Bridge Design - Seismic Design of Concrete Bridges

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Division of Engineering Services

BRIDGE DESIGN PRACTICE

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CHAPTER 21
SEISMIC DESIGN OF CONCRETE BRIDGES

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CHAPTER 21
SEISMIC DESIGN OF CONCRETE BRIDGES

21.1 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 1.7 (Caltrans 2013). Information presented herein is based on *SDC* (Caltrans 2013), AASHTO *LRFD Bridge Design Specifications* (AASHTO 2012) with *California Amendments* (Caltrans 2014), and other Caltrans Structure Design documents such as *Bridge Memo to Designers* (*MTD*). 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 21.2, 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 21.3, 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 1.7 provisions.

21.2 DESIGN CONSIDERATIONS

21.2.1 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, if necessary, to refine the design of various components to satisfy Extreme Events Limit States such as seismic performance requirements. In practice, however, 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 configuration, column size, column reinforcement requirements, and bent cap width.

21.2.1.1 Sizing the Column and Bent Cap

(1) Column size

SDC Section 7.6.1 specifies that the column size should satisfy the following equations:

$$0.70 \le \frac{D_c}{D_s} \le 1.00 \qquad (SDC\ 7.6.1\text{-}1)$$

$$0.7 \le \frac{D_{ftg}}{D_c} \qquad (SDC\ 7.6.1\text{-}2)$$

where:

D_c = column cross sectional dimension in the direction of interest (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 meet the joint shear, superstructure capacity, and ductility requirements.

(2) Bent Cap Width

SDC Section 7.4.2.1 specifies the minimum cap width required for adequate joint shear transfer as follows:

$$B_{cap} = D_c + 2 \quad \text{(ft)} \qquad (SDC\ 7.4.2.1\text{-}1)$$

21.2.1.2 Column Reinforcement Requirements

(1) Longitudinal Reinforcement

Maximum Longitudinal Reinforcement Area, $A_{st,\max} = 0.04 \times A_g$ (*SDC* 3.7.1-1)

Minimum Longitudinal Reinforcement Area:

$$A_{st,\min} = 0.01(A_g) \quad \text{for columns} \qquad (SDC\ 3.7.2\text{-}1)$$

$$A_{st,\min} = 0.005(A_g) \quad \text{for Pier walls} \qquad (SDC\ 3.7.2\text{-}2)$$

where:

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

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

(2) Transverse Reinforcement

Either spirals or hoops can be used as transverse reinforcement in the column. However, hoops are preferred (see *MTD* 20-9) 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 7.6.2 for analytic 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 4.1.

$$\rho_s = \frac{4(A_b)}{D'(s)}$$ for columns with circular or interlocking cores (*SDC* 3.8.1-1)

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

$$\rho_s = \frac{A_v}{D_c' s} \qquad (SDC\ 3.8.1\text{-}2)$$

where:

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

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

s = transverse reinforcement spacing (in.)

In addition, the transverse reinforcement should meet the column shear requirements as specified in *SDC* Section 3.6.3.

- Outside the Plastic Hinge Region

As specified in *SDC* Section 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 layout should satisfy the following spacing requirements for effectiveness and constructability:

- Longitudinal Reinforcement
  Maximum and minimum spacing requirements are given in *AASHTO* Article 5.10 (2012).

- Transverse Reinforcement
  According to *SDC* Section 8.2.5, the maximum spacing in the plastic hinge region shall not exceed the smallest of:

- $\frac{1}{5}$ of the least column cross-section dimension for columns and ½ of the least cross-section dimension for piers
- 6 times the nominal diameter of the longitudinal bars
- 8 in.

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

21.2.1.3 Balanced Stiffness

(1) Stiffness Requirements

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. To this end, the *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:

$$\frac{k_i^e}{k_j^e} \geq 0.5 \qquad \text{For constant width frame} \qquad (SDC\ 7.1.1\text{-}1)$$

$$2 \geq \left(\frac{k_i^e / m_i}{k_j^e / m_j} \right) \geq 0.5 \qquad \text{For variable width frame} \qquad (SDC\ 7.1.1\text{-}2)$$

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:

$$\frac{k_i^e}{k_j^e} \geq 0.75 \qquad \text{For constant width frame} \qquad (SDC\ 7.1.1\text{-}3)$$

$$1.33 \geq \left(\frac{k_i^e / m_i}{k_j^e / m_j} \right) \geq 0.75 \qquad \text{For variable width frame} \qquad (SDC\ 7.1.1\text{-}4)$$

where:

k_i^e = smaller effective bent or column stiffness (kip/in.)
m_i = tributary mass of column or bent i (kip-sec^2/ft)
k_j^e = larger effective bent or column stiffness (kip/in.)
m_j = tributary mass of column or bent j (kip-sec^2/ft)

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

If these requirements of 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 will 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 ($M-\phi$) relationship are determined by using a computer program such as *xSECTION* (Mahan 2006) or similar tool. Effort should be made to keep the dead load axial forces in columns to about 10% of their ultimate compressive capacity ($P_u = A_g f_c'$) 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. When the column axial load ratio starts approaching 15%, increasing the column size or adding an extra column should be considered.

Material Properties

- Concrete

  Concrete compressive stress f_c' = 4,000 psi is commonly used for superstructure, columns, piers, and pile shafts. For other components like abutments, wingwalls, and footings, f_c' =3,600 psi is typically specified.

  SDC Section 3.2 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 will also be used to calculate member stiffness. For concrete, the expected compressive strength, f_{ce}', is taken as:

$$f_{ce}' = \text{Greater of} \begin{pmatrix} 1.3(f_c') \\ \text{and} \\ 5{,}000\,\text{psi} \end{pmatrix} \qquad (SDC\ 3.2.6\text{-}3)$$

  Other concrete properties are listed in *SDC* Section 3.2.6.

- Steel

  Grade A706/A706M is typically used for reinforcing steel bar. Material properties for Grade A706/A706M steel are given in *SDC* Section 3.2.3.

Effective Moment of Inertia

It is well known that concrete cover spalls off at very low ductility levels. Therefore, the effective (cracked) moment of inertia values are used to assess the seismic response of all ductile members. This is obtained from a moment-curvature analysis of the member cross-section.

21.2.1.4 Balanced Frame Geometry

SDC Section 7.1.2 requires that the ratio of fundamental periods of vibration for adjacent frames in the longitudinal and transverse directions satisfy:

$$\frac{T_i}{T_j} \geq 0.7 \qquad (SDC\ 7.1.2\text{-}1)$$

where:

T_i = natural period of the less flexible frame (sec.)
T_j = natural period of the more flexible frame (sec.)

The consequences of not meeting the fundamental period requirements of *SDC* Equation 7.1.2-1 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 collision 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 7.1.3) may be employed to adjust the dynamic characteristics:

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

If the column reinforcement exceeds the preferred maximum, the following additional revisions as outlined in *MTD* 6-1 (Caltrans 2009) may help:

- Pin columns in multi-column bents and selected single columns adjacent to abutments at their bases
- Use higher strength concrete
- Shorten spans and add bents
- Use pile shafts in lieu of footings
- Add more columns per bent

21.2.2 Minimum Local Displacement Ductility Capacity

Before undertaking 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 good practice to ensure that basic *SDC* ductility requirements are met. *SDC* Section 3.1 requires that each ductile member shall have a minimum local displacement ductility capacity μ_c of 3 to ensure dependable rotational capacity in the plastic hinge regions regardless of the displacement demand imparted to the member.

$$\Delta_c = \Delta_Y^{col} + \Delta_p \qquad (SDC\ 3.1.3\text{-}1)$$

$$\Delta_Y^{col} = \frac{L^2}{3}(\phi_Y) \qquad (SDC\ 3.1.3\text{-}2)$$

$$\Delta_p = \theta_p\left(L - \frac{L_p}{2}\right) \qquad (SDC\ 3.1.3\text{-}3)$$

$$\theta_p = L_p\phi_p \qquad (SDC\ 3.1.3\text{-}4)$$

$$\phi_p = \phi_u - \phi_Y \qquad (SDC\ 3.1.3\text{-}5)$$

$$\Delta_{c1} = \Delta_{Y1}^{col} + \Delta_{p1}; \quad \Delta_{c2} = \Delta_{Y2}^{col} + \Delta_{p2} \qquad (SDC\ 3.1.3\text{-}6)$$

$$\Delta_{Y1}^{col} = \frac{L_1^2}{3}(\phi_{Y1}); \quad \Delta_{Y2}^{col} = \frac{L_2^2}{3}(\phi_{Y2}) \qquad (SDC\ 3.1.3\text{-}7)$$

$$\Delta_{p1} = \theta_{p1}\left(L_1 - \frac{L_{p1}}{2}\right); \quad \Delta_{p2} = \theta_{p2}\left(L_2 - \frac{L_{p2}}{2}\right) \qquad (SDC\ 3.1.3\text{-}8)$$

$$\theta_{p1} = L_{p1}\phi_{p1}; \quad \theta_{p2} = L_{p2}\phi_{p2} \qquad (SDC\ 3.1.3\text{-}9)$$

$$\phi_{p1} = \phi_{u1} - \phi_{Y1}; \ \phi_{p2} = \phi_{u2} - \phi_{Y2} \qquad (SDC\ 3.1.3\text{-}10)$$

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 7.6.2 (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 representation of the cross section's M-ϕ curve, see *SDC* Figures 3.3.1-1 and 3.3.1-2 (rad/in.)

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

θ_p = plastic rotation capacity (radian)

ϕ_u = curvature capacity at the Failure Limit State, defined as the concrete strain reaching ε_{cu} or the longitudinal reinforcing steel reaching the reduced ultimate strain ε_{su}^R (rad/in.)

It is Caltrans' practice to use an idealized bilinear M-ϕ curve to estimate the idealized yield displacement and deformation capacity of ductile members.

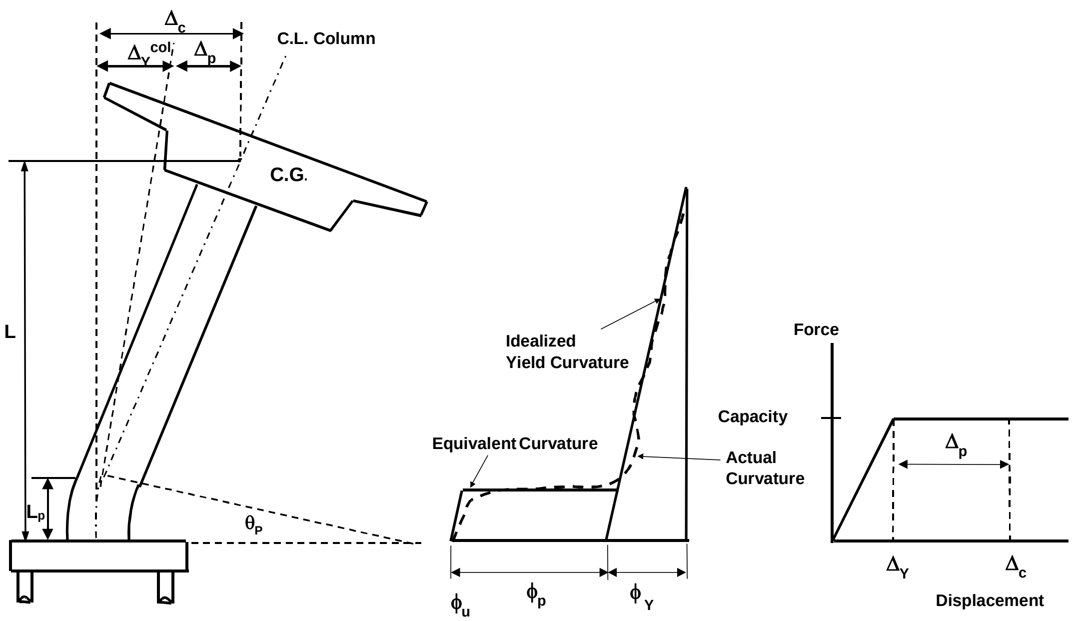


Figure *SDC* 3.1.3-1 Local Displacement Capacity – Cantilever Column with Fixed Base

