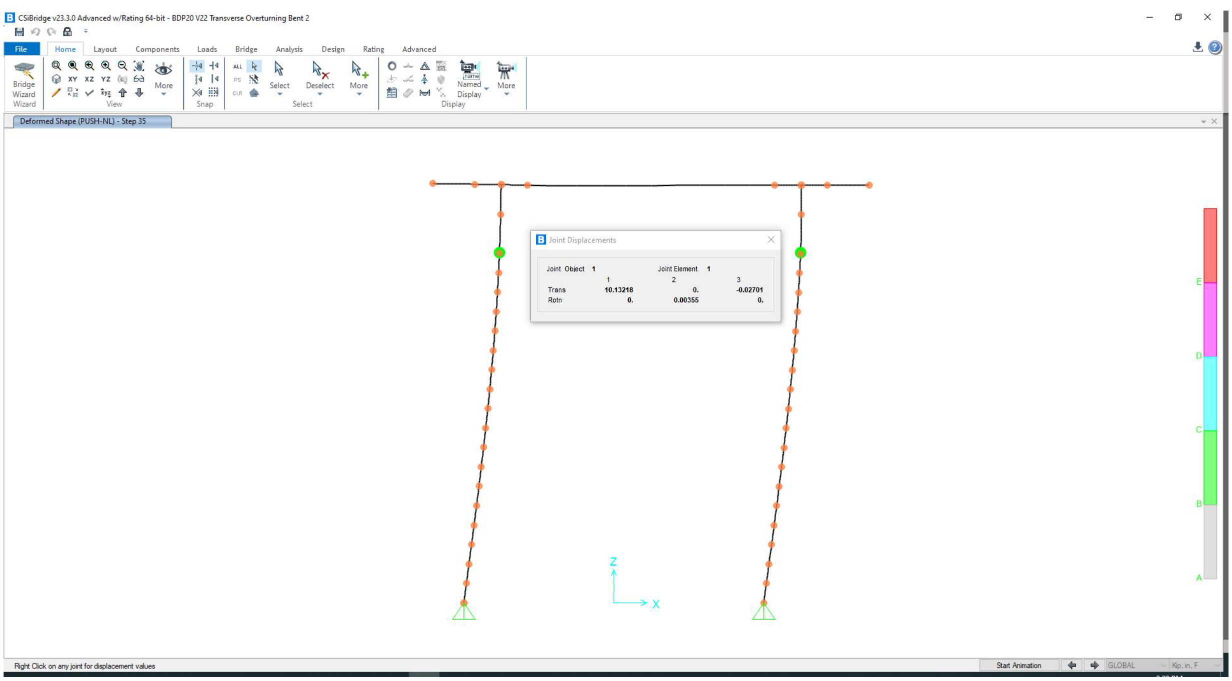
Note: units on the table - Kip, ft, F

APPENDIX 20.1.3–6. Bent 2 Transverse Pushover Analysis using *CSiBridge*

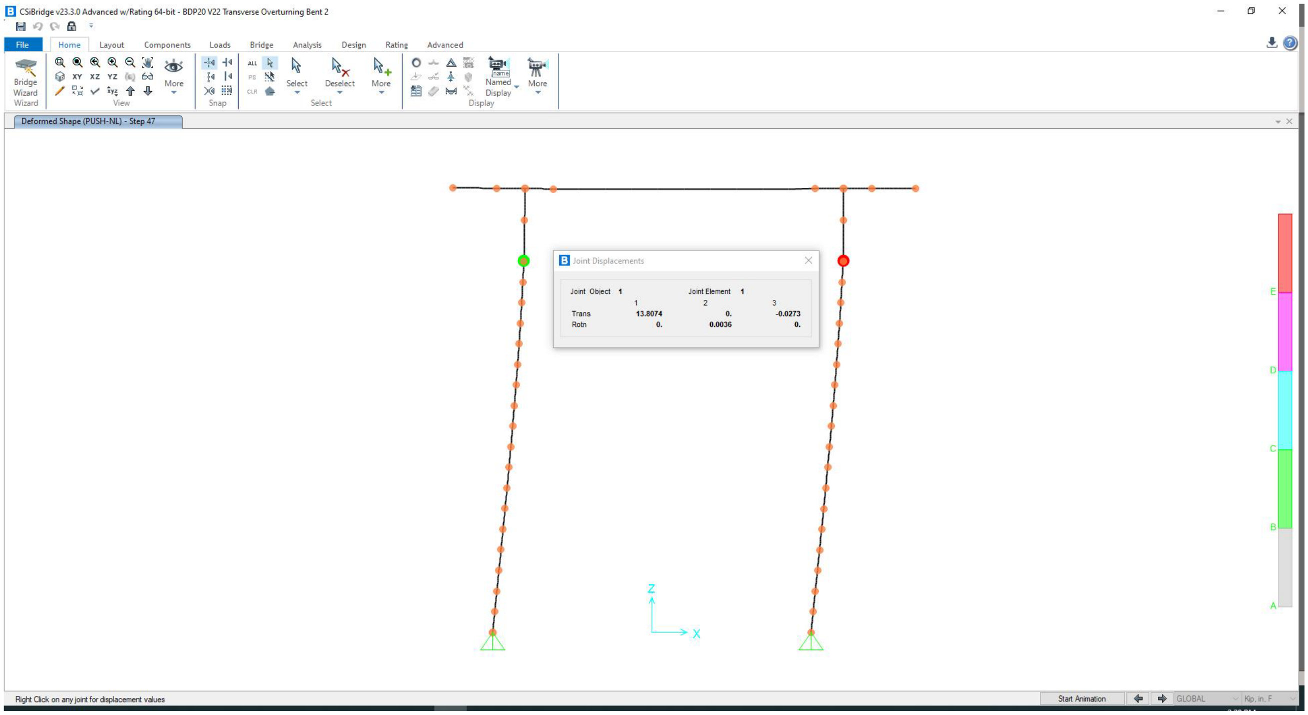
I. Displacement Yield Capacity When Front Column Yield

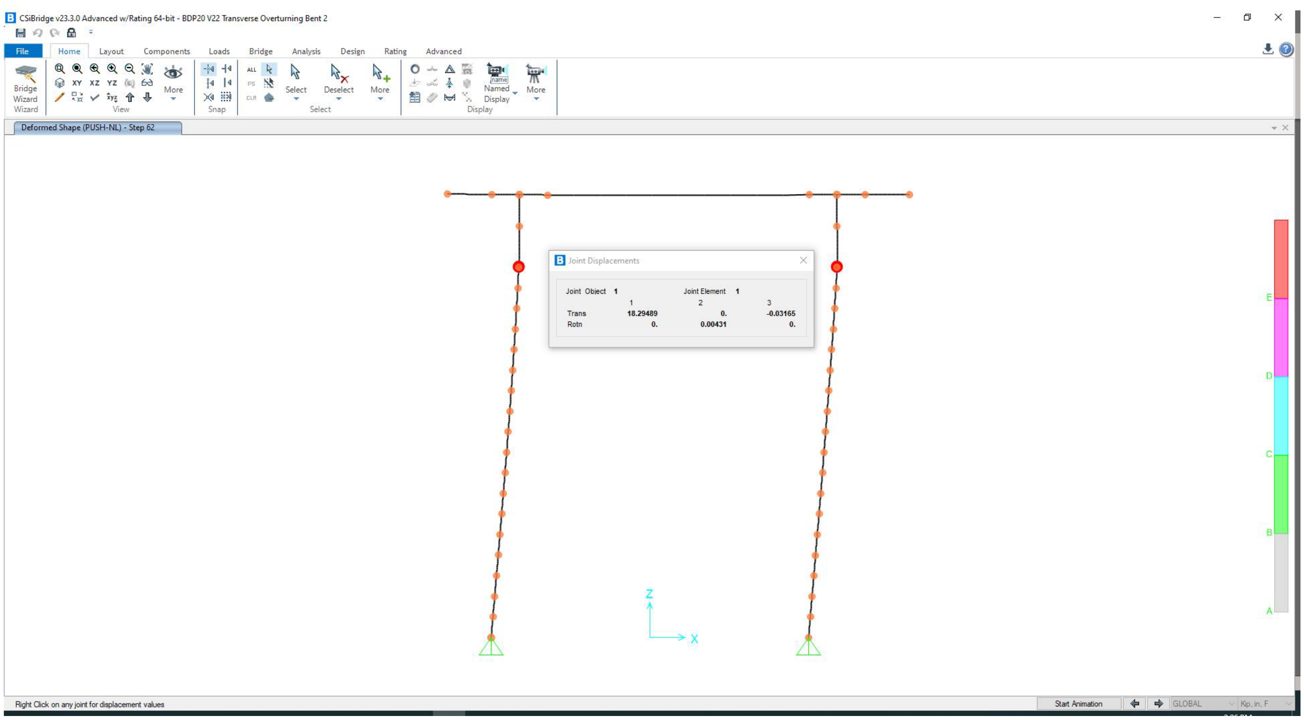


II. Yield Displacement When Both Columns Yield



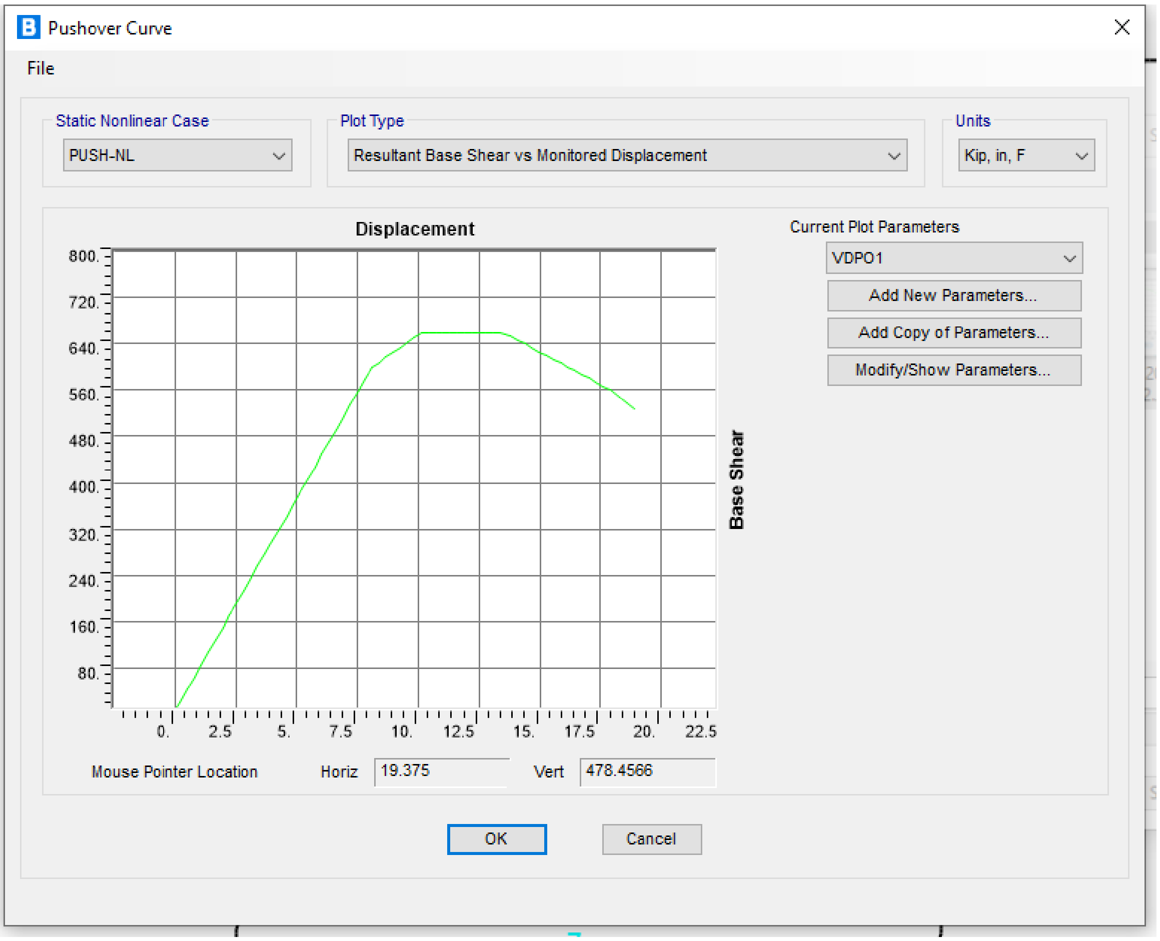
III. Bent 2 - Maximum Plastic Yield Displacement Capacity



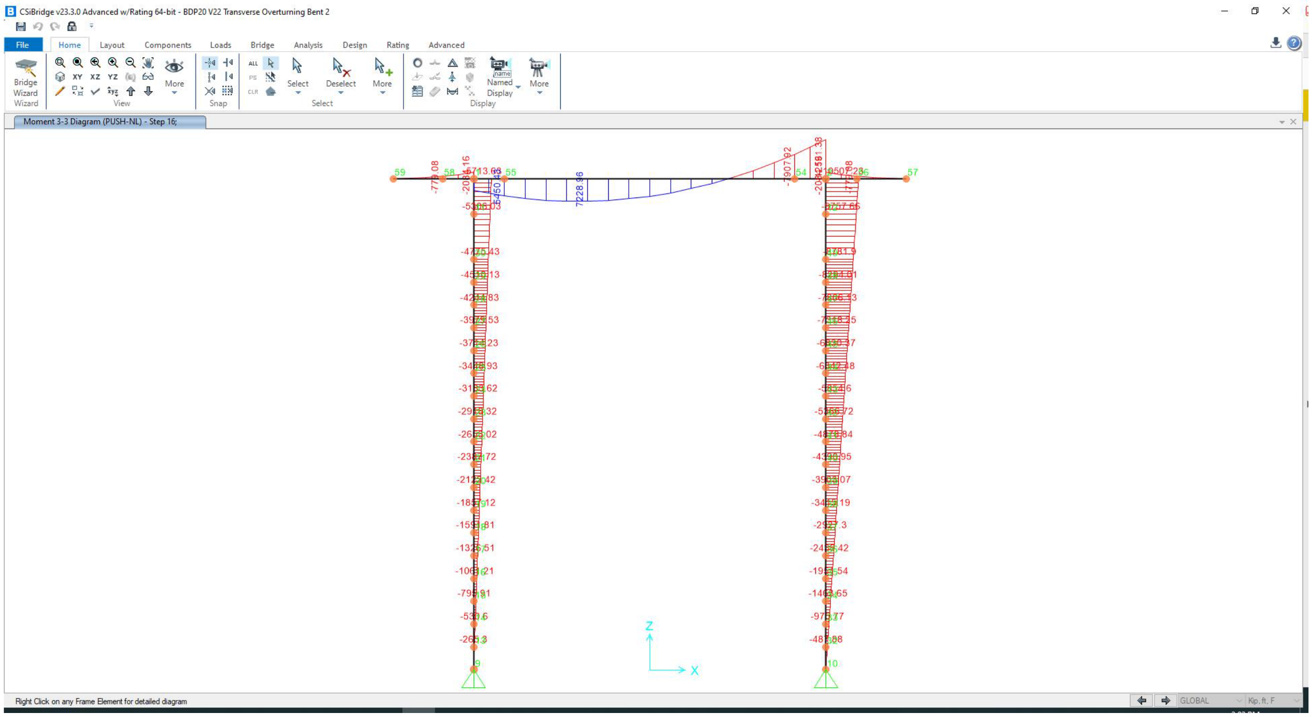


APPENDIX 20.1.3–7. Bent 2 - Force – Displacement Relationship that Includes Overturning Effect

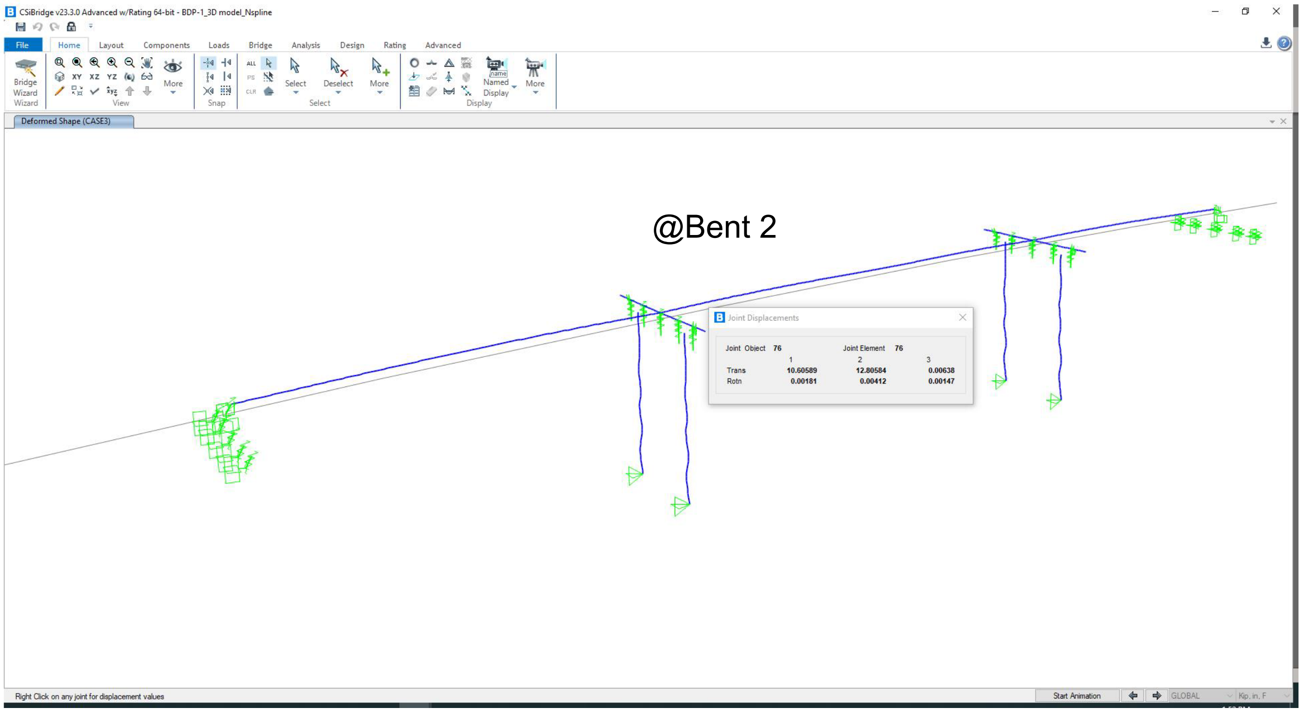
a. Bent 2:– Force – Displacement Relationship

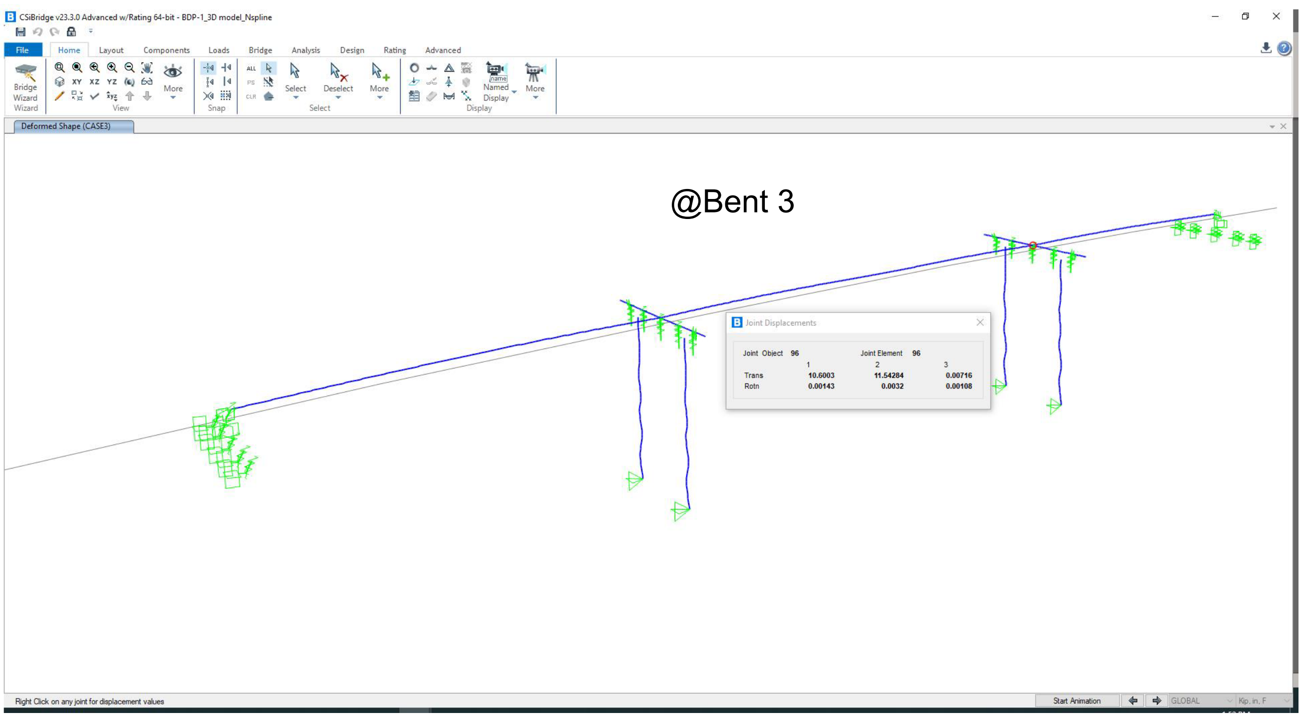


b. Bent 2: Moment Demand at 10% of tributary weight applied laterally



APPENDIX 20.1.3–8. *CSiBridge* 3D Response Spectrum Analysis for Displacement Demand





APPENDIX 20.1.3–9. Joint Movement Calculation

JOINT MOVEMENT RATING (MR) CALCULATIONS

EA	DISTRICT	COUNTY	ROUTE	PM	BRIDGE NAME AND NUMBER
910076	59	ES	999	99	Prototype Bridge

TYPE STRUCTURE	TYPE ABUTMENT	TYPE EXPANSION (2" elasto pads etc.)
CIP/PS BOX GIRDER	Seat Type	Elastomeric Bearing Pads