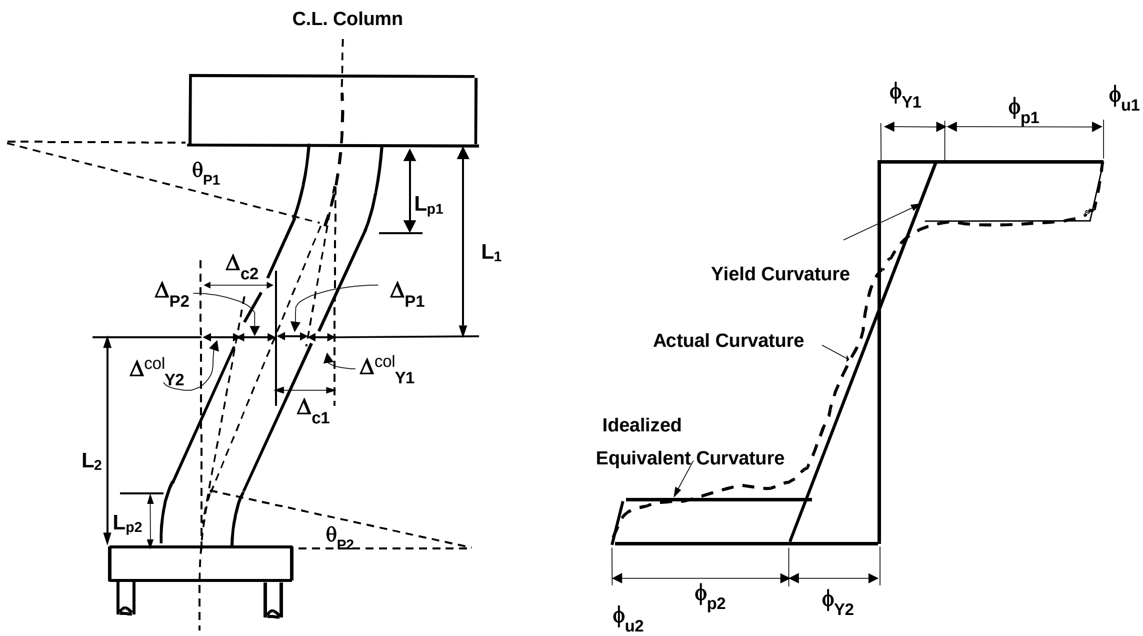


Figure *SDC* 3.1.3-2 Local Displacement Capacity – Framed Column, Assumed as Fixed-Fixed

21.2.3 Displacement Ductility Demand Requirements

The displacement ductility demand is mathematically defined as

$$\mu_D = \frac{\Delta_D}{\Delta_{Y(i)}} \qquad (SDC\ 2.2.3\text{-}1)$$

where:

Δ_D = the estimated global/frame displacement demand

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

To reduce the required strength of ductile members and minimize the demand imparted to adjacent capacity protected components, *SDC* Section 2.2.4 specifies target upper limits of displacement ductility demand values, μ_D, for various bridge components.

Single Column Bents supported on fixed foundation	$\mu_D \leq 4$
Multi-Column Bents supported on fixed or pinned footings	$\mu_D \leq 5$
Pier Walls (weak direction) supported on fixed or pinned footings	$\mu_D \leq 5$
Pier Walls (strong direction) supported on fixed or pinned footings	$\mu_D \leq 1$

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

$$\Delta_D < \Delta_C \qquad (SDC\ 4.1.1\text{-}1)$$

where:

Δ_C = the bridge or frame displacement capacity when the first ultimate capacity is reached by any plastic hinge (in.)

Δ_D = the displacement generated from the global analysis, stand-alone analysis, or the larger of the two if both types of analyses are necessary (in.)

The seismic demand can be estimated using Equivalent Static Analysis (ESA). As described in *SDC* Section 5.2.1, 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 realistic values for the structure's period and demand.

The displacement demand, Δ_D, can be calculated from Equation 21.2-1.

$$\Delta_D = \frac{ma}{k_e} \qquad (21.2\text{-}1)$$

where:

m = tributary superstructure mass on the bent/frame
a = demand spectral acceleration
k_e = effective frame stiffness

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 5.2.2 for more details regarding EDA.

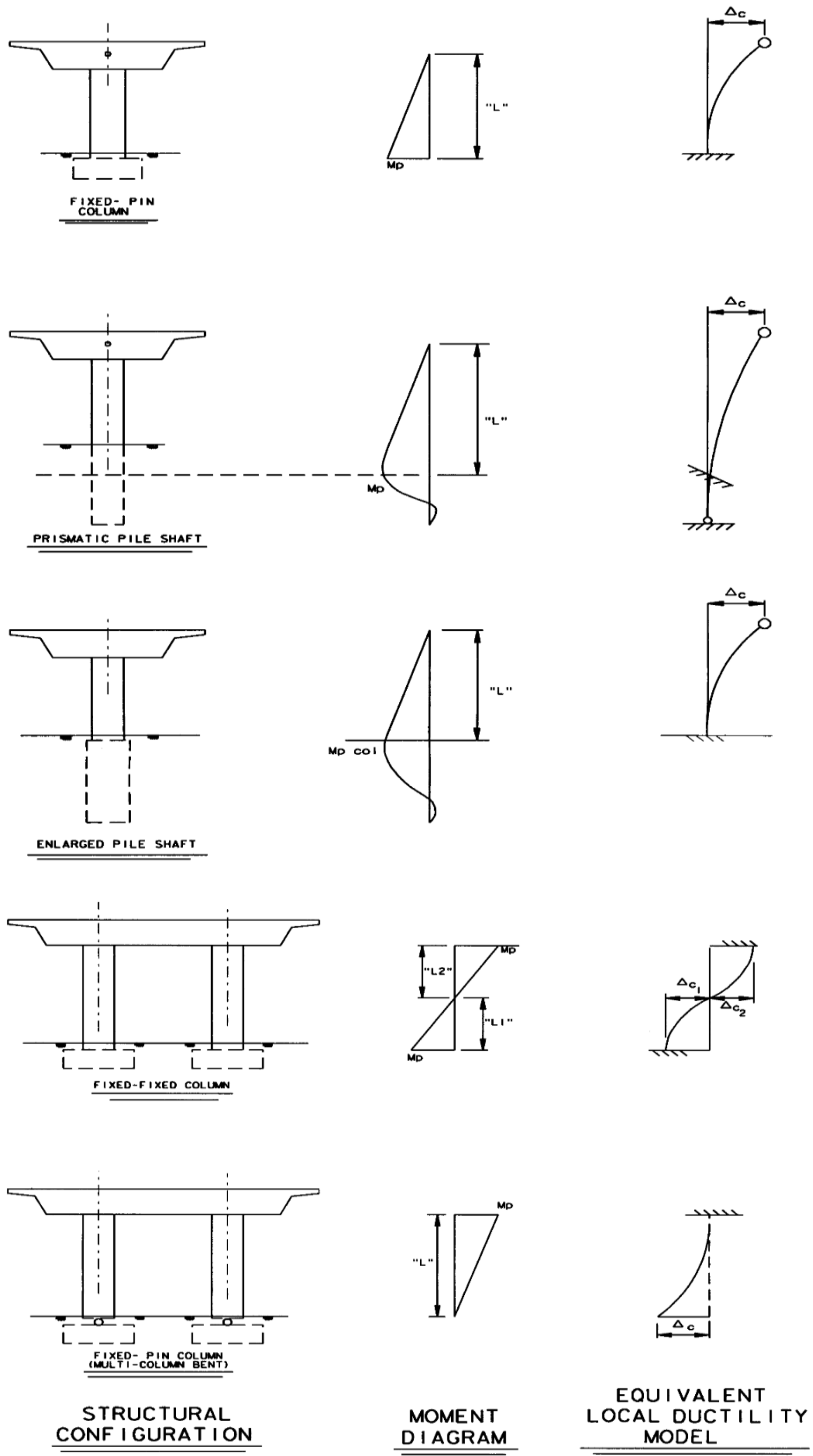


Figure *SDC* 3.1.4.1-1 Local Ductility Assessment

21.2.4 Displacement Capacity Evaluation

SDC Section 5.2.3 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 response such as yielding of ductile components and effects of surrounding soil as well as the effects of foundation flexibility and flexibility of capacity protected components such as bent caps. 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 expected material properties of modeled members to provide a more realistic estimate of design strength. As required by *SDC* Section 3.4, 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 these components.

Caltrans' in-house computer program *wFRAME* (Mahan 1995) or similar tool may be used to perform pushover analysis. If *wFRAME* program is used, the following conventions are applicable to both the transverse and longitudinal analyses:

- The model is two-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 fixed connection, the plastic hinge forms at the top of the column below the superstructure soffit.
- The soil 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 21.2.3.

21.2.4.1 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 proper soil springs within the push analysis. The spacing of the nodes selected on the pile members would naturally change the values of 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. This spring may be idealized as a bi-linear spring placed in the boundary condition section of the push analysis input file.

21.2.4.2 Transverse Pushover Analysis

During the transverse movement of a multi-column frame, a strong 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. The effect of change in the axial force in a column section due to overturning moments can be summarized as follows:

- M_p changes
- The tension column(s) will become more ductile while the compression column(s) will become less ductile.
- The required flexural capacity of cap beam that is needed to make sure that the hinge forms at column top will obviously become larger.

With the changes in column axial loads, the section properties (M_p and I_e) should be updated and a second iteration of the *wFRAME* program performed if using *wFRAME* for the analysis.

The effective bent cap width to be used for the pushover analysis is calculated as follows:

$$B_{eff} = B_{cap} + (12t) \qquad (SDC\ 7.3.1.1\text{-}1)$$

where:

t = thickness of the top or bottom slab (in.)
B_{cap} = bent cap width (in.)

21.2.4.3 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 longitudinal push analysis:

- If *wFRAME* program is used, columns are lumped together

- For prestressed superstructures, gross moment of inertia is used for the superstructure
- Bent overturning is ignored
- The abutment is modeled as a linear spring whose 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 21.2-1 must be calculated and used. If using xSECTION , this can be achieved by specifying in the xSECTION input file, the angle between the column section coordinate system and the longitudinal direction of the bridge as shown in the sketch below.

Both left and right longitudinal pushover analyses of the bridge should be performed.

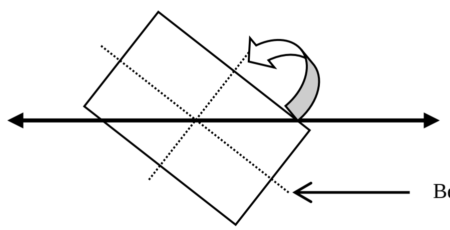


Figure 21.2-1 Bridge Longitudinal Direction

It is Caltrans' practice to design the abutment backwall so that it breaks off in shear during a seismic event. *SDC* Section 7.8.1 requires 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 abutment embankment fill stiffness is non-linear and is highly dependent upon the properties of the backfill. The initial embankment fill stiffness, K_i, is estimated at 50 kip/in./ft for embankment fill material meeting the requirements of *Caltrans Standard Specifications* and 25 kip/in./ft, if otherwise.