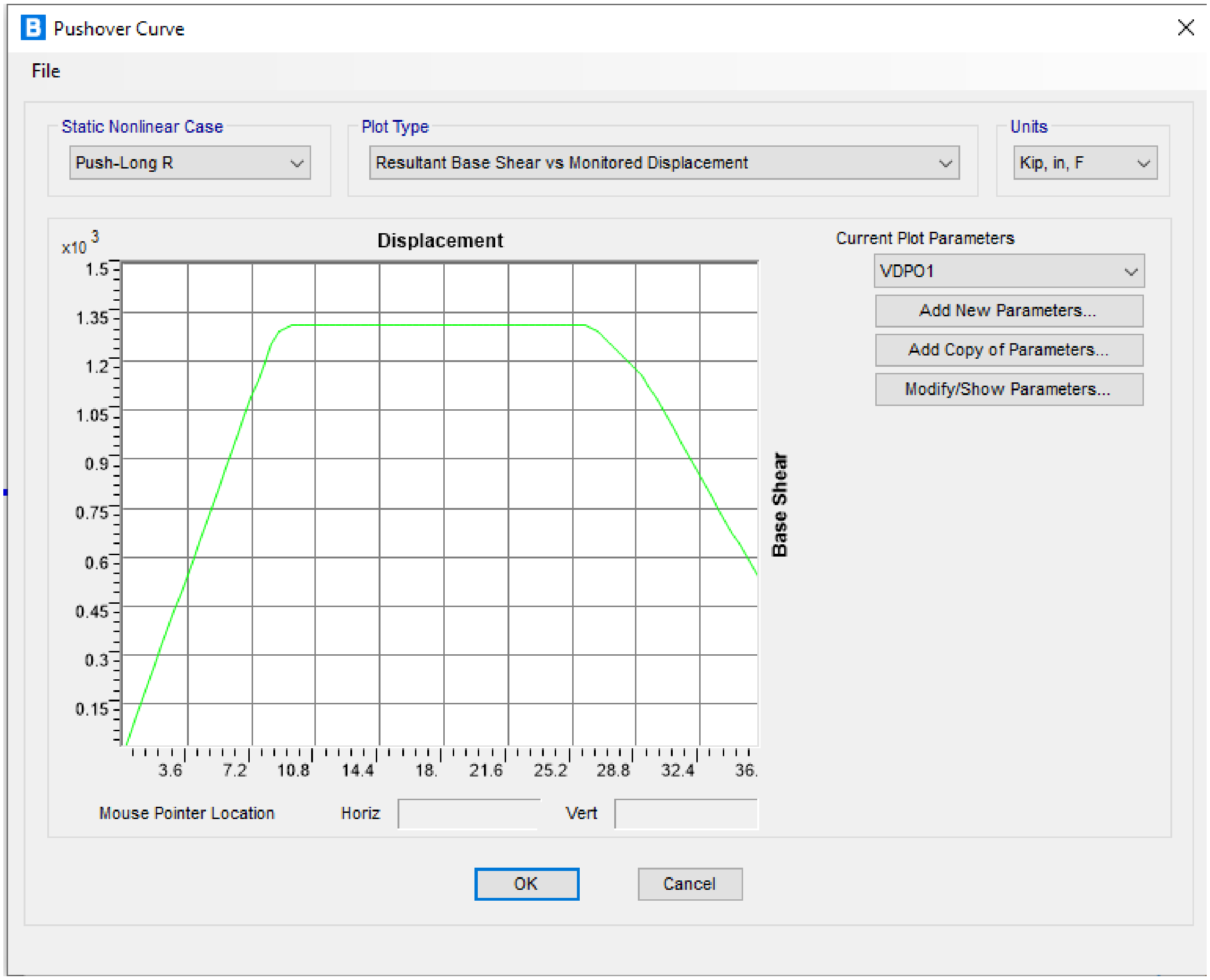
(1) TEMPERATURE EXTREMES per Proj. Scope Summary Report		TYPE OF STRUCTURE		(2) THERMAL MOVEMENT (inches/100 feet)		ANTICIPATED SHORTENING (inches/100 feet)		(3) MOVEMENT FACTOR (inches/100 feet)
MAXIMUM	110 F	__ Steel	Range(0	F)*(.0000065 x 1200) =		+ 0.00	=	
- MINIMUM	23 F	X Concrete (Conventional)	Range(0	F)*(.0000060 x 1200) =		+ 0.06	=	
		__ Concrete (Pretensioned)	Range(0	F)*(.0000060 x 1200) =		+ 0.12	=	
= RANGE	87 F	__ Concrete (Post tensioned)	Range(87	F)*(.0000060 x 1200) =	0.6264	+ 0.63	=	1.26

To be Filled in by OSD		To be Filled in by SR	
Name:	Date: 5/3/2022	Name:	Date:

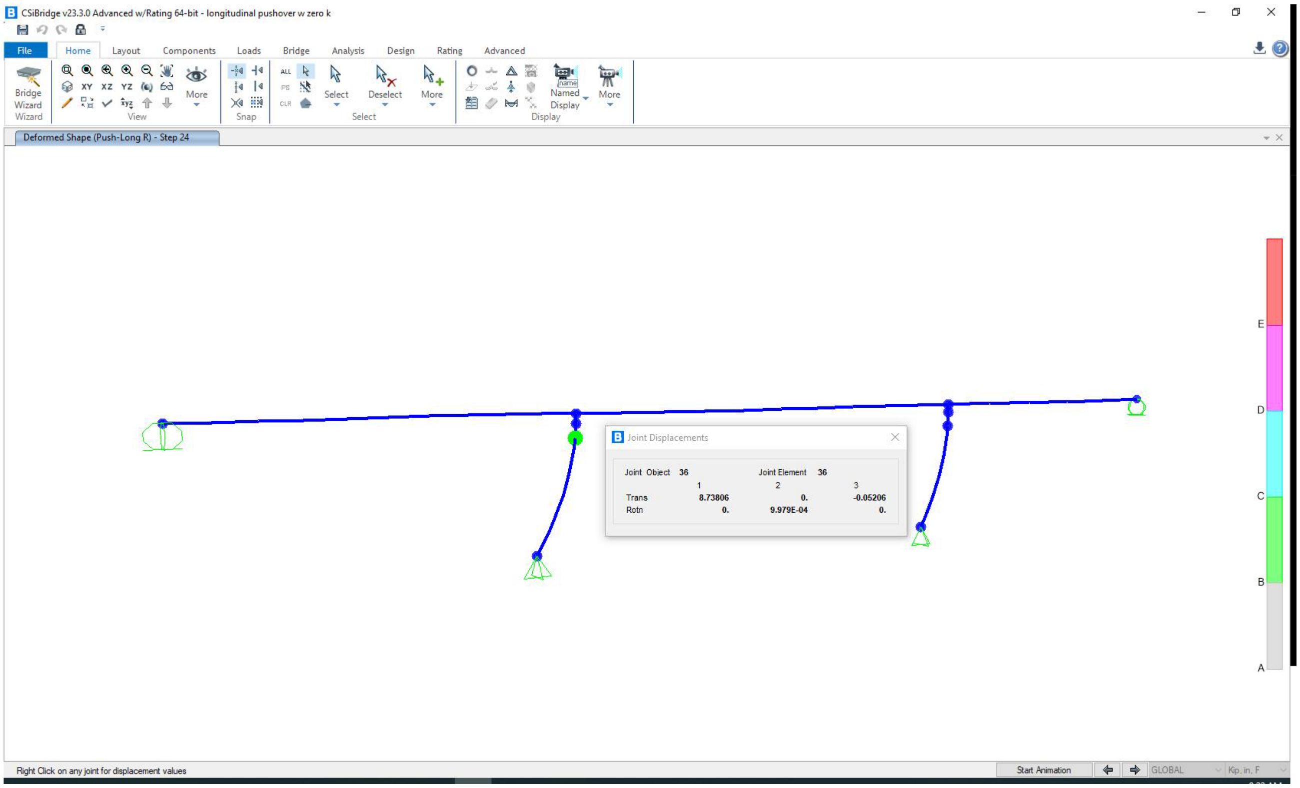
LOCATION	(EQ) Seismic Movement (inches)	Skew (degrees)	(4) Contributing Length (feet)	Calculated Movement (3) x (4) / 100 (inches)	M.R. (Round up to 1/2") (inches)	Seal Type A, B, (Others) or Open Joint	Seal Width Limits: Catalog Number	Seal Width Limits: W1 (inches) Maximum	Seal Width Limits: (5) W2 (inches) Min. @ Max Temp.	Groove (saw cut) Width or Installation Width: Structure Temperture (F)°	Groove (saw cut) Width or Installation Width: (6) Adjust from Max. Temp. (inches) k/(1) x (2) x (4)/100	Groove (saw cut) Width or Installation Width: Width at Temp. Listed (inches) w = (5) + (6)
Abut. 1		0.00	202.00	2.54	2.50	Joint Seal Assembly (Strip Seal)						
Abut. 4		0.00	210.00	2.64	3.00	Joint Seal Assembly (Strip Seal)						

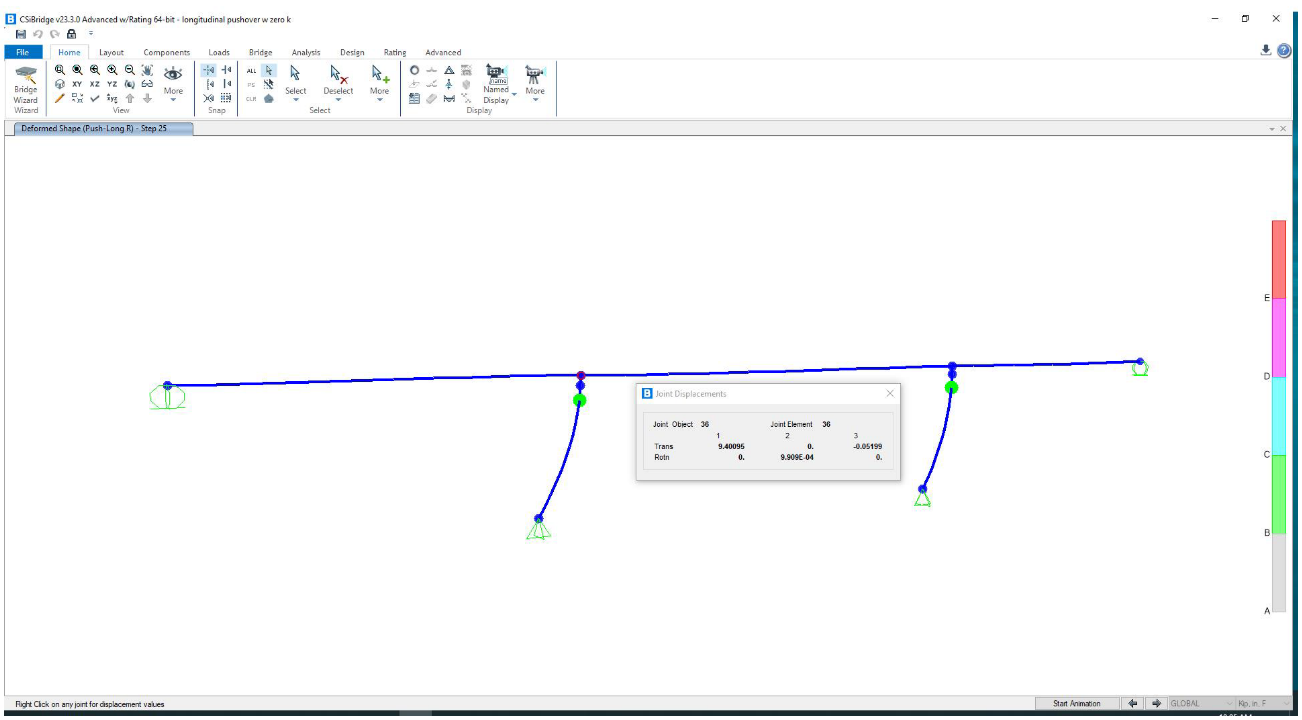
$$Anticipated\ Shortening = \frac{1.26}{100} X \left(\frac{210 + 202}{2} \right) = 2.60\, in.$$

APPENDIX 20.1.3–10. Longitudinal Pushover:– Force - Displacement Relationship, Right Push

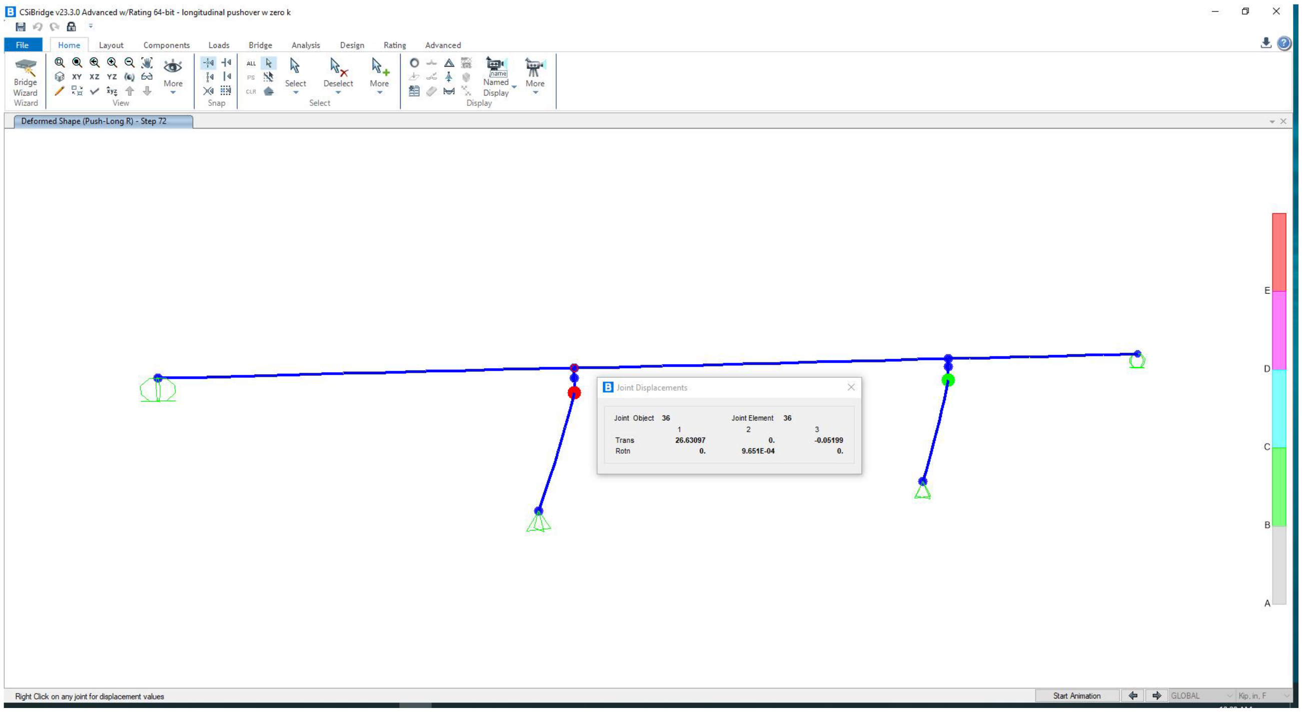


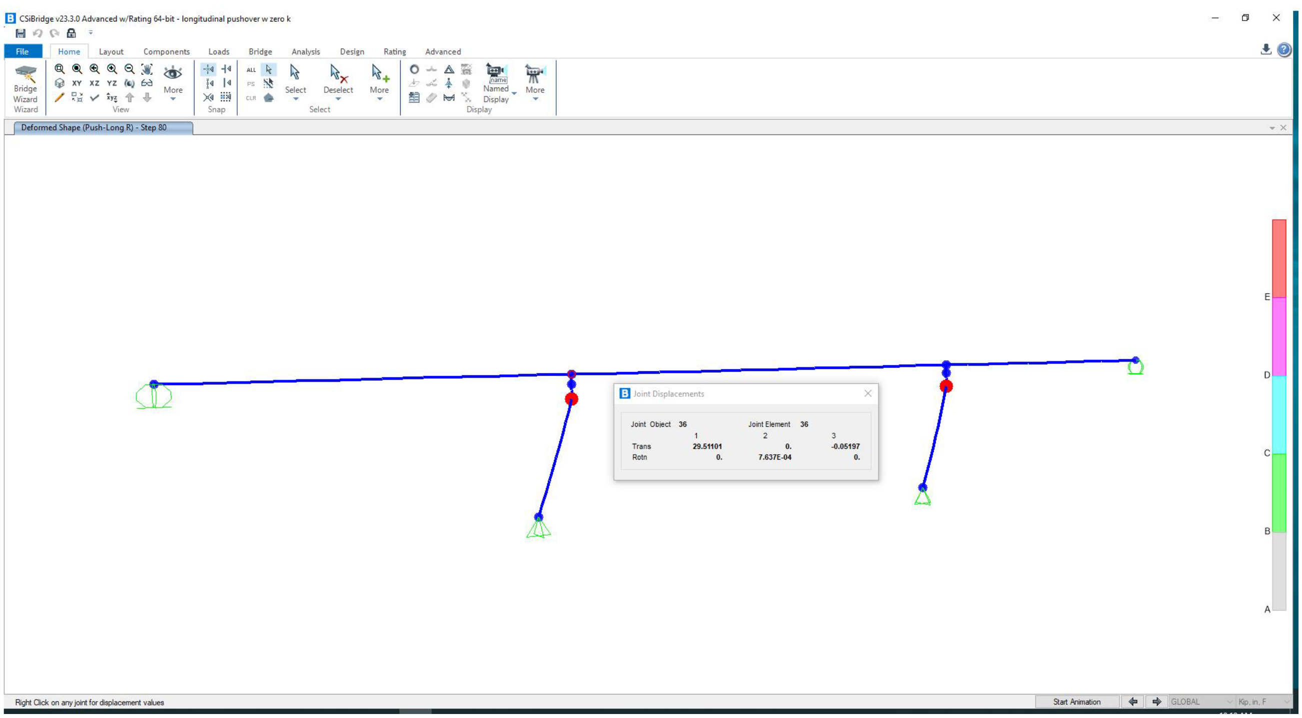
Yield Displacements





Plastic Displacements

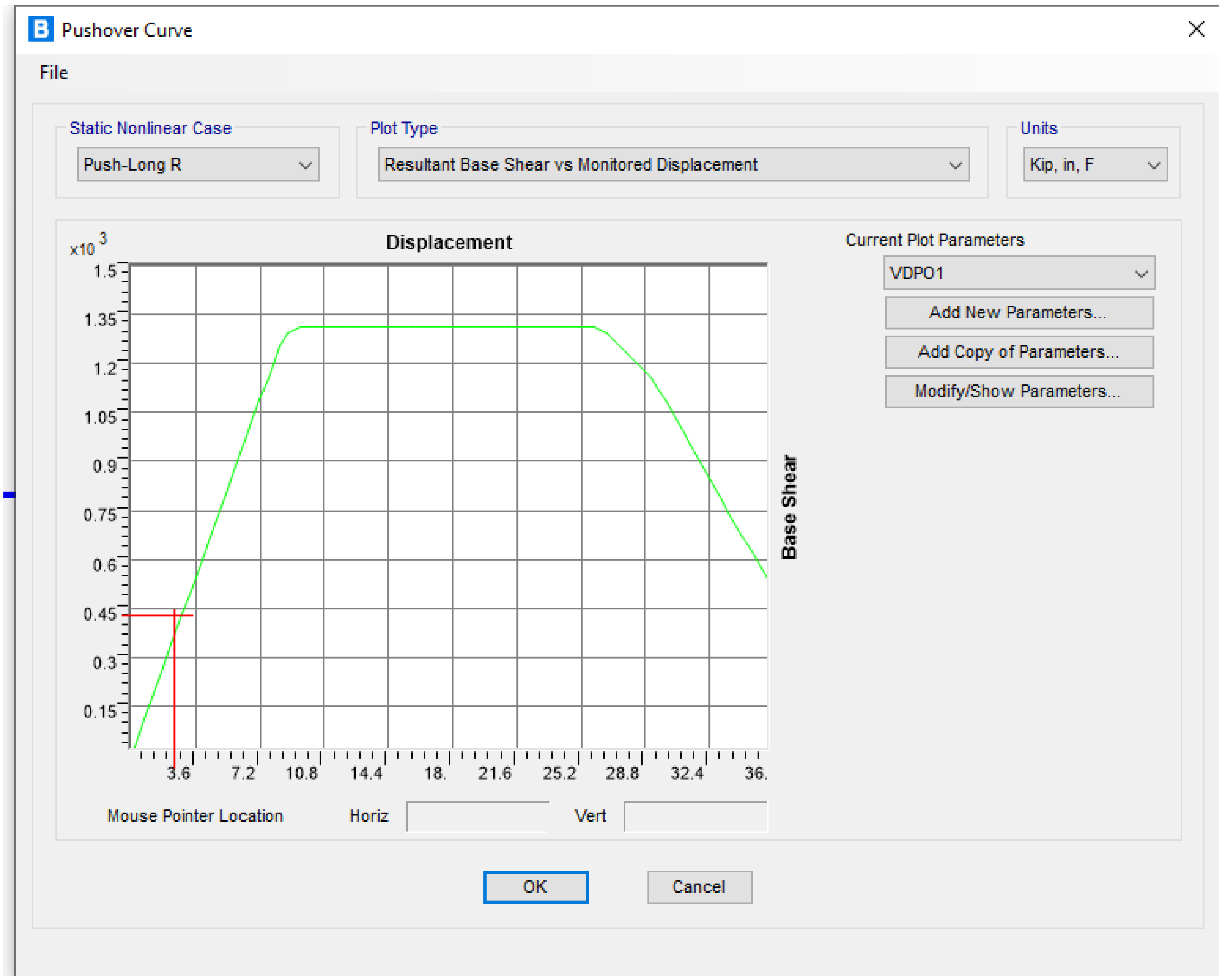




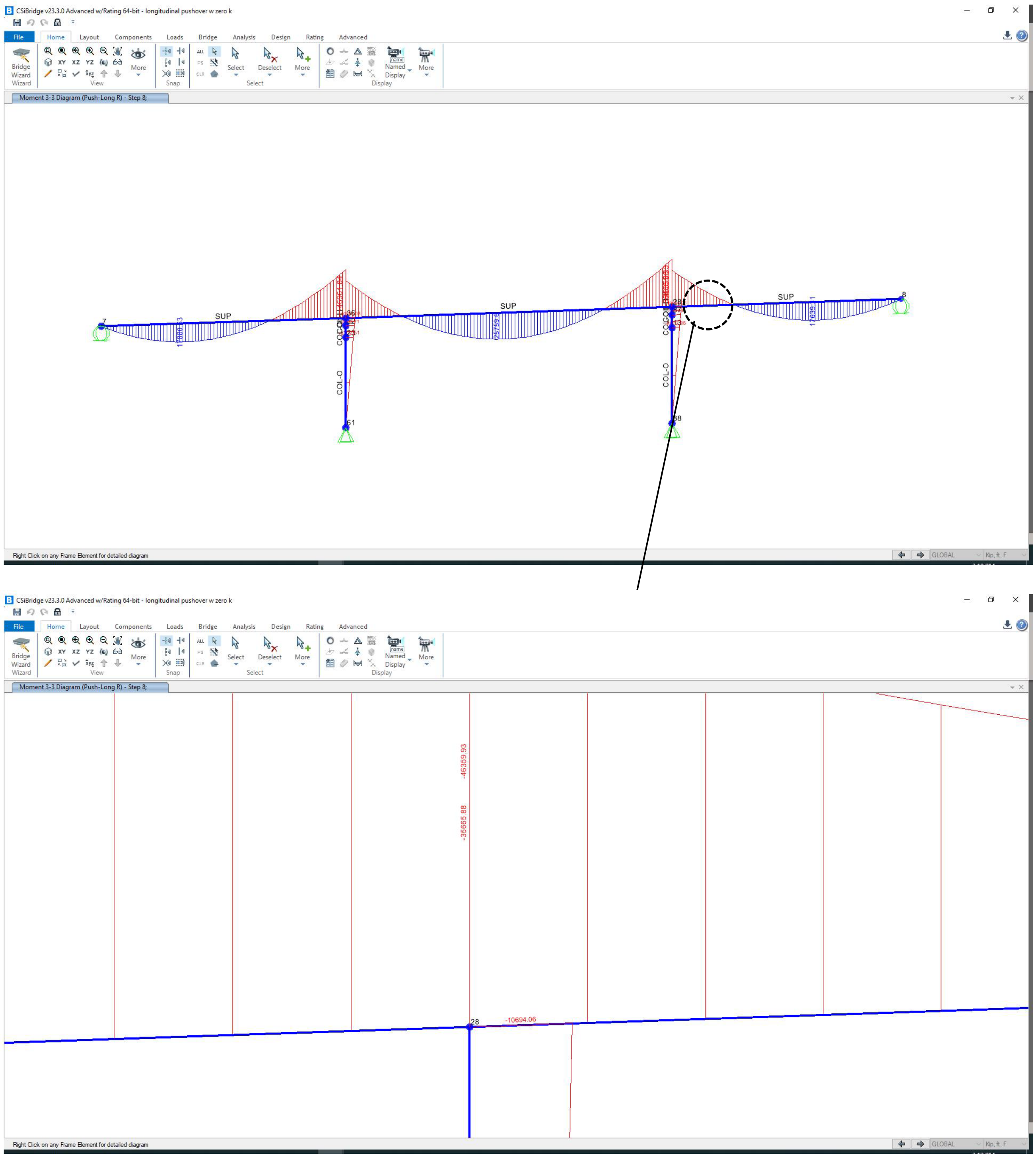
APPENDIX 20.3–11. Longitudinal Direction Soffit Moment Demand @ 10% Tributary Weight

a. Longitudinal Pushover: Force – Displacement Relationship

To Determine Displacement at 419 kips (which 10% of tributary weight) applied laterally and using *CSiBridge* longitudinal pushover curve to get moment demand at bottom of soffit (see Appendix 20.1.3-11b)



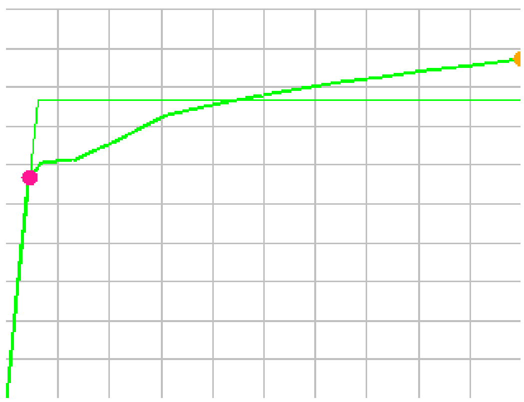
b. Moment Demand at 10% of Tributary weight applied as a lateral load (@ structure Displacement equal to 2.88 in.)