The initial stiffness, K_i shall be adjusted proportional to the backwall/diaphragm height as follows:

$$K_{abut} = K_i w \left(\frac{h}{5.5} \right) \qquad (SDC\ 7.8.1\text{-}2)$$

where:

w = projected width of the backwall or diaphragm for seat and diaphragm abutments, respectively (ft)

h = height of the backwall or diaphragm for seat and diaphragm abutments, respectively (ft)

The passive pressure resisting movement at the abutment, P_w, is given as:

$$P_w = A_e(5)\left(\frac{h_{bw} \text{ or } h_{dia}}{5.5}\right) \quad \text{kip} - \text{ft} \qquad (SDC\ 7.8.1\text{-}3)$$

where:

$$A_e = \begin{cases} h_{bw} w_{bw} & \text{For seat abutments} \\ h_{dia} w_{dia} & \text{For diaphragm abutments} \end{cases} \qquad (SDC\ 7.8.1\text{-}4)$$

The terms h_{bw}, h_{dia}, w_{bw}, and w_{dia}, are defined in *SDC* Figure 7.8.1-2.

SDC Section 7.8.1 specifies that the effectiveness of the abutment shall be assessed by the coefficient:

$$R_A = \Delta_D / \Delta_{eff} \qquad (SDC\ 7.8.1\text{-}5)$$

where:

R_A = abutment displacement coefficient
Δ_D = the longitudinal displacement demand at the abutment from elastic analysis
Δ_{eff} = the effective longitudinal abutment displacement at idealized yield

Details on the interpretation and use of the coefficient R_A value are given in *SDC* Section 7.8.1.

21.2.5 *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:

$$P_{dl}\Delta_r \le 0.20 M_p^{col} \qquad (SDC\ 4.2\text{-}1)$$

where:

M_p^{col} = idealized plastic moment capacity of a column calculated from M-ϕ analysis
P_{dl} = dead load axial force
Δ_r = relative lateral offset between the base of the plastic hinge and the point of contra-flexure

21.2.6 Minimum Lateral Strength

SDC Section 3.5 specifies that each bent shall have a minimum lateral flexural 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.

21.2.7 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 nominal material properties, as follows:

$$\phi V_n \geq V_0 \qquad (SDC\ 3.6.1\text{-}1)$$

where:

$$V_n = V_c + V_s \qquad (SDC\ 3.6.1\text{-}2)$$

$$\phi = 0.90$$

21.2.7.1 Shear Demand V_o

Shear demand associated with overstrength moment may be calculated from:

$$V_0 = \frac{M_0^{col}}{L} \qquad (21.2\text{-}2)$$

where:

$$M_0^{col} = 1.2 M_p^{col} \qquad (SDC\ 4.3.1\text{-}1)$$

L = clear length of column

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

21.2.7.2 Concrete Shear Capacity

$$V_c = v_c A_e \qquad (SDC\ 3.6.2\text{-}1)$$

where:

$$A_e = (0.8) A_g \qquad (SDC\ 3.6.2\text{-}2)$$

$$v_c = f_1 f_2 \sqrt{f_c'} \leq 4\sqrt{f_c'} \quad \text{(Inside the plastic hinge region)} \qquad (SDC\ 3.6.2\text{-}3)$$

$$= 3 f_2 \sqrt{f_c'} \leq 4\sqrt{f_c'} \quad \text{(Outside the plastic hinge region)} \qquad (SDC\ 3.6.2\text{-}4)$$

$$0.3 \le f_1 = \frac{\rho_s f_{yh}}{0.150} + 3.67 - \mu_d \le 3 \quad (f_{yh} \text{ in ksi}) \qquad (SDC\ 3.6.2\text{-}5)$$

$$\rho_s f_{yh} \le 0.35\text{ksi} \qquad (21.2\text{-}3)$$

$$f_2 = 1 + \frac{P_c}{2{,}000 A_g} < 1.5 \quad (P_c \text{ is in lb, } A_g \text{ is in in.}^2) \qquad (SDC\ 3.6.2\text{-}6)$$

21.2.7.3 Transverse Reinforcement Shear Capacity V_s

$$V_s = \left(\frac{A_v f_{yh} D'}{s}\right) \qquad (SDC\ 3.6.3\text{-}1)$$

where:

$$A_v = n\left(\frac{\pi}{2}\right)A_b \qquad (SDC\ 3.6.3\text{-}2)$$

n = number of individual interlocking spiral or hoop core sections

21.2.7.4 Maximum Shear Reinforcement Strength, $V_{s,max}$

$$V_{s,\max} \le 8\sqrt{f_c'}A_e \quad \text{(psi)} \qquad (SDC\ 3.6.5.1\text{-}1)$$

21.2.7.5 Minimum Shear Reinforcement

$$A_{v,\min} \ge 0.025\frac{D' s}{f_{yh}} \quad \left(\text{in.}^2\right) \qquad (SDC\ 3.6.5.2\text{-}1)$$

21.2.7.6 Column Shear Key

The area of interface shear key reinforcement, A_{sk} in hinged column bases shall be calculated as shown in the following equations:

$$A_{sk} = \frac{1.2(F_{sk} - 0.25P)}{f_y} \quad \text{if } P \text{ is compressive} \qquad (SDC\ 7.6.7\text{-}1)$$

$$A_{sk} = \frac{1.2(F_{sk} + P)}{f_y} \quad \text{if } P \text{ is tensile} \qquad (SDC\ 7.6.7\text{-}2)$$

where:

$$A_{sk} \ge 4 \text{ in.}^2 \qquad (21.2\text{-}4)$$

F_{sk} = 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)
= lowest column axial load if net P is compressive considering overturning effects
= largest column axial load if net P is tensile, considering overturning effects

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

$$A_{cv} \geq \frac{4.0F_{sk}}{f_c^{'}} \quad (SDC\ 7.6.7\text{-}3)$$

$$A_{cv} \geq 0.67F_{sk} \quad (SDC\ 7.6.7\text{-}4)$$

In addition, the area of concrete section used in the hinge must be enough to meet the axial resistance requirements as provided in AASHTO Article 5.7.4.4 (AASHTO 2012), based on the column with the largest axial load.

21.2.8 Bent Cap Flexural and Shear Capacity

According to *SDC* Section 3.4, 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 shear capacity of the bent cap is calculated according to AASHTO Article 5.8 (AASHTO 2012).

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 column moment capacity as M_0 and then compared with the available flexural and shear capacity of the bent cap.

The effective bent cap width to be used is calculated as follows:

$$B_{eff} = B_{cap} + (12t) \quad (SDC\ 7.3.1.1\text{-}1)$$

t = thickness of the top or bottom slab

21.2.9 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 3.4 and 4.3.2, a capacity design approach is adopted to ensure that the superstructure has an appropriate strength reserve above demands generated from probable column plastic hinging. *MTD* 20-6 (Caltrans 2001a) describes the philosophy, design criteria, and a procedure for determining the seismic demands in the superstructure, and also recommends a method for determining the flexural capacity of the superstructure at all critical locations.

21.2.9.1 General Assumptions

As discussed in *MTD* 20-6, some of the assumptions made to simplify the process of calculating seismic demands in the superstructure include:

- 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 = cross sectional dimension of the column (in.)
D_s = depth of the superstructure (in.)

21.2.9.2 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. The following sign convention (see Figure 21.2-2a) for positive moments, shears and axial forces, is recommended. The sign convention used in *wFRAME* program is shown in Figure 21.2-2b. It should be noted that although the *wFRAME* element level sign convention is different from the standard sign convention adopted here, the resulting member force conditions (for example, member in positive or negative bending, tension or compression, etc.) are the same as furnished by the standard convention. In particular, note that the inputs M_p and M_n for the beam element in the *wFRAME* program correspond to tension at the beam bottom (i.e., positive bending) and tension at the beam top (i.e., negative bending), respectively. The engineer should also ensure that results obtained while using the computer program CTBridge (Caltrans 2007) are consistent with the above sign conventions when comparing outputs or employing the results of one program as inputs into another program.

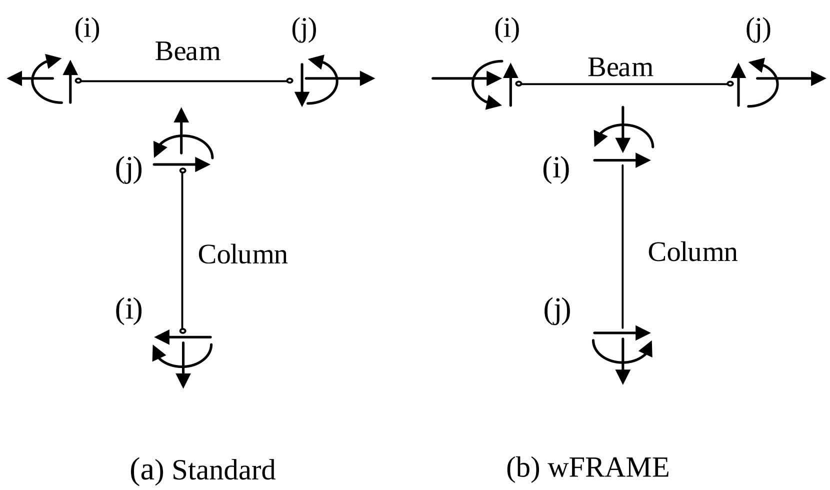


Figure 21.2-2 Sign Convention for Positive Moment, Shear and Axial force (Element Level)

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 21.2-3 shows schematically this approach of calculating columns seismic forces.

As recommended in *MTD* 20-6, 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 SAP2000 (CSI 2007) to compute the moment demand in the superstructure members. Another method involves using the wFRAME program to perform a longitudinal pushover analysis by specifying the required seismic moments in the columns as the plastic hinge capacities of the column ends. The pushover is continued until all the plastic hinges have formed.

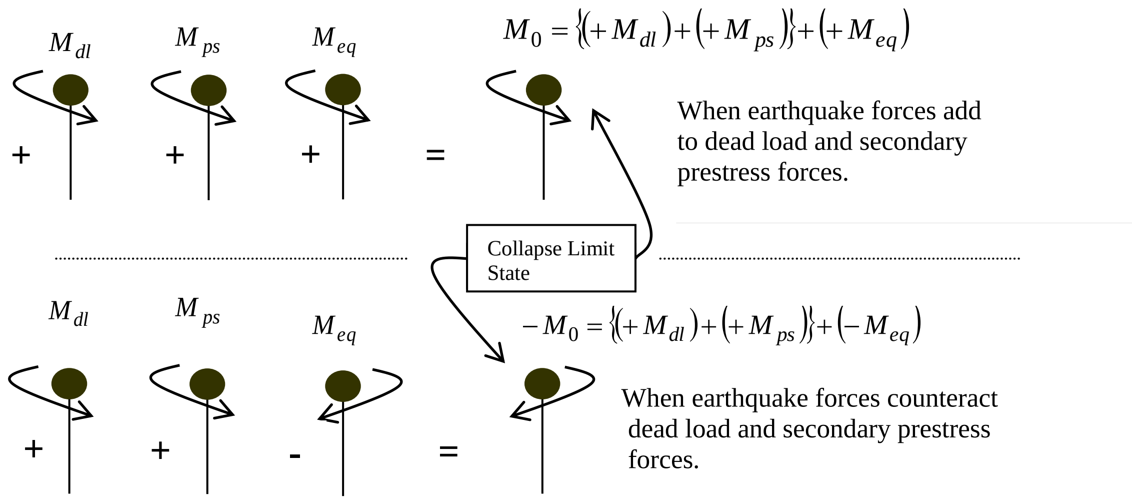


Figure 21.2-3 Column Forces Corresponding to Two Seismic Loading Cases

Note that CTBridge is a three-dimensional analysis program where force results are oriented in the direction of each member's local axis. If wFRAME (a two-dimensional frame analysis program) is used to determine the distribution of seismic forces to the superstructure, it must be ensured that the dead load and secondary prestress moments lie in the same plane prior to using them in any calculations. This must be done especially when horizontal curves or skews are involved.