APPENDIX 20.1.3–12. Bent Cap –Flexural Section Capacity, Positive – *CSiBridge* Section-Designer Output

MOMENT CURVATURE *(M-ϕ)* GRAPH - Kip, ft, F

Axial Load = 0
Moment Angle = 0



Results For Exact-Integration

$\phi_{y(Initial)}$ = 4.504E-04
M_y = 14172
$\phi_{y(Idealized)}$ = 6.100E-04
M_p = 19193
I_{crack} = 49.9765
$\phi_{concrete}$ = N/A
$M_{concrete}$ = N/A
ϕ_{steel} = 0.01
M_{steel} = 21780

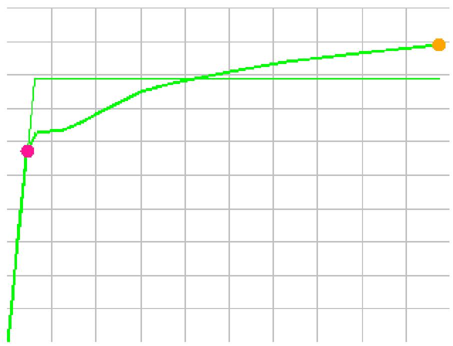
Concrete Strain	Neutral Axis	Steel Strain	Tendon Strain	Concrete Comp	Steel Comp	Steel Tension	P/S Force	Net Force	Curvature	Moment (kip-ft)
0	0	0.00	0	0	0.0	0.0	0	0.00	0	0
-1.08E-04	2.2506	4.96E-04	0	-438	-94.3	531.6	0	-0.69	9.57E-05	3137
-2.69E-04	2.2514	1.24E-03	0	-1094	-235.4	1329.1	0	0.05	2.39E-04	7838
-4.85E-04	2.2479	2.23E-03	0	-1963	-425.7	2390.8	0	2.55	4.31E-04	14086
-6.13E-04	2.46	3.61E-03	0	-2075	-472.0	2545.9	0	-0.59	6.70E-04	15122
-7.26E-04	2.6159	5.31E-03	0	-2076	-474.5	2545.9	0	-4.24	9.57E-04	15227
-8.48E-04	2.7184	7.30E-03	0	-2086	-463.3	2545.9	0	-3.53	1.29E-03	15302
-9.93E-04	2.7821	9.57E-03	0	-2174	-457.7	2630.2	0	-1.75	1.68E-03	15860
-1.15E-03	2.827	1.21E-02	0	-2292	-465.9	2756.1	0	-1.40	2.11E-03	16563
-1.33E-03	2.8599	0.015	0	-2424	-500.5	2923.1	0	-0.95	2.58E-03	17335
-1.53E-03	2.8843	0.0181	0	-2568	-538.6	3105.6	0	-0.70	3.11E-03	18170
-1.72E-03	2.9077	0.0215	0	-2669	-565.4	3234.1	0	-0.20	3.68E-03	18603
-1.93E-03	2.9258	0.0252	0	-2773	-595.3	3368.4	0	0.08	4.31E-03	19040
-2.17E-03	2.9396	0.0292	0	-2880	-630.2	3509.7	0	-0.07	4.98E-03	19496
-2.42E-03	2.9506	0.0335	0	-2978	-668.4	3644.8	0	-1.28	5.69E-03	19888
-2.69E-03	2.9585	0.038	0	-3073	-715.3	3785.1	0	-3.68	6.46E-03	20302
-2.99E-03	2.9646	0.0429	0	-3157	-768.1	3925.1	0	0.21	7.27E-03	20672
-3.31E-03	2.9685	0.048	0	-3233	-832.2	4065.2	0	0.35	8.13E-03	21052
-3.66E-03	2.9706	0.0534	0	-3299	-908.9	4203.2	0	-4.50	9.04E-03	21434
-4.03E-03	2.9717	0.059	0	-3350	-991.1	4341.5	0	0.89	0.01	21780

Note: units on the table - Kip, ft, F

APPENDIX 20.1.3–13. Bent Cap – Flexural Section Capacity, Negative, – *CSiBridge* Section-Designer Output

MOMENT CURVATURE *(M-ϕ)* GRAPH - Kip, ft, F

Axial Load = 0
Moment Angle = 180



Results For Exact-Integration

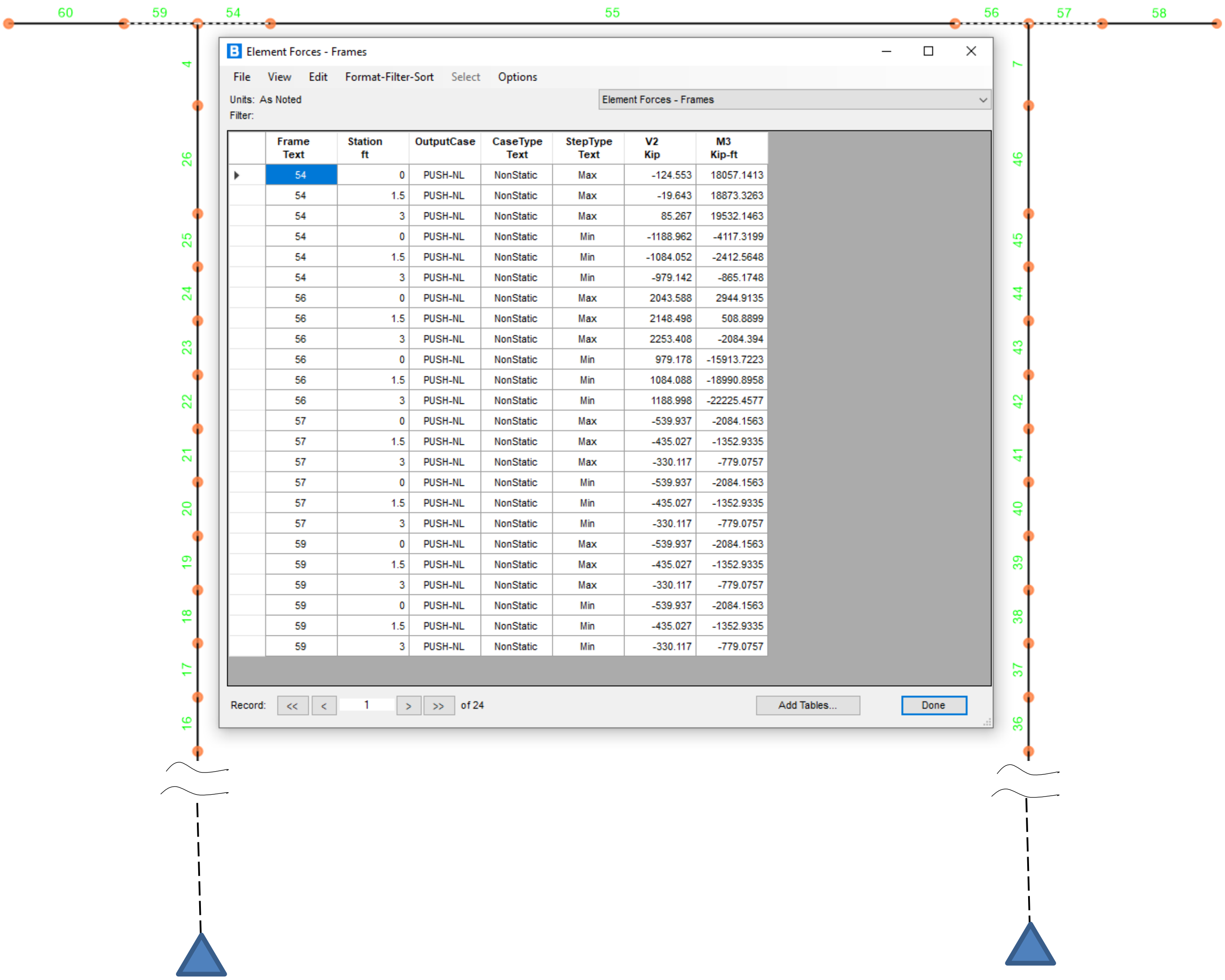
$\phi_{y(Initial)}$ = 4.411E-04
M_y = 17120
$\phi_{y(Idealized)}$ = 6.099E-04
M_p = 23673
I_{crack} = 61.6501
$\phi_{concrete}$ = N/A
$M_{concrete}$ = N/A
ϕ_{steel} = 9.737E-03
M_{steel} = 26757

Concrete Strain	Neutral Axis	Steel Strain	Tendon Strain	Concrete Comp	Steel Comp	Steel Tension	P/S Force	Net Force	Curvature	Moment
0	0	0	0	0	0.0	0.0	0	0.00	0	0
-1.21E-04	2.0768	4.93E-04	0	-556	-79.8	631.7	0	-4.05	9.32E-05	3782
-3.01E-04	2.0826	1.23E-03	0	-1379	-198.0	1580.9	0	4.16	2.33E-04	9426
-5.46E-04	2.0734	2.22E-03	0	-2478	-360.6	2840.6	0	1.79	4.19E-04	16940
-7.05E-04	2.2948	3.59E-03	0	-2737	-404.1	3139.6	0	-1.08	6.52E-04	18866
-8.34E-04	2.4796	5.30E-03	0	-2752	-390.4	3139.6	0	-2.84	9.32E-04	18998
-9.71E-04	2.6028	7.31E-03	0	-2783	-358.8	3139.6	0	-2.19	1.26E-03	19091
-1.14E-03	2.6788	9.61E-03	0	-2902	-330.6	3231.8	0	-1.13	1.63E-03	19721
-1.32E-03	2.7297	1.22E-02	0	-3062	-302.3	3363.5	0	-0.52	2.05E-03	20580
-1.53E-03	2.7666	0.015	0	-3240	-270.2	3509.8	0	-0.35	2.52E-03	21522
-1.76E-03	2.7932	0.0182	0	-3435	-237.7	3669.3	0	-3.04	3.03E-03	22543
-2.00E-03	2.8181	0.0216	0	-3572	-221.4	3795.8	0	2.48	3.59E-03	23115
-2.26E-03	2.8368	0.0254	0	-3708	-216.1	3923.7	0	-0.18	4.19E-03	23642
-2.54E-03	2.8504	0.0294	0	-3843	-214.1	4056.4	0	-0.36	4.85E-03	24182
-2.85E-03	2.8604	0.0337	0	-3964	-214.9	4178.0	0	-0.78	5.54E-03	24644
-3.20E-03	2.8666	0.0382	0	-4077	-222.4	4298.2	0	-1.51	6.29E-03	25110
-3.58E-03	2.8699	0.0431	0	-4177	-237.6	4412.1	0	-2.41	7.08E-03	25553
-3.99E-03	2.871	0.0482	0	-4258	-261.2	4518.8	0	0.11	7.92E-03	25964
-4.45E-03	2.8699	0.0536	0	-4321	-295.8	4618.0	0	1.48	8.81E-03	26367
-4.95E-03	2.8669	0.0592	0	-4364	-342.6	4706.2	0	-0.34	0.009737	26757