(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 (Reference MTD 20-6, *SDC* 4.3.2)

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}.

$$M_0^{col@soffit} = M_{dl}^{col@soffit} + M_{ps}^{col@soffit} + M_{eq}^{col@soffit} \qquad (21.2\text{-}5)$$

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}, are calculated from Equation 21.2-5 based upon the principle of superposition as follows:

$$M_{eq}^{col@soffit} = M_0^{col@soffit} - \left(M_{dl}^{col@soffit} + M_{ps}^{col@soffit}\right) \qquad (21.2\text{-}6)$$

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

$$M_o^{col} = 1.2 M_p^{col} \qquad (SDC\ 4.3.1\text{-}1)$$

(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 obtained directly from the wFRAME output or 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:

$$M_D^L = M_{dl}^L + M_{ps}^L + M_{eq}^L \tag{21.2-7}$$

$$M_D^R = M_{dl}^R + M_{ps}^R + M_{eq}^R \tag{21.2-8}$$

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:

$$V_D^L = V_{dl}^L + V_{ps}^L + V_{eq}^L \tag{21.2-9}$$

$$V_D^R = V_{dl}^R + V_{ps}^R + V_{eq}^R \tag{21.2-10}$$

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 demands 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 2.1.3 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.

21.2.9.3 Superstructure Section Capacity

(1) General

To ensure that the superstructure has sufficient capacity to resist the extreme seismic demands determined in Section 21.2.9.2, *SDC* Section 4.3.2 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.,

$$M_{ne}^{\sup(R)} \geq \sum M_{dl}^{R} + M_{p/s}^{R} + M_{eq}^{R} \qquad (SDC\ 4.3.2\text{-}1)$$

$$M_{ne}^{\sup(L)} \geq \sum M_{dl}^{L} + M_{p/s}^{L} + M_{eq}^{L} \qquad (SDC\ 4.3.2\text{-}2)$$

where:

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

(2) Superstructure Flexural Capacity

MTD 20-6 (Caltrans 2001a) describes the philosophy behind the flexural section capacity calculations. 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 21.2-4.

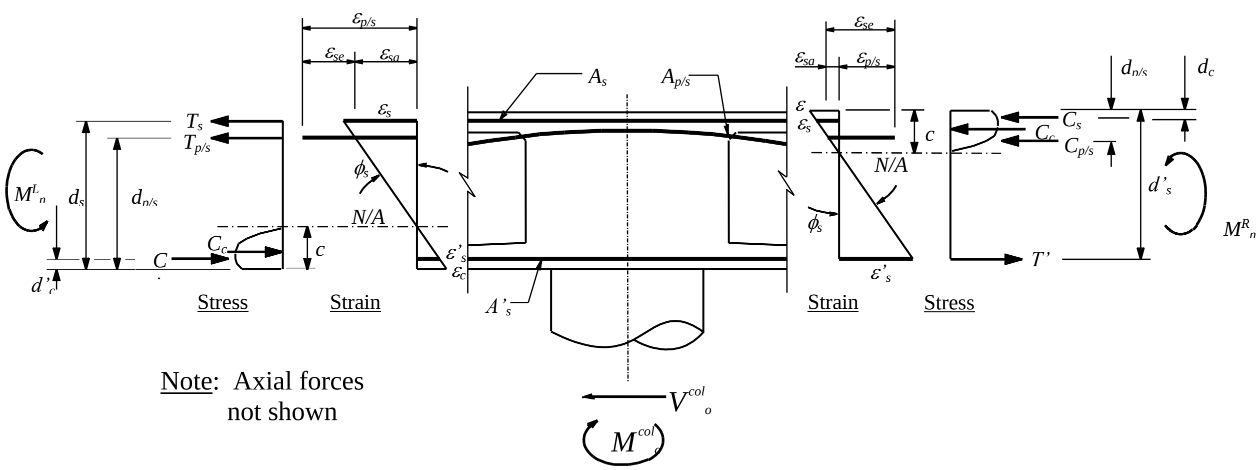


Figure 21.2-4 Superstructure Capacity Provided by Internal Couple

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 3.2.4. According to *MTD* 20-6, 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 3.4, 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 3.4 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

$$A_{ps} = \frac{P_{jack}}{(0.75)(270)} \tag{21.2-11}$$

- 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* 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

MTD 20-6 specifies that the superstructure shear capacity is calculated according to *AASHTO* Article 5.8. As shear failure is brittle, nominal rather than expected material properties are used to calculate the shear capacity of the superstructure.

21.2.10 Joint Shear Design

21.2.10.1 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 compression, p_c: $p_c \le 0.25 f_c'$ (psi) (*SDC* 7.4.2-1)

For principal tension, p_t: $p_t \le 12\sqrt{f_c'}$ (psi) (*SDC* 7.4.2-2)

$$p_t = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \quad (SDC\ 7.4.4.1\text{-}1)$$

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \quad (SDC\ 7.4.4.1\text{-}2)$$

$$v_{jv} = \frac{T_c}{A_{jv}} \quad (SDC\ 7.4.4.1\text{-}3)$$

$$A_{jv} = l_{ac} B_{cap} \quad (SDC\ 7.4.4.1\text{-}4)$$

$$f_v = \frac{P_c}{A_{jh}} \quad (SDC\ 7.4.4.1\text{-}5)$$

$$A_{jh} = (D_c + D_s) B_{cap} \quad (SDC\ 7.4.4.1\text{-}6)$$

$$f_h = \frac{P_b}{B_{cap} D_s} \quad (SDC\ 7.4.4.1\text{-}7)$$

where:

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

f_v = average normal stress in the vertical direction (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_0^{col}/h) associated with the column overstrength plastic hinging moment, M_0^{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* – See Section 21.2.1.1

(3) Minimum Joint Shear Reinforcement

SDC 7.4.4.2 specifies that, if the principal tensile stress, p_t is less than or equal to $3.5\sqrt{f_c'}\ \text{(psi)}$, no additional joint reinforcement is required. However, a minimum area of joint shear reinforcement in the form of column transverse steel continued into the bent cap shall be provided. The volumetric ratio of the transverse column reinforcement ($\rho_{s,\min}$) continued into the cap shall not be less than:

$$\rho_{s,\min} = \frac{3.5\sqrt{f_c'}}{f_{yh}} \quad \text{(psi)} \qquad (SDC\ 7.4.4.2\text{-}1)$$

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.

21.2.10.2 Joint Description

The following types of joints are considered as "T" joints for joint shear analysis (*SDC* Section 7.4.3):

- Integral 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 extends beyond the joint (i.e. column face) far enough to develop the longitudinal cap reinforcement

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 *SDC* Figure 7.4.3-1).

$$S < D_c \qquad (SDC\ 7.4.3\text{-}1)$$

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 = column dimension measured along the centerline of bent

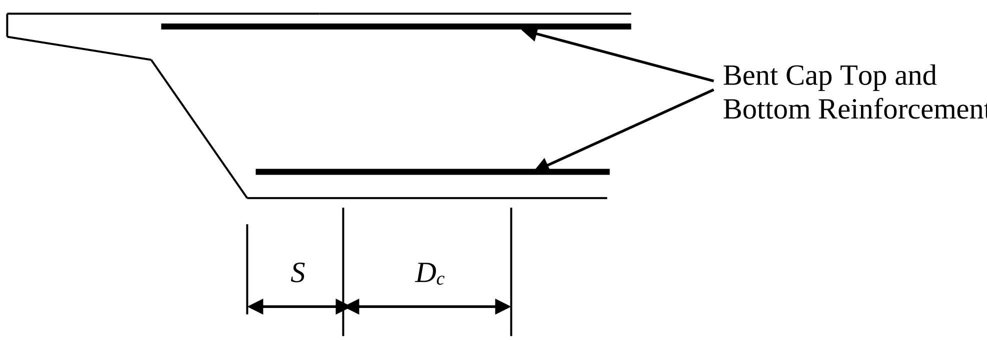


Figure *SDC* 7.4.3-1 Knee Joint Parameters

21.2.10.3 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. The required vertical stirrup area A_s^{jv} is given as

$$A_s^{jv} = 0.2 \times A_{st} \qquad (SDC\ 7.4.4.3\text{-}1)$$

where A_{st} = Total area of column main reinforcement anchored in the joint. Refer to *SDC* Section 7.4.4.3 for placement of the vertical stirrups.

(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 at not more than 18 inches.

$$A_s^{jh} = 0.1 \times A_{st} \quad (SDC\ 7.4.4.3\text{-}2)$$

This horizontal reinforcement shall be placed within a distance D_c extending from either side of the column centerline.

(3) Horizontal Side Reinforcement

The total longitudinal side face reinforcement in the bent cap shall at least be equal to the greater of the area specified in *SDC* Equation 7.4.4.3-3.

$$A_s^{sf} \geq \max \begin{cases} 0.1 \times A_{cap}^{top} \\ \\ 0.1 \times A_{cap}^{bot} \end{cases} \quad (SDC\ 7.4.4.3\text{-}3)$$

where:

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

The side reinforcement 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" bars (dowels) hooked around the longitudinal top deck steel extending alternately 24 in. and 30 in. into the bent cap are required. The J-dowel reinforcement shall be equal to or greater than the area specified as:

$$A_s^{j-bar} = 0.08 A_{st} \quad (SDC\ 7.4.4.3\text{-}4)$$

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

Transverse reinforcement in the joint region shall consist of hoops with a minimum reinforcement ratio specified as:

$$\rho_s = 0.4 \left[\frac{A_{st}}{l_{ac,provided}^2} \right] \quad (SDC\ 7.4.4.3\text{-}5)$$

where:

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

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

For interlocking cores, ρ_s 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.

(6) Anchorage for Main Column Reinforcement

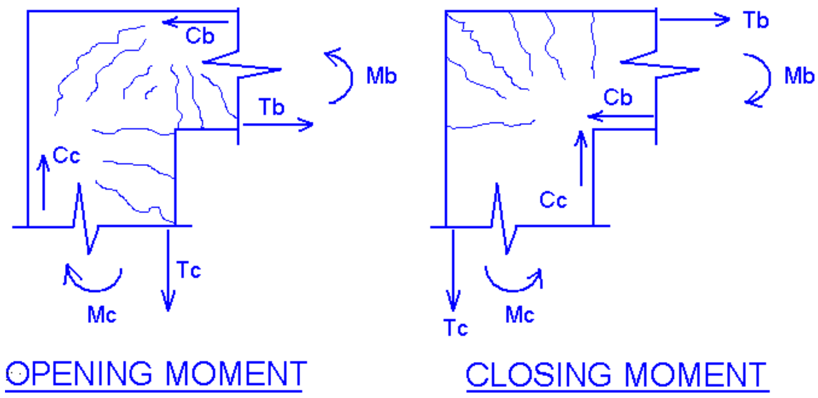
The main column reinforcement shall extend into the cap as deep as possible to fully develop the compression strut mechanism in the joint. If the minimum joint shear reinforcement prescribed in *SDC* Equation 7.4.4.2-1 is met, and the column longitudinal reinforcement extension into the cap beam is confined 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:

$$l_{ac,\,required} = 24 d_{bl} \qquad \text{(}SDC\text{ 8.2.1-1)}$$

With the exception of slab bridges where the provisions of *MTD* 20-7 shall govern, the development length specified above shall not be reduced by use of hooks or mechanical anchorage devices.

21.2.10.4 Knee Joint Shear Reinforcement

Knee joints may fail in either "opening" or "closing" modes (see Figure *SDC* 7.4.5-1). Therefore, both loading conditions must be evaluated. Refer to *SDC* Section 7.4.5 for the description of Knee joint failure modes.