Note: units on the table - Kip, ft, F

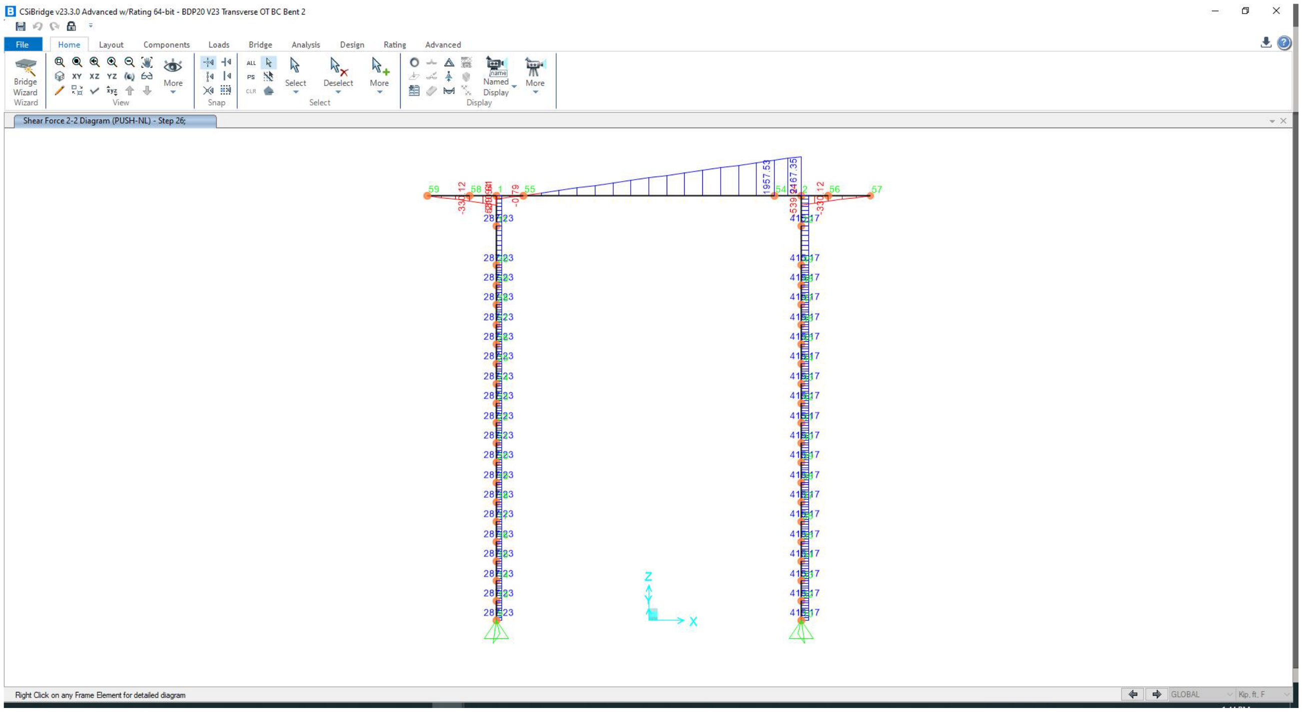
APPENDIX 20.1.3–14. Bent Cap Beam – Seismic Moment And Shear Demands

Bent 2- Bent Cap Moment and Shear Demand:

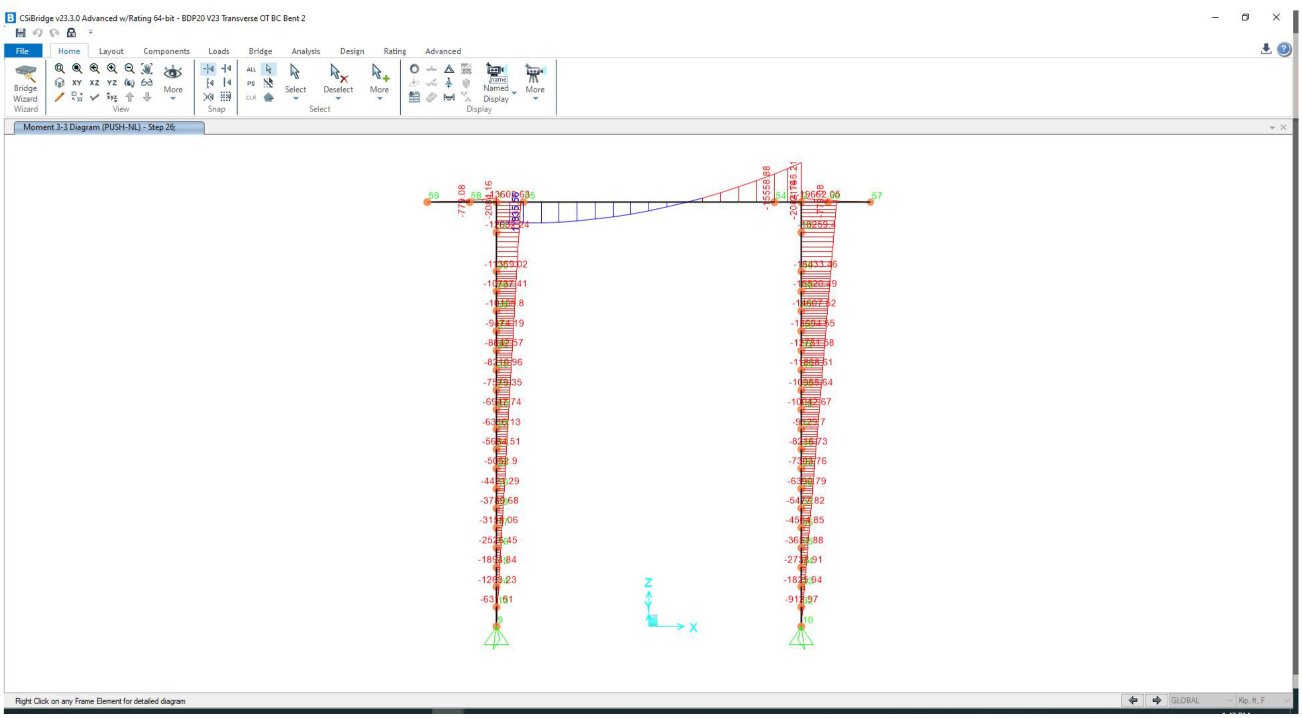


Frame Text	Station ft	OutputCase	CaseType Text	StepType Text	V2 Kip	M3 Kip-ft
54	0	PUSH-NL	NonStatic	Max	-124.553	18057.1413
54	1.5	PUSH-NL	NonStatic	Max	-19.643	18873.3263
54	3	PUSH-NL	NonStatic	Max	85.267	19532.1463
54	0	PUSH-NL	NonStatic	Min	-1188.962	-4117.3199
54	1.5	PUSH-NL	NonStatic	Min	-1084.052	-2412.5648
54	3	PUSH-NL	NonStatic	Min	-979.142	-865.1748
56	0	PUSH-NL	NonStatic	Max	2043.588	2944.9135
56	1.5	PUSH-NL	NonStatic	Max	2148.498	508.8899
56	3	PUSH-NL	NonStatic	Max	2253.408	-2084.394
56	0	PUSH-NL	NonStatic	Min	979.178	-15913.7223
56	1.5	PUSH-NL	NonStatic	Min	1084.088	-18990.8958
56	3	PUSH-NL	NonStatic	Min	1188.998	-22225.4577
57	0	PUSH-NL	NonStatic	Max	-539.937	-2084.1563
57	1.5	PUSH-NL	NonStatic	Max	-435.027	-1352.9335
57	3	PUSH-NL	NonStatic	Max	-330.117	-779.0757
57	0	PUSH-NL	NonStatic	Min	-539.937	-2084.1563
57	1.5	PUSH-NL	NonStatic	Min	-435.027	-1352.9335
57	3	PUSH-NL	NonStatic	Min	-330.117	-779.0757
59	0	PUSH-NL	NonStatic	Max	-539.937	-2084.1563
59	1.5	PUSH-NL	NonStatic	Max	-435.027	-1352.9335
59	3	PUSH-NL	NonStatic	Max	-330.117	-779.0757
59	0	PUSH-NL	NonStatic	Min	-539.937	-2084.1563
59	1.5	PUSH-NL	NonStatic	Min	-435.027	-1352.9335
59	3	PUSH-NL	NonStatic	Min	-330.117	-779.0757

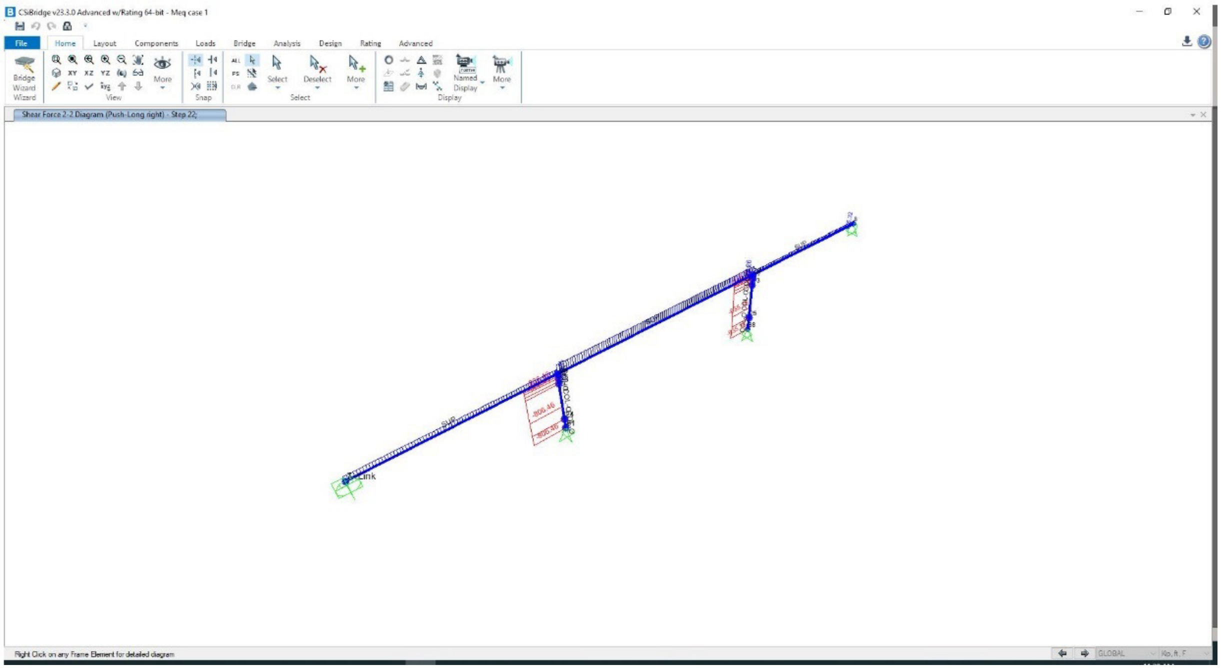
Bent Cap Shear Demand:- Screen Shot



Bent Cap Moment Demand:- Screen Shot



APPENDIX 20.1.3–15. Select Output – Superstructure Shear Forces Due To Column Hinging, Case 1



Span 1:

Element Forces - Frames

File View Edit Format-Filter-Sort Select Options

Units: As Noted

Filter:

Frame Text	Station ft	OutputCase	CaseType Text	StepType Text	V2 Kip
1	0	Push-Long r...	NonStatic	Max	129.052
1	1.6153	Push-Long r...	NonStatic	Max	129.053
1	3.2307	Push-Long r...	NonStatic	Max	129.053
1	4.846	Push-Long r...	NonStatic	Max	129.054
1	6.4614	Push-Long r...	NonStatic	Max	129.055
1	8.0767	Push-Long r...	NonStatic	Max	129.056
1	9.6921	Push-Long r...	NonStatic	Max	129.057
1	9.6921	Push-Long r...	NonStatic	Max	129.057
1	11.3074	Push-Long r...	NonStatic	Max	129.057
1	12.9227	Push-Long r...	NonStatic	Max	129.058
1	14.5381	Push-Long r...	NonStatic	Max	129.059
1	16.1534	Push-Long r...	NonStatic	Max	129.06

1	116.3047	Push-Long r...	NonStatic	Max	129.11
1	117.9206	Push-Long r...	NonStatic	Max	129.111
1	119.5364	Push-Long r...	NonStatic	Max	129.112
1	121.1523	Push-Long r...	NonStatic	Max	129.112
1	122.7682	Push-Long r...	NonStatic	Max	129.113
1	124.3841	Push-Long r...	NonStatic	Max	129.114
1	126	Push-Long r...	NonStatic	Max	129.115

Span 2:

Element Forces - Frames

File View Edit Format-Filter-Sort Select Options

Units: As Noted

Filter:

Element Forces - Frames

Frame Text	Station ft	OutputCase	CaseType Text	StepType Text	V2 Kip
11	0	Push-Long r...	NonStatic	Max	248.238
11	1.6465	Push-Long r...	NonStatic	Max	248.239
11	3.293	Push-Long r...	NonStatic	Max	248.24
11	4.9395	Push-Long r...	NonStatic	Max	248.241
11	6.586	Push-Long r...	NonStatic	Max	248.242
11	8.2326	Push-Long r...	NonStatic	Max	248.242
11	9.8791	Push-Long r...	NonStatic	Max	248.243
11	9.8791	Push-Long r...	NonStatic	Max	248.243
11	11.5261	Push-Long r...	NonStatic	Max	248.244
11	13.1732	Push-Long r...	NonStatic	Max	248.245
11	14.8202	Push-Long r...	NonStatic	Max	248.246
11	16.4673	Push-Long r...	NonStatic	Max	248.246
11	151.5261	Push-Long r...	NonStatic	Max	248.314
11	153.1732	Push-Long r...	NonStatic	Max	248.315
11	154.8202	Push-Long r...	NonStatic	Max	248.316
11	156.4673	Push-Long r...	NonStatic	Max	248.317
11	158.1144	Push-Long r...	NonStatic	Max	248.317
11	158.1144	Push-Long r...	NonStatic	Max	248.317
11	159.762	Push-Long r...	NonStatic	Max	248.318
11	161.4096	Push-Long r...	NonStatic	Max	248.319
11	163.0572	Push-Long r...	NonStatic	Max	248.32
11	164.7048	Push-Long r...	NonStatic	Max	248.321
11	166.3524	Push-Long r...	NonStatic	Max	248.322
11	168	Push-Long r...	NonStatic	Max	248.322

Span 3:

Element Forces - Frames

File View Edit Format-Filter-Sort Select Options

Units: As Noted

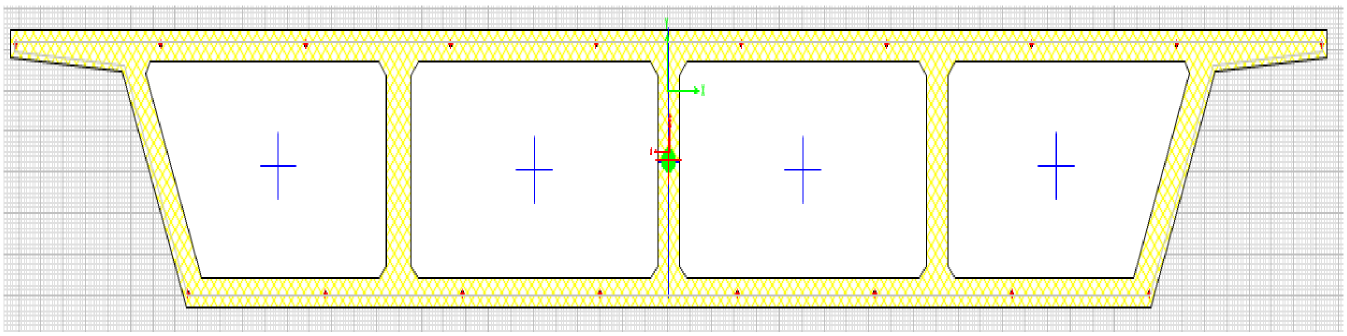
Filter:

Element Forces - Frames

Frame Text	Station ft	OutputCase	CaseType Text	StepType Text	V2 Kip
20	0	Push-Long r...	NonStatic	Max	108.451
20	1.6384	Push-Long r...	NonStatic	Max	108.452
20	3.2768	Push-Long r...	NonStatic	Max	108.453
20	4.9152	Push-Long r...	NonStatic	Max	108.454
20	6.5536	Push-Long r...	NonStatic	Max	108.454
20	8.1919	Push-Long r...	NonStatic	Max	108.455
20	9.8303	Push-Long r...	NonStatic	Max	108.456
20	9.8303	Push-Long r...	NonStatic	Max	108.456
20	11.4693	Push-Long r...	NonStatic	Max	108.457
20	13.1082	Push-Long r...	NonStatic	Max	108.458
20	14.7471	Push-Long r...	NonStatic	Max	108.459
20	16.3861	Push-Long r...	NonStatic	Max	108.459
20	18.025	Push-Long r...	NonStatic	Max	108.46
20	109.8053	Push-Long r...	NonStatic	Max	108.506
20	111.4443	Push-Long r...	NonStatic	Max	108.507
20	113.0832	Push-Long r...	NonStatic	Max	108.508
20	114.7221	Push-Long r...	NonStatic	Max	108.509
20	116.3611	Push-Long r...	NonStatic	Max	108.509
20	118	Push-Long r...	NonStatic	Max	108.51

APPENDIX 20.1.3–16. *CSiBridge* Section-Designer Model For Superstructure Flexural Capacity

Span 1 at 1.5 ft Superstructure model is shown as an example



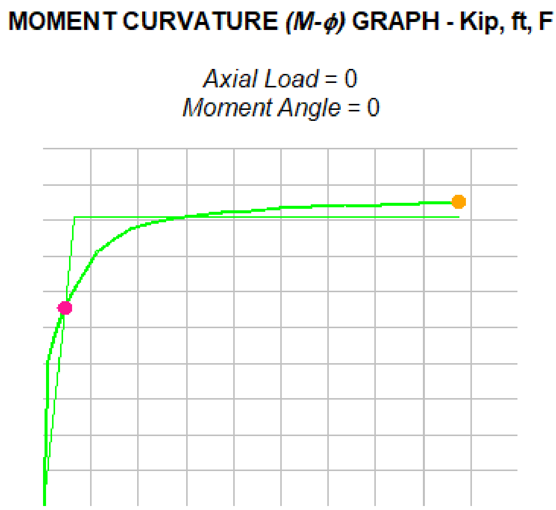
Note: Green dot is the location of the prestressed cable and varies along the span length

See Appendix 20.1.3-1 for Material input properties for section Designer which is the expected material properties per *SDC* Table 3.3.3-1 and Table 3.3.6-1.

APPENDIX 20.1.3–17. Superstructure Flexural Capacity Using *CSiBridge* Section-Designer

Positive and Negative Moment Capacity of Superstructure for Span 1 @ 1.5 FT is shown below:

(a) Positive Moment Capacity:



Results For Exact-Integration

Initial Prestress State
P = 6141
M_{33} = -788.0243
M_{22} = -361.5263
ε_o = 1.066E-04
ϕ_{33o} = -1.930E-06
ϕ_{22o} = -2.546E-08

$\phi_{y(Initial)}$ = 4.403E-04
M_y = 22217
$\phi_{y(Idealized)}$ = 6.413E-04
M_p = 32361
I_{crack} = 80.1514
$\phi_{concrete}$ = N/A
$M_{concrete}$ = N/A
ϕ_{steel} = 8.740E-03
M_{steel} = 34012

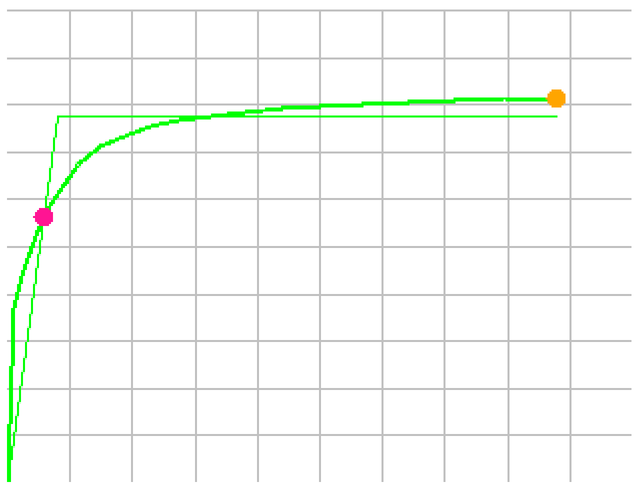
Concrete Strain	Neutral Axis	Steel Strain	Tendon Strain	Concrete Comp	Steel Comp	Steel Tension	Prestress Force	Net Force	Curvature	Moment
-1.00E-04	54.862	0.000	0.006	-6096	-45.29	0.00	6141	0.03	0	0.1453
-2.53E-04	-0.149	0.000	0.006	-6251	-54.45	49.58	6255	-0.79	8.36E-05	16447
-3.37E-04	1.321	0.001	0.006	-6707	-64.43	181.64	6584	-5.40	0.0002091	18899
-4.33E-04	1.791	0.002	0.006	-7328	-73.84	360.68	7041	-0.72	0.0003764	21519
-5.42E-04	2.017	0.003	0.007	-7969	-83.18	429.76	7622	-0.01	0.0005855	23803
-6.68E-04	2.145	0.005	0.008	-8655	-93.10	429.76	8326	6.85	0.0008364	25994
-8.18E-04	2.220	0.006	0.008	-9470	-105.27	429.76	9144	-1.79	0.001129	28515
-9.60E-04	2.289	0.008	0.009	-9874	-112.16	436.39	9549	-0.44	0.001464	29909
-1.10E-03	2.347	0.011	0.010	-10152	-115.48	452.90	9809	-5.13	0.00184	30950
-1.24E-03	2.396	0.013	0.011	-10330	-115.00	471.37	9973	-0.29	0.002258	31686
-1.38E-03	2.437	0.016	0.013	-10446	-111.60	491.78	10062	-3.85	0.002718	32213
-1.53E-03	2.471	0.019	0.014	-10521	-105.44	509.40	10114	-3.30	0.00322	32584
-1.68E-03	2.501	0.022	0.016	-10579	-96.65	519.45	10155	-1.84	0.003764	32859
-1.83E-03	2.526	0.026	0.017	-10632	-85.27	530.30	10185	-2.22	0.004349	33101
-1.98E-03	2.547	0.030	0.019	-10680	-71.71	540.95	10207	-3.75	0.004976	33306
-2.15E-03	2.566	0.034	0.021	-10722	-56.14	550.48	10224	-3.58	0.005645	33477
-2.31E-03	2.582	0.038	0.023	-10763	-38.52	560.62	10238	-3.06	0.006356	33633
-2.49E-03	2.596	0.043	0.025	-10803	-18.85	569.89	10249	-2.95	0.007109	33771
-2.67E-03	2.608	0.048	0.027	-10843	-0.04	581.92	10259	-2.57	0.007904	33898
-2.86E-03	2.619	0.053	0.030	-10882	0.00	613.87	10267	-1.66	0.00874	34012

Note: units on the table - Kip, ft, F

(b) *Negative Moment Capacity:*

MOMENT CURVATURE *(M-ϕ)* GRAPH - Kip, ft, F

Axial Load = 0
Moment Angle = 180



Initial Prestress State
P = 6141
M_{33} = -788.0243
M_{22} = -361.5263
ε_o = 1.066E-04
ϕ_{33o} = -1.930E-06
ϕ_{22o} = -2.546E-08

$\phi_{y(Initial)}$ = 4.602E-04
M_y = 28225
$\phi_{y(Idealized)}$ = 6.309E-04
M_p = 38691
I_{crack} = 97.4174
$\phi_{concrete}$ = N/A
$M_{concrete}$ = N/A
ϕ_{steel} = 7.037E-03
M_{steel} = 40685

Concrete Strain	Neutral Axis	Steel Strain	Tendon Strain	Concrete Comp	Steel Comp	Steel Tension	Prestress Force	Net Force	Curvature	Moment
-1.13E-04	-54.862	0.000	0.006	-6096	-45.29	0.00	6141	0.03	0	0.1453
-3.09E-04	-0.658	0.000	0.006	-6179	-52.46	32.43	6198	-1.13	6.73E-05	18596
-4.11E-04	1.392	0.001	0.006	-6592	-64.83	164.16	6492	-0.89	0.0001683	22034
-5.12E-04	2.125	0.001	0.006	-7195	-75.06	348.04	6921	-1.54	0.000303	25052
-6.26E-04	2.481	0.002	0.007	-7952	-85.54	565.76	7471	-0.22	0.0004714	28451
-7.49E-04	2.695	0.004	0.007	-8617	-95.50	565.76	8147	0.06	0.0006734	30880
-8.93E-04	2.824	0.005	0.008	-9394	-107.28	565.76	8935	-0.54	0.0009091	33656
-1.04E-03	2.927	0.007	0.009	-9940	-116.65	565.76	9491	-0.74	0.001178	35673
-1.17E-03	3.016	0.008	0.010	-10192	-122.35	573.81	9741	0.67	0.001481	36735
-1.31E-03	3.083	0.010	0.011	-10416	-127.86	593.08	9951	0.74	0.001818	37733
-1.46E-03	3.137	0.013	0.012	-10517	-132.29	614.37	10035	0.90	0.002189	38319
-1.62E-03	3.181	0.015	0.014	-10601	-135.74	637.67	10099	0.14	0.002593	38828
-1.78E-03	3.218	0.018	0.015	-10673	-137.81	662.99	10145	-2.96	0.00303	39277
-1.94E-03	3.251	0.021	0.017	-10713	-138.20	676.22	10175	-0.02	0.003502	39559
-2.11E-03	3.279	0.024	0.018	-10748	-137.71	688.39	10198	1.08	0.004007	39802
-2.28E-03	3.303	0.027	0.020	-10782	-136.27	701.39	10217	0.66	0.004545	40025
-2.46E-03	3.324	0.030	0.022	-10812	-133.77	713.50	10232	0.18	0.005118	40221
-2.65E-03	3.342	0.034	0.024	-10839	-130.52	724.74	10245	-0.11	0.005724	40390
-2.84E-03	3.358	0.038	0.026	-10865	-126.64	736.62	10255	-0.31	0.006364	40546
-3.04E-03	3.371	0.042	0.029	-10889	-122.04	747.67	10263	-0.44	0.007037	40685