(a) (b)

Figure *SDC* 7.4.5-1 Knee Joint Failure Modes

Two cases of Knee joints are identified as follows:

Case 1: $S < \dfrac{D_c}{2}$ (*SDC* 7.4.5.1-1)

Case 2: $\dfrac{D_c}{2} \le S < D_c$ (*SDC* 7.4.5.1-2)

The following reinforcement is required for Knee joints.

(1) 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:

$$A_s^{u-bar}{}_{\min} = 0.33A_{st}$$ (*SDC* 7.4.5.1-3)

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.

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

Vertical stirrups or ties, A_s^{jv} as specified in *SDC* Equation 7.4.5.1-4, shall be placed transversely within each of regions 1, 2, and 3 of Figure *SDC* 7.4.5.1-1.

$$A_s^{jv} = 0.2 \times A_{st}$$ (*SDC* 7.4.5.1-4)

The stirrups provided in the overlapping areas shown in Figure *SDC* 7.4.5.1-1 shall count towards meeting the requirements of both areas creating the overlap. These stirrups can be used to meet other requirements documented elsewhere including shear in the bent cap.

(3) Horizontal Stirrups - Use for both Cases 1 and 2

Horizontal stirrups or ties, A_s^{jh}, as specified in *SDC* Equation 7.4.5.1-5, 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* 7.4.4.3-2, 7.4.4.3-4, and 7.4.5.1-5 for rebar placement).

$$A_s^{jh} = 0.1 \times A_{st}$$ (*SDC* 7.4.5.1-5)

The horizontal reinforcement shall be placed within the limits shown in Figures *SDC* 7.4.5.1-2 and *SDC* 7.4.5.1-3.

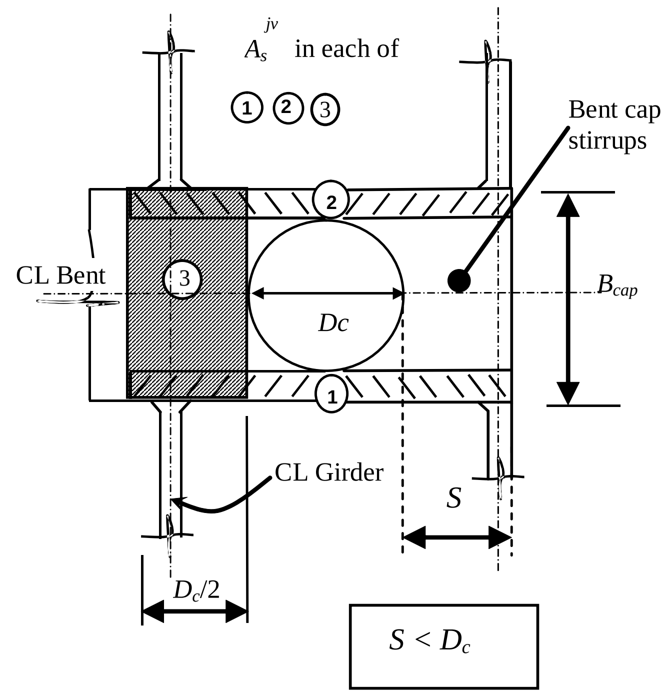


Figure *SDC* 7.4.5.1-1 Location of Knee Joint Vertical Shear Reinforcement (Plan View)

(4) Horizontal Side Reinforcement- Use for both Cases 1 and 2

The total longitudinal side face reinforcement in the bent cap shall be at least equal to the greater of the area specified as:

$$A_s^{sf} \geq \begin{cases} 0.1 \times A_{cap}^{top} \\ \quad or \\ 0.1 \times A_{cap}^{bot} \end{cases} \qquad (SDC\ 7.4.5.1\text{-}6)$$

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})

This 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 7.4.5.1-4 and 7.4.5.1-5 for placement details.

(5) Horizontal Cap End Ties for Case 1 Only

The total area of horizontal ties placed at the end of the bent cap is specified as:

$$A_{s\ \min}^{jhc} = 0.33A_s^{u-bar} \qquad (SDC\ 7.4.5.1\text{-}7)$$

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.

(6) J-Dowels - Use for both Cases 1 and 2

Same as in Section 21.2.10.3 for T joints, except that placement limits shall be as shown in *SDC* Figure 7.4.5.1-3.

(7) Transverse Reinforcement

Transverse reinforcement in the joint region shall consist of hoops with a minimum reinforcement ratio as specified in *SDC* Equations 7.4.5.1-9 to 7.4.5.1-11.

$$\rho_s = \frac{0.76A_{st}}{D_c l_{ac,\,provided}} \qquad \text{(For Case 1 Knee joint)} \qquad (SDC\ 7.4.5.1\text{-}9)$$

$$\rho_s = 0.4\left[\frac{A_{st}}{l_{ac,\,provided}^2}\right] \qquad \text{(For Case 2 Knee joint, Integral bent cap)}$$

(*SDC* 7.4.5.1-10)

$$\rho_s = 0.6\left[\frac{A_{st}}{l_{ac,\,provided}^2}\right] \qquad \text{(For Case 2 Knee joint, Non-integral bent cap)}$$

(*SDC* 7.4.5.1-11)

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

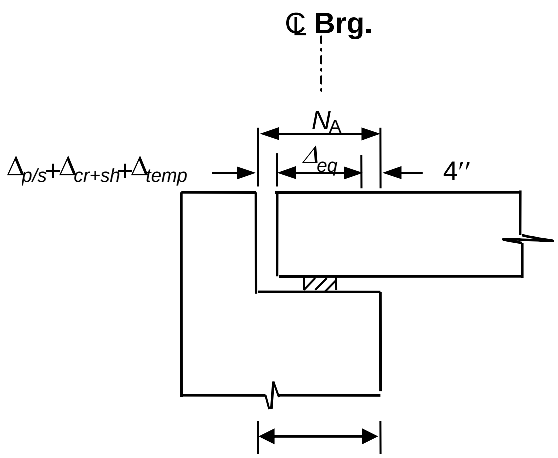
A_{st} and D_c of each core (for Case 1 Knee joints) and on area of reinforcement, A_{st} of each core (for Case 2 Knee joints). All vertical column bars shall be extended as close as possible to the top bent cap reinforcement.

21.2.11 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.

21.2.12 Abutment Seat Width Requirements

Sufficient seat width shall be provided to prevent the superstructure from unseating when the Design Seismic Hazards occur. Per *SDC* Section 7.8.3, the abutment seat width measured normal to the centerline of the bent, N_A, as shown in Figure *SDC* 7.8.3-1 shall be calculated as follows:



Minimum Seat Width, N_A = 30 in.

Figure *SDC* 7.8.3-1 Abutment Seat Width Requirements

$$N_A \geq \Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 4 \quad \text{(in.)} \qquad (SDC\ 7.8.3\text{-}1)$$

where:

N_A = abutment seat width normal to the centerline of bearing. Note that for abutments skewed at an angle θ_{sk}, the minimum seat width measured along the longitudinal axis of the bridge is $N_A/\cos\,\theta_{sk}$ (in.)

$\Delta_{p/s}$ = displacement attributed to pre-stress shortening (in.)

Δ_{cr+sh} = displacement attributed to creep and shrinkage (in.)

Δ_{temp} = displacement attributed to thermal expansion and contraction (in.)

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

The minimum seat width normal to the centerline of bearing as calculated above shall be not less than 30 in.

21.2.13 Hinge Seat Width Requirements

For adjacent frames with ratio of fundamental periods of vibration of the less flexible and more flexible frames greater than or equal to 0.7, *SDC* Section 7.2.5.4 requires that enough hinge seat width be provided to accommodate the anticipated thermal movement (Δ_{temp}), prestress shortening ($\Delta_{p/s}$), creep and shrinkage (Δ_{cr+sh}), and the relative longitudinal earthquake displacement demand between the two frames (Δ_{eq}) - see Figure *SDC* 7.2.5.4-1. The minimum hinge seat width measured normal to the centerline of bent, N_H is given by:

$$N_H \geq \text{the larger of} \begin{cases} \left(\Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 4 \text{ in.}\right) \\ \quad\text{or} \\ 24 \text{ (in.)} \end{cases} \qquad (SDC\ 7.2.5.4\text{-}1)$$

where:

$$\Delta_{eq} = \sqrt{\left(\Delta_D^1\right)^2 + \left(\Delta_D^2\right)^2} \qquad (SDC\ 7.2.5.4\text{-}2)$$

Δ_{eq} = relative earthquake displacement demand at an expansion joint (in.)

$\Delta_D^{(i)}$ = the larger earthquake displacement demand for each frame calculated by the global or stand-alone analysis (in.)

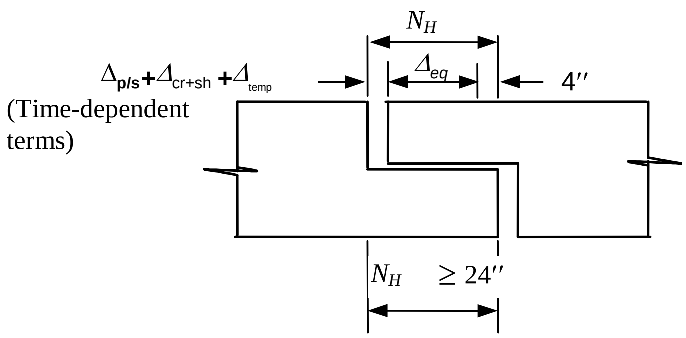


Figure *SDC* 7.2.5.4-1 Minimum Hinge Seat Width

21.2.14 Abutment Shear Key Design

21.2.14.1 General

According to *SDC* Section 7.8.4, abutment shear key force capacity, F_{sk} shall be determined as follows:

$$F_{sk} = \alpha(0.75V_{piles} + V_{ww}) \quad \text{For Abutment on piles} \quad (SDC\ 7.8.4\text{-}1)$$

$$F_{sk} = \alpha\, P_{dl} \quad \text{For Abutment on Spread footing} \quad (SDC\ 7.8.4\text{-}2)$$

$$0.5 \le \alpha \le 1 \quad (SDC\ 7.8.4\text{-}3)$$

where:

V_{piles} = Sum of lateral capacity 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 = 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 7.8.4-1:

$$F_{sk} = \alpha P_{dl}^{\sup} \quad (SDC\ 7.8.4\text{-}4)$$

where:

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

21.2.14.2 Abutment Shear Key Reinforcement

The *SDC* provides two methods for designing abutment shear key reinforcement, namely, Isolated and Non-isolated methods.

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

$$A_{sk} = \frac{F_{sk}}{1.8 f_{ye}} \quad \text{Isolated shear key} \quad (SDC\ 7.8.4.1\text{A-}1)$$

$$A_{sk} = \frac{1}{1.4 f_{ye}}\left(F_{sk} - 0.4 \times A_{cv}\right) \quad \text{Non-isolated shear key} \quad (SDC\ 7.8.4.1\text{A-}2)$$

$$0.4A_{cv} < F_{sk} \le \min\begin{pmatrix} 0.25 f'_{ce} A_{cv} \\ 1.5 A_{cv} \end{pmatrix} \quad (SDC\ 7.8.4.1\text{A-}3)$$

$$A_{sk} \geq \frac{0.05A_{cv}}{f_{ye}} \qquad (SDC\ 7.8.4.1A\text{-}4)$$

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 in^2, and F_{sk} is in kip. See *SDC* Figure 7.8.4.1-1 for placement of shear key reinforcement for both methods.

(2) Horizontal Reinforcement in the Stem Wall (Hanger Bars), A_{sh}

$$A_{sh} = (2.0)A^{Iso}_{sk(provided)} \qquad \text{Isolated shear key} \qquad (SDC\ 7.8.4.1B\text{-}1)$$

$$A_{sh} = \max\begin{cases}(2.0)A^{Non-iso}_{sk(provided)} \\ \dfrac{F_{sk}}{f_{ye}}\end{cases} \qquad \text{Non-isolated shear key} \qquad (SDC\ 7.8.4.1B\text{-}2)$$

where:

$A^{Iso}_{sk(provided)}$ = area of interface shear reinforcement provided in *SDC* Equation 7.8.4.1A-1(in.^{2})

$A^{Non-iso}_{sk(provided)}$ = area of interface shear reinforcement provided in *SDC* Equation 7.8.4.1A-2 (in.^{2})

For the isolated key design method, the vertical shear key reinforcement, A_{sk} should be positioned relative to the horizontal reinforcement, A_{sh} to maintain a minimum length $L_{\min}$ given by (see Figure *SDC* 7.8.4.1-1A):

$$L_{\min,hooked} = 0.6(a+b) + l_{dh} \qquad (SDC\ 7.8.4.1B\text{-}3)$$

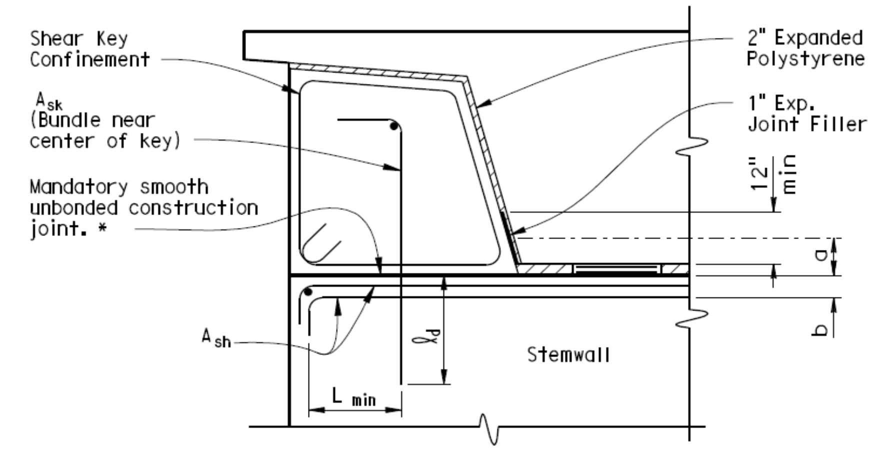
$$L_{\min,headed} = 0.6(a+b) + 3\ \text{in.} \qquad (SDC\ 7.8.4.1B\text{-}4)$$

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 (see Figure *SDC* 7.8.4.1-1(A))

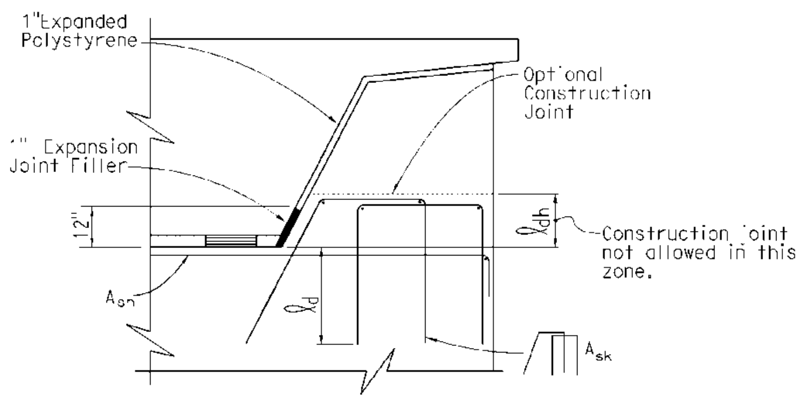
b = vertical distance from the top surface of the stem wall to the centroid of the lowest layer of shear key horizontal reinforcement

l_{dh} = development length in tension of standard hooked bars as specified in AASHTO (2012)



* Smooth construction joint is required at the shear key interfaces with the stemwall 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.

(A) Isolated Shear Key



(B) Non-Isolated Shear Key

NOTES:

(a) Not all shear key bars shown

(b) 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.

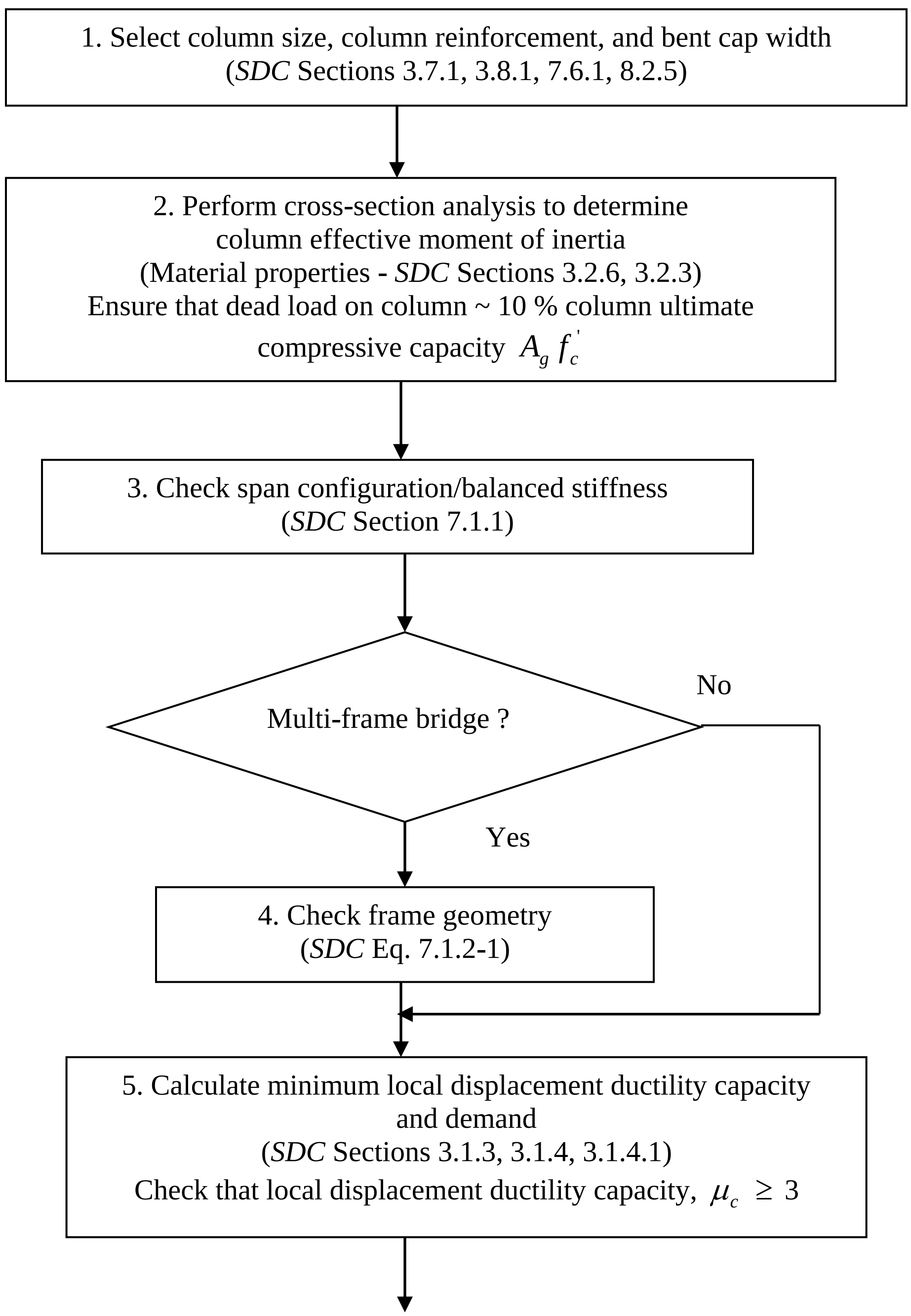
Figure *SDC* 7.8.4.1-1 Abutment Shear Key Reinforcement Details

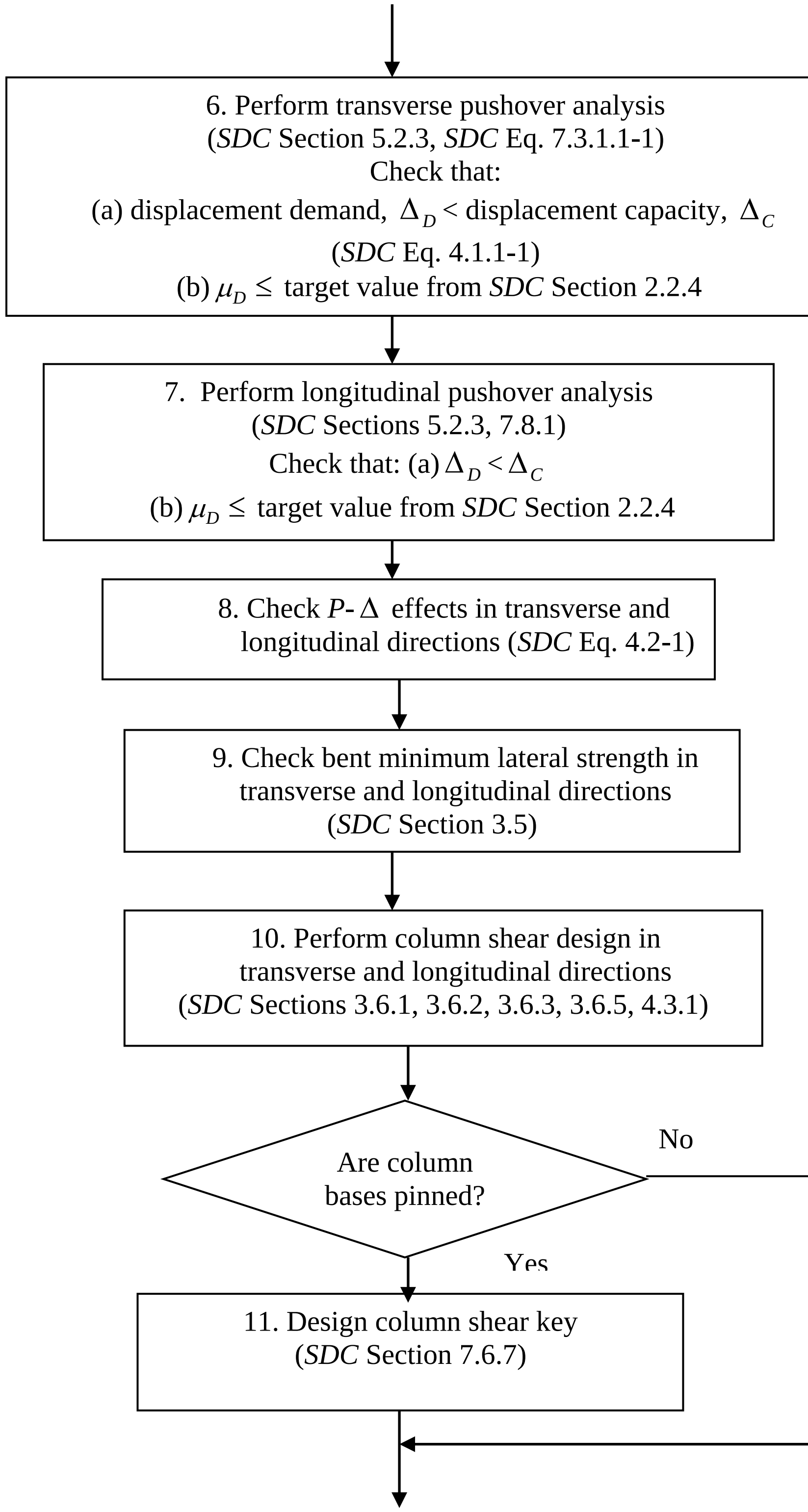
21.2.15 No-Splice Zone Requirements

No splices in longitudinal column reinforcement are allowed in the plastic hinge regions (see *SDC* Section 7.6.3) 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 allowed if the rebar cage is less than 60 ft. long. Refer to *SDC* Section 8.1.1 for more provisions for "No-Splice Zones" in ductile members.