Note: units on the table - Kip, ft, F

NOTATION

A_b = area of individual reinforcing steel bar (in.^{2})
A_{cap}^{top} = area of bent cap top flexural steel (in.^{2})
A_{cap}^{bot} = area of bent cap bottom flexural steel (in.^{2})
A_{cv} = area of concrete engaged in interface shear transfer (in.^{2})
A_e = effective shear area; effective abutment wall area (in.^{2})
A_g = gross cross section area (in.^{2})
A_{jh} = effective horizontal area of a moment resisting joint (in.^{2})
A_{jv} = effective vertical area for a moment resisting joint (in.^{2})
A_{ps} = prestressing steel area (in.^{2})
A_s = area of supplemental non-prestressed tension reinforcement (in.^{2})
A_s^{jh} = area of horizontal joint shear reinforcement required at moment resisting joints (in.^{2})
A_s^{jhc} = total area of horizontal ties placed at the end of the bent cap in Case 1 Knee joints (in.^{2})
A_s^{jv} = area of vertical joint shear reinforcement required at moment resisting joints (in.^{2})
A_s^{j-bar} = area of vertical "J" bar reinforcement required at moment resisting joints with a skew angle > 20° (in.^{2})
A_s^{sf} = area of bent cap side face steel required at moment resisting joints (in.^{2})
A_{sk} = area of interface shear reinforcement crossing the shear plane (Vertical shear key reinforcement) (in.^{2})
$A_{st,\max}$ = maximum longitudinal reinforcement area (in.^{2})
$A_{st,\min}$ = minimum longitudinal reinforcement area (in.^{2})
A_{st} = total area of column longitudinal reinforcement anchored in the joint; total area of column/pier wall longitudinal reinforcement (in.^{2})
A_s^{u-bar} = area of bent cap top and bottom reinforcement bent in the form of "U" bars in Knee joints (in.^{2})
A_{sh} = area of horizontal shear key reinforcement (hanger bars) (in.^{2})
$A_{sk(provided)}^{Iso}$ =area of interface shear reinforcement provided for isolated shear key (in.^{2})
$A_{sk(provided)}^{Non-iso}$ =area of interface shear reinforcement provided for non-isolated shear key (in.^{2})
A_v = area of shear reinforcement perpendicular to flexural tension reinforcement (in.^{2})
a = demand spectral acceleration
B_{cap} = bent cap width (in.)
B_{eff} = effective width of the superstructure for resisting longitudinal seismic moments (in.)
b_v = effective web width taken as the minimum web width within the shear depth d_v (in.)

D_c = column cross sectional dimension in the direction of interest (in.)

D_{ftg} = depth of footing (in.)

D_s = depth of superstructure at the bent cap (in.)

DSH = *Design Seismic Hazards*

D' = cross-sectional dimension of confined concrete core measured between the centerline of the peripheral hoop or spiral (in.)

D'_c = confined column cross-section dimension, measured out to out of ties, in the direction parallel to the axis of bending (in.)

d_{bl} = nominal bar diameter of longitudinal column reinforcement (in.^{2})

d_v = effective shear depth defined as the distance between resultants of tensile and compressive forces due to flexural, but need not be taken less than $0.9d_e$ or 0.72h (in.)

E_c = modulus of elasticity of concrete (ksi)

EDA = *Elastic Dynamic Analysis*

ESA = *Elastic Static Analysis*

F1, *F2* = concrete shear factors for SCMs.

F_{abut} = idealized ultimate passive capacity of the backfill behind abutment backwall or diaphragm (kip)

F_{sk} = abutment shear key force capacity; Shear force associated with column overstrength moment, including overturning effects (ksi)

f_h = average normal stress in the horizontal direction within a moment resisting joint (ksi)

f_v = average normal stress in the vertical direction within a moment resisting joint (ksi)

f_y = nominal yield stress for A706 reinforcement (ksi)

f_{ye} = expected yield stress for A706 reinforcement (ksi)

f_{yh} = nominal yield stress of transverse column reinforcement, hoops/spirals (ksi)

f'_c = compressive strength of unconfined concrete (psi)

f'_{cc} = confined compression strength of concrete (psi)

f'_{ce} = expected compressive strength of unconfined concrete (psi)

g = acceleration due to gravity, $32.2\,\text{ft}/\text{sec}^2$

h = distance from the center of gravity of the tensile force to the center of gravity of the compressive force of the column section (in.)

h_{abut} = height of the backwall or diaphragm for seat and diaphragm abutments, respectively (ft)

h_{dia} = backwall height for diaphragm abutment (in.)

h_{bw} = backwall height for seat abutment (in.)

I_{eff} , I_e = effective moment of inertia for computing member stiffness (in.^{4})

ISA = *Inelastic Static Analysis*

K = effective stiffness of the bent or frame

K_{abut} = abutment backwall stiffness (kip/in./ft)

K_{eff} = effective abutment backwall stiffness (kip/in./ft)

K_i = Initial abutment backwall stiffness (kip/in./ft)

k_i^e , k_j^e = smaller and larger effective bent or column stiffness, respectively (kip/in.)

L = member length from the point of maximum moment to the point of contra-flexure (in); length of bridge deck between adjacent expansion joints

$L_{\min,headed}$ = minimum horizontal length from the end of the lowest layer of headed hanger bar to the intersection with the shear key vertical reinforcement (in.)

$L_{\min,hooked}$ = minimum horizontal length from the end of the lowest layer of hanger bar hooks to the intersection with the shear key vertical reinforcement (in.)

L_p = equivalent analytical plastic hinge length (in.)

l_{ac} = minimum length of column longitudinal reinforcement extension into the bent cap (in.)

$l_{ac,provided}$ = actual length of column longitudinal reinforcement embedded into the bent cap (in.)

l_d = development length of the main reinforcement (in.)

l_{dh} = development length in tension of standard hooked bars (in.)

M_{dl} = moment attributed to dead load (kip-ft)

M_{eq}^{col} = column moment when coupled with any existing M_{dl} & $M_{p/s}$ will equal the column's overstrength moment capacity, M_o^{col} (kip-ft)

$M_{eq}^{R,L}$ = portion of M_{eq}^{col} distributed to the left or right adjacent superstructure spans (kip-ft)

M_n = nominal moment capacity based on the nominal concrete and steel strengths when the concrete strain reaches 0.003 (kip-ft)

M_{ne} = nominal moment capacity based on the expected material properties and a concrete strain, $\varepsilon_c = 0.003$ (kip-ft)

$M_{ne}^{\sup R,L}$ = expected nominal moment capacity of the right and left superstructure spans utilizing expected material properties (kip-ft)

M_o^{col} = column overstrength moment (kip-ft)

M_p^{col} = Idealized plastic moment capacity of a column calculated by M-ϕ analysis (kip-ft)

$M_{p/s}$ = moment attributed to secondary prestress effects (kip-ft)

M_y = Moment capacity of a ductile component corresponding to the first reinforcing bar yielding (kip-ft)

M-ϕ = moment curvature analysis

m_i = tributary mass of column or bent i, $m = W/g$ (kip-sec^2/ft)

m_j = tributary mass of column or bent *j,* m = W/g (kip-sec^2/ft)
N_H = minimum hinge seat width normal to the centerline of bent (in.)
N_A = abutment support width normal to centerline of bearing (in.)
P = absolute value of the net axial force normal to the shear plane (kip)
P_b = beam axial force at the center of the joint including prestressing (kip)
P_c = column axial force including the effects of overturning (kip)
P_{dia} = passive pressure force resisting movement at diaphragm abutment (ksf)
P_{dl} = superstructure dead load reaction at the abutment plus weight of the abutment and its footing (kip)
P_{dl}^{sup} = superstructure axial load resultant at the abutment (kip)
P/S = prestressed concrete; prestressing strand
P_{jack} = total prestress jacking force (kip)
P_n = nominal axial resistance (kip)
P_{bw} = passive pressure force resisting movement at seat abutment (ksf)
p_{bw} = maximum abutment backwall soil pressure (ksf)
p_c = nominal principal compression stress in a joint (psi)
p_t = nominal principal tension stress in a joint (psi)
R_A = abutment displacement coefficient
R_{sk} = skew reduction factor
S = cap beam short stub length (ft)
S_a = design spectral acceleration coefficient at the structure period
SCM = Seismic Critical Member
SDC = Seismic Design Criteria
s = spacing of shear/transverse reinforcement (in.)
T = natural period of vibration, (seconds), $T = 2\pi\sqrt{m/k}$
T_c = total tensile force in column longitudinal reinforcement associated with M_o^{col} (kip)
T_i = natural period of the stiffer frame (sec.)
T_j = natural period of the more flexible frame (sec.)
t = top or bottom slab thickness (in.)
V_c = nominal shear strength provided by concrete (kip)
V_n = nominal shear strength (kip)
V_o = overstrength shear associated with the overstrength moment M_o (kip)
V_o^{col} = column overstrength shear, typically defined as M_o^{col}/L (kip)
V_p^{col} = column plastic shear, typically defined as M_p^{col}/L (kip)
V_s = nominal shear strength provided by shear reinforcement (kip)

V_{ww} = shear capacity of one wingwall (kip)

v_{jv} = nominal vertical shear stress in a moment resisting joint (psi)

v_c = permissible shear stress carried by concrete (psi)

W = tributary weight of the structure

w_{abut} = width of the backwall or diaphragm, as appropriate (in.)

α = factor defining the range over which F_{sk} is allowed to vary

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear

ε_{su}^{R} = reduced ultimate tensile strain for A706 reinforcement

Δ_{abut} = abutment displacement at idealized yield (in.)

Δ_c = local member displacement capacity (in.)

Δ_{col} = displacement attributed to the elastic and plastic deformation of the column (in.)

Δ_C = global displacement capacity (in.)

Δ_{cr+sh} = displacement due to creep and shrinkage (in.)

Δ_d = local member displacement demand (in.)

Δ_D = global system displacement (in.)

Δ_{eff} = effective longitudinal abutment displacement at idealized yield (in.)

Δ_{eq} = relative longitudinal displacement demand at an expansion joint due to earthquake (in.)

Δ_{gap} = width of expansion gap at seat abutment (in.)

Δ_p = idealized plastic displacement capacity due to rotation of the plastic hinge (in.)

$\Delta_{p/s}$ = displacement due to prestress shortening (in.)

Δ_r = relative lateral offset between the point of contra-flexure and the base of the plastic hinge (in.)

Δ_{tem} = displacement due to temperature variation (in.)

Δ_Y = idealized yield displacement of the subsystem at the formation of the plastic hinge (in.)

$\Delta_{Y(i)}$ = idealized yield displacement of the subsystem at the formation of plastic hinge (*i*) (in.)

Δ_Y^{col} = idealized yield displacement of a column at the formation of the plastic hinge (in.)

θ = angle of inclination of diagonal compressive stresses (radians); abutment skew angle (degree)

θ_p = plastic rotation capacity (radians)

θ_{sk} = skew angle (degree)

ρ_s = amount of transverse reinforcement expressed as volumetric ratio

ϕ = resistance factor

ϕ_p = idealized plastic curvature (1/in.)

ϕ_u = ultimate curvature capacity (1/in.)

ϕ_y = yield curvature corresponding to the first yield of the reinforcement in a ductile component (1/in.)

ϕ_Y = idealized yield curvature (1/in.)

μ_d = local displacement ductility demand

μ_D = global displacement ductility demand

μ_c = local displacement ductility capacity

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