21.2.16 Seismic Design Procedure Flowchart




6. Perform transverse pushover analysis
(*SDC* Section 5.2.3, *SDC* Eq. 7.3.1.1-1)
Check that:
(a) displacement demand, Δ_D < displacement capacity, Δ_C
(*SDC* Eq. 4.1.1-1)
(b) $\mu_D \leq$ target value from *SDC* Section 2.2.4
7. Perform longitudinal pushover analysis
(*SDC* Sections 5.2.3, 7.8.1)
Check that: (a) $\Delta_D < \Delta_C$
(b) $\mu_D \leq$ target value from *SDC* Section 2.2.4
8. Check P-Δ effects in transverse and longitudinal directions (*SDC* Eq. 4.2-1)
9. Check bent minimum lateral strength in transverse and longitudinal directions (*SDC* Section 3.5)
10. Perform column shear design in transverse and longitudinal directions (*SDC* Sections 3.6.1, 3.6.2, 3.6.3, 3.6.5, 4.3.1)
Are column bases pinned?
No
Yes
11. Design column shear key (*SDC* Section 7.6.7)

12. Check bent cap flexural and shear capacity
(*AASHTO* Article 5.8, *SDC* Sect. 3.4, *SDC* Eq. 7.3.1.1-1)

13. Calculate column seismic load moments
(*SDC* Sect. 4.3.2, *MTD* 20-6, *SDC* Eq. 4.3.1-1)

$$M_{eq}^{col@soffit} = M_0^{col@soffit} - \left(M_{dl}^{col@soffit} + M_{ps}^{col@soffit}\right)$$

14. Distribute column seismic moments into the superstructure (S/S) to obtain S/S seismic demands
(Perform right and left Pushover analyses)

15. Calculate S/S moment demands at location of interest
(*SDC* Eq. 7.2.1.1-1, *MTD* 20-6)

$$M_D^L = M_{dl}^L + M_{ps}^L + M_{eq}^L; \quad M_D^R = M_{dl}^R + M_{ps}^R + M_{eq}^R$$

16. Calculate S/S shear demands at location of interest
(*SDC* Eq. 7.2.1.1-1, *MTD* 20-6)

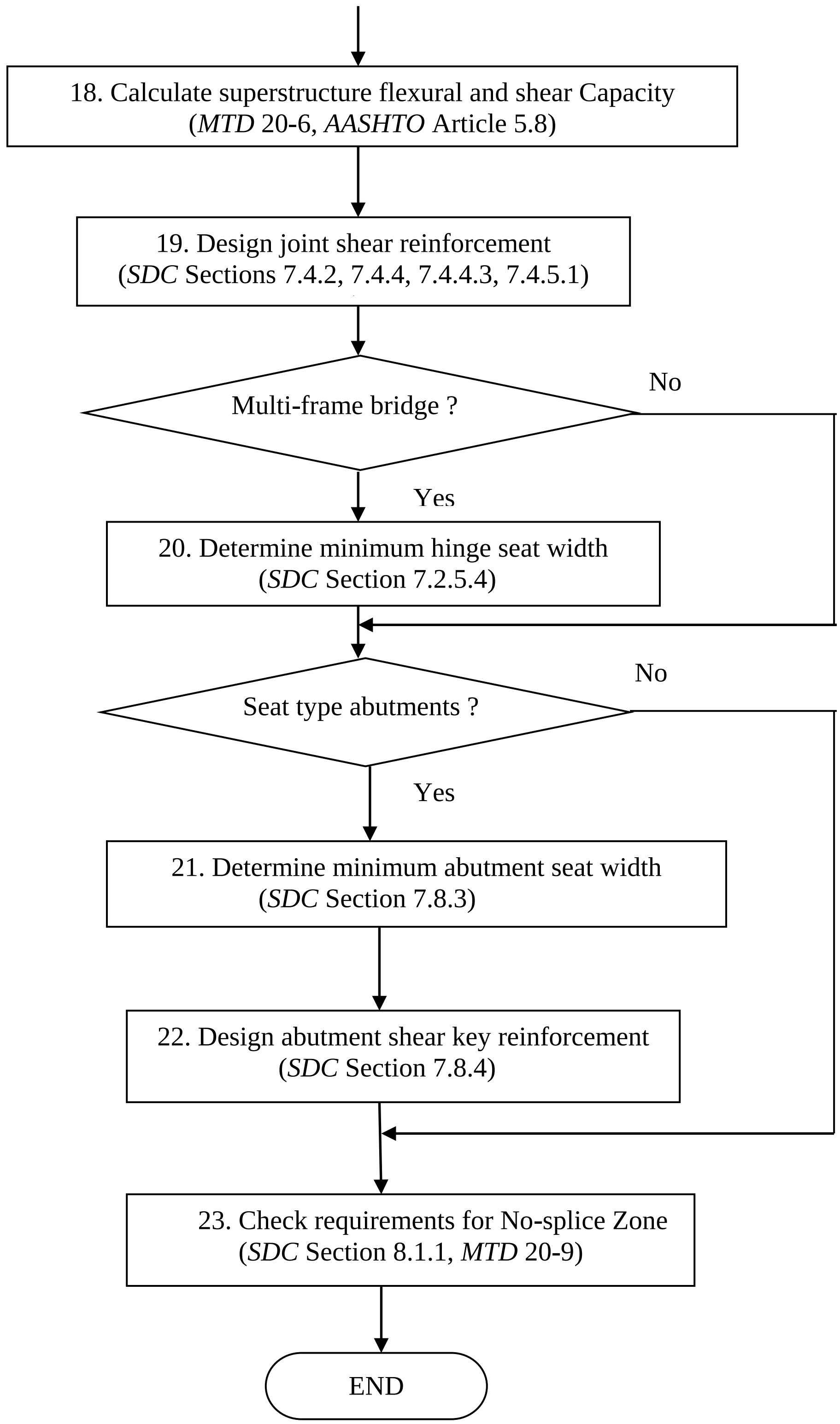
$$V_D^L = V_{dl}^L + V_{ps}^L + V_{eq}^L; \quad V_D^R = V_{dl}^R + V_{ps}^R + V_{eq}^R$$

Is
Site PGA $\geq$ 0.6g ?

No

Yes

17. Perform vertical acceleration analysis
(*SDC* Sections 7.2.2 and 2.1.3)


18. Calculate superstructure flexural and shear Capacity
(*MTD* 20-6, *AASHTO* Article 5.8)
19. Design joint shear reinforcement
(*SDC* Sections 7.4.2, 7.4.4, 7.4.4.3, 7.4.5.1)
Multi-frame bridge ?
No
Yes
20. Determine minimum hinge seat width
(*SDC* Section 7.2.5.4)
Seat type abutments ?
No
Yes
21. Determine minimum abutment seat width
(*SDC* Section 7.8.3)
22. Design abutment shear key reinforcement
(*SDC* Section 7.8.4)
23. Check requirements for No-splice Zone
(*SDC* Section 8.1.1, *MTD* 20-9)
END

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

21.3.1 Bridge Data

The three-span Prestress Reinforced Concrete Box Girder Bridge shown in Figure 21.3-1 will be used to illustrate the principles 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, $f_y = 60$ 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. Because of poor soil conditions, the footing is supported on piles. The ground motion at the bridge site is assumed to be:

Soil Profile:	Type *C*
Magnitude:	8.0 ± 0.25
Peak Ground Acceleration:	0.5*g*

Figure 21.3-2 shows the assumed design spectrum. For more information on Design Spectrum development, refer to *SDC* Section 2.1.1 and Appendix B.

21.3.2 Design Requirements

Perform seismic analysis and design in accordance with Caltrans *SDC* Version 1.7 (Caltrans 2013).

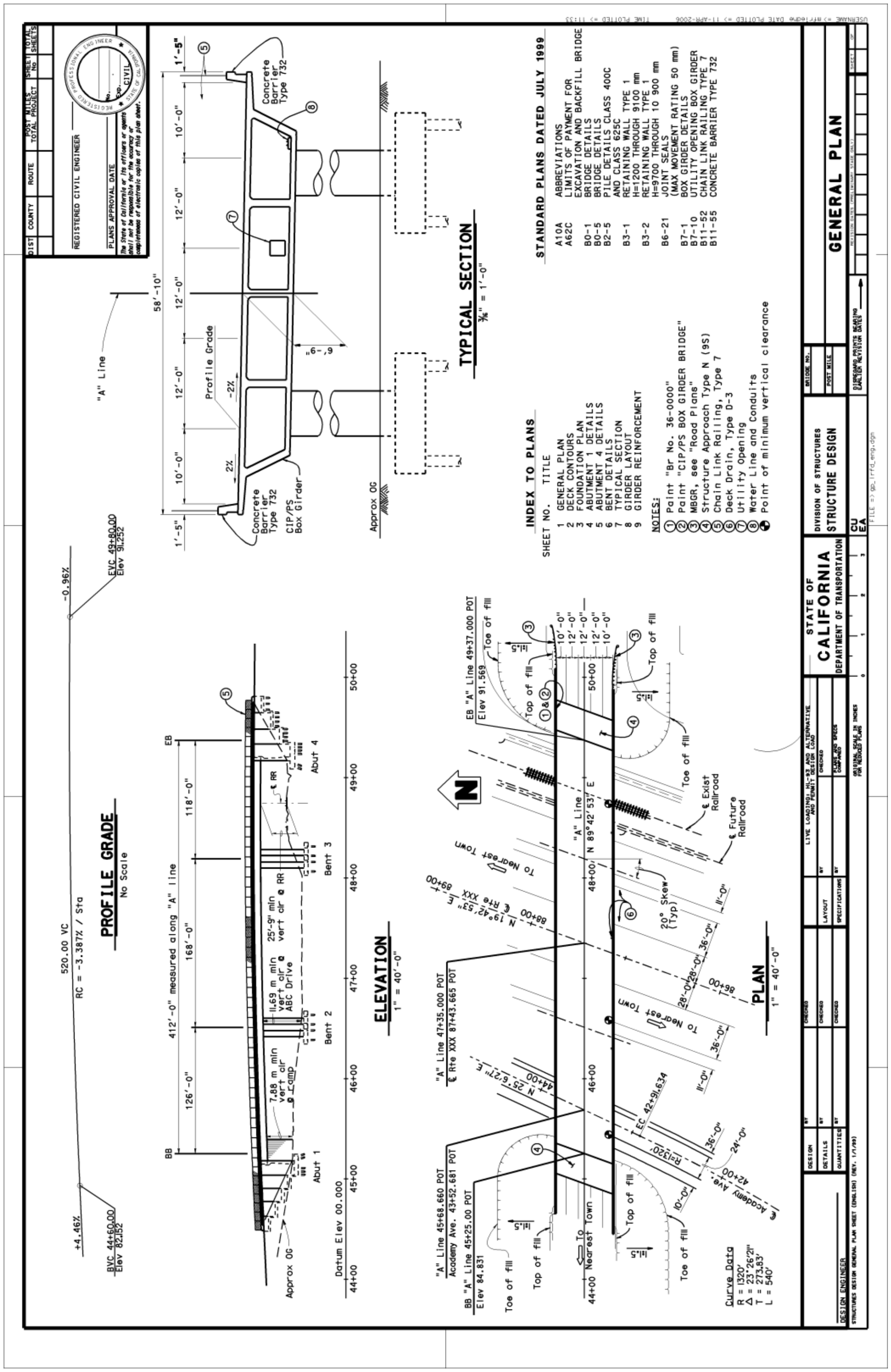


Figure 21.3-1 General Plan (Bridge Design Academy Prototype Bridge)

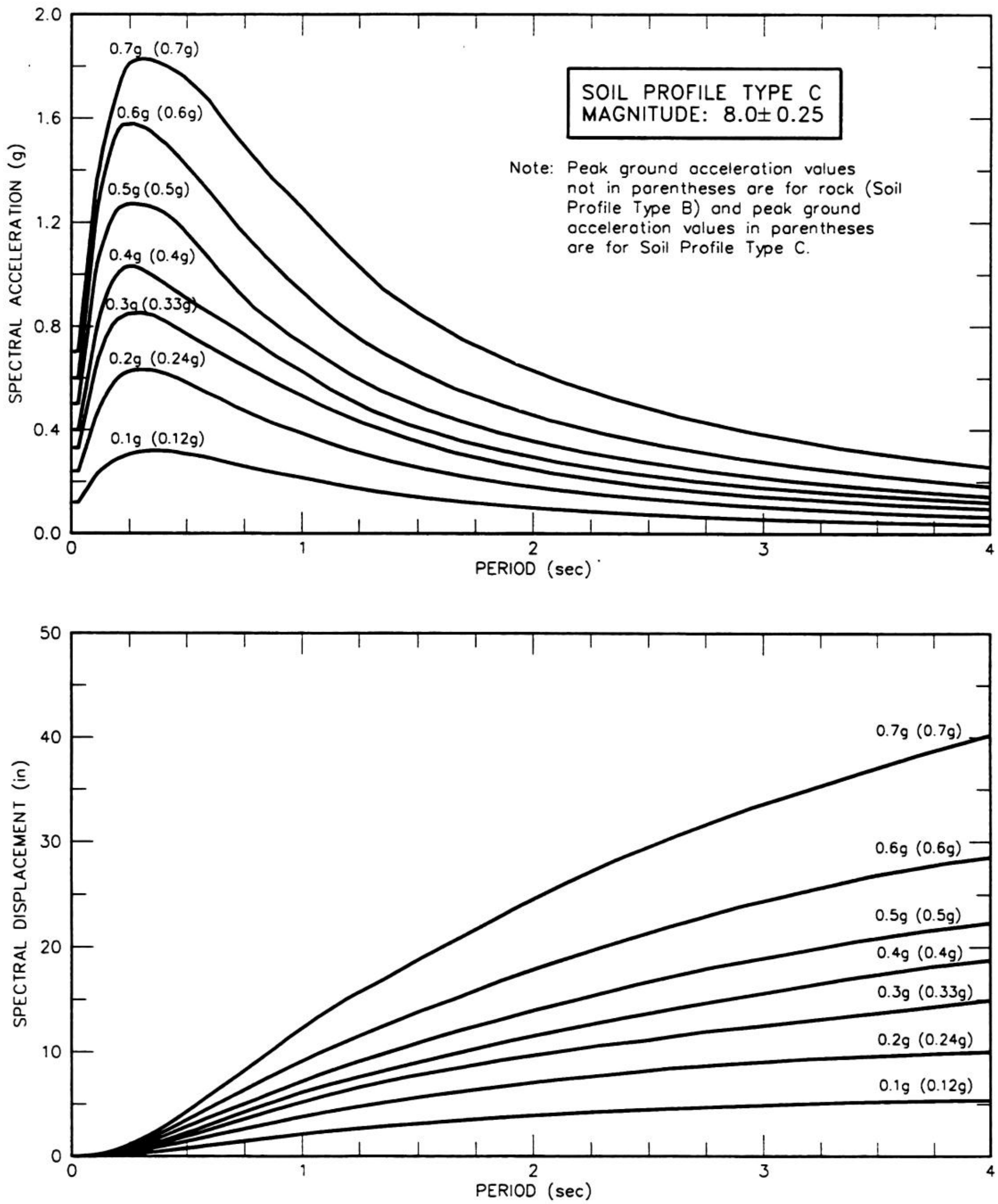


Figure 21.3-2 Design Spectrum for Soil Profile C (M = 8.0±0.25)

21.3.3 Step 1- Select Column Size, Column Reinforcement, and Bent Cap Width

21.3.3.1 Column size

Given $D_s = 6.75$ ft from the strength limit state design, we select a column width $D_c = 6.00$ ft so that $0.70 \le [D_c / D_s = 0.89] \le 1.00$. OK. (*SDC* 7.6.1-1)

21.3.3.2 Bent Cap Width

$$B_{cap} = D_c + 2 = 6 + 2 = 8 \text{ ft} \qquad \text{(}SDC\text{ 7.4.2.1-1)}$$

21.3.3.3 Column Longitudinal and Transverse Reinforcement

$$A_s = 0.015A_g = 0.015\left(\frac{\pi}{4}\right)(6.00(12))^2 = 0.015(4071.5) = 61.07 \text{in.}^2$$

Use: #14 bars for longitudinal reinforcement
#8 hoops @ 5 in c/c for the plastic hinge region

Maximum spacing of hoops = 5 in. < 8 in. < 6×1.693 = 10.2 in. < 72/5 = 14.4 in.
OK. (*SDC* Section 8.2.5)

Number of #14 bars = $\dfrac{61.07}{2.25} = 27.1$

Let us use 26-#14 longitudinal bars (i.e., 1.44% of A_g) $1.0 < 1.44 < 4.0$ OK.
(*SDC* 3.7.1-1/3.7.2-1)

Assuming a concrete cover of 2 in. as specified in *CA Amendment* Table 5.12.3-1 for minimum concrete cover (Caltrans 2014).

Diameter of longitudinal reinforcement loop (from centerline to centerline of longitudinal bars):

$$d_M = 72 - 2(2) - 2(1.13) - 2\left(\frac{1.88}{2}\right) = 63.86 \text{ in.}$$

$\therefore$ Spacing of longitudinal bars = $\dfrac{\pi\, d_M}{26} = 7.7$ in. > 1.5(1.693) in. > 1.5 in.
OK. (AASHTO 5.10.3.1)

Note: If the provided spacing turns out to be more that the maximum spacing allowed, then a smaller bar size can be used.

21.3.4 Step 2 - Perform Cross-section Analysis

21.3.4.1 Calculate Dead Load Axial Force

As a first step toward calculating effective section properties of the column, the dead load axial force at column top (location of potential plastic hinge) is calculated. These column axial forces are obtained from CTBridge output. 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 bottom slab instead of solid cap section. In this example, weight of the whole solid cap was added to the CTBridge results (conservative).

As read from the CTBridge output results, the column dead load axial forces are:

	Column 1	Column 2
Bent 2 (P_c) (kip)	1,489	1,494
Bent 3 (P_c) (kip)	1,425	1,453

$$\text{Average Bent Cap Length} = \frac{\text{Deck Width} + \text{Soffit Width}}{2}\left(\frac{1}{\cos(\text{Skew Angle})}\right)$$

$$= \frac{49.83 + 43.08}{2}\left(\frac{1}{\cos(20^0)}\right) = 49.44\text{ft}$$

$$\text{Bent Cap Weight} = 8(6.75)(49.44)(0.150) = 400\,\text{kips}$$

Adding this bent cap weight, the total axial force in each column becomes:

	Column 1	Column 2
Bent 2 (P_c) (kip)	1,689	1,694
Bent 3 (P_c) (kip)	1,625	1,653

21.3.4.2 Check Column Dead Load Axial Force Ratio

Using Column 2 of Bent 2 (worst case): $\dfrac{1694 \times 100\%}{4071.5(4)} = 10.4\% \sim 10\%$ OK.

21.3.4.3 Material and Section Properties for Section Analysis Using *xSECTION* Program

Expected compressive strength of concrete

$$f'_{ce} = 1.3(4{,}000) = 5{,}200\,\text{psi}\ * > 5{,}000\,\text{psi} \qquad \text{OK.} \qquad (SDC\ 3.2.6\text{-}3)$$

* The *xSECTION* input file was originally created with the value of $f'_{ce} = 5.28\,\text{ksi}$. The resulting values of ductility parameters are not significantly different from the corresponding values obtained using $f'_{ce} = 5.20\,\text{ksi}$. Therefore, the results with $f'_{ce} = 5.28\,\text{ksi}$ are retained.

Other concrete properties used are listed in *SDC* Section 3.2.6.

The following values are used as input to *xSECTION* program:

- Column Diameter = 72.0 in. Concrete cover = 2 in.
- Main Reinforcement: #14 bars, total 26.
- Lateral Reinforcement: #8 hoops @ 5 in c/c, $f'_{ce} = 5{,}200$ psi *
- The program calculates the modulus of elasticity of concrete internally.

For Grade A706 bar reinforcing steel,

- $\varepsilon_{su}^{R} = \begin{cases} 0.09 & \text{Transverse steel} \\ 0.06 & \text{Longitudinal steel} \end{cases}$
- Select Output for Bent 2 Column *xSECTION* run is shown in Appendix 21.3-1.
- Moment-Curvature ($M - \phi$) diagram for Bent 2 Column is shown in Appendix 21.3-2.
- Bent 2 Column Axial Force, P_c = 1,694 kips.
- Bent 3 Column Axial Force, P_c = 1,653 kips.

From M-ϕ analysis results, cracked moment of inertia, I_e = 23.717 ft^4 for Bent 2 columns (See Appendices 21.3-1 and 21.3-2). For Bent 3, I_e = 23.612 ft^4.

21.3.5 Step 3 - Check Span Configuration/Balanced Stiffness

21.3.5.1 Bent 2 Stiffness

$$E_c = 33(w_c)^{1.5}\sqrt{f'_c}\ (psi) = \left[\frac{33(150)^{1.5}\sqrt{5{,}200}}{1{,}000}\right] = 4{,}372\,\text{ksi} \qquad (SDC\ 3.2.6\text{-}1)$$

$$k_2^e = (2)\frac{3EI_e}{L^3} = (2)\left[\frac{(3)(4{,}372)(23.717)(12^4)}{(44\times(12))^3}\right] = 87.64\ \text{kip/in.}$$

21.3.5.2 Bent 3 Stiffness

$$k_3^e = (2)\left[\frac{(3)(4{,}372)(23.612)(12^4)}{(47(12))^3}\right] = 71.59 \text{ kip/in.}$$

m_2 = Total tributary mass at Bent 2 = $\dfrac{(2)(1{,}694)}{(32.2)(12)} = 8.77 \text{ kip} - \text{s}^2/\text{in.}$

m_3 = Total tributary mass at Bent 3 = $\dfrac{(2)(1{,}653)}{(32.2)(12)} = 8.56 \text{ kip} - \text{s}^2/\text{in.}$

$$\frac{k_i^e}{k_j^e} = \frac{71.59}{87.64} = 0.82 > 0.75 > 0.5 \qquad \text{OK.} \qquad (\textit{SDC}\ 7.1.1\text{-}1 \text{ and } 7.1.1\text{-}3)$$

It is seen that the balanced stiffness criteria and span layout configuration are satisfied. Note that since this is a constant width bridge with only two bents and two columns in each bent, we only need to satisfy the more onerous of *SDC* Equations (7.1.1-1) and (7.1.1-3).

21.3.6 Step 4 - Check Frame Geometry

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

21.3.7 Step 5 – Calculate Minimum Local Displacement Ductility Capacity and Demand

21.3.7.1 Displacement Ductility Capacity

(1) Bent 2 Columns

L = 44 ft

$\phi_Y = 0.0000781$ rad/in. as read from the $M - \phi$ data listed in Appendix 21.3-1.

$$L_p = 0.08L + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl}$$

$$= 0.08(528) + 0.15(68)(1.693) = 59.51 \text{in.} > \big(0.3(68)(1.693)\big) = 34.54 \text{in.}$$

OK. (*SDC* 7.6.2.1-1)

$$\Delta_Y = \left(\frac{L^2}{3}\right)\phi_Y = \frac{1}{3}(528)^2(0.0000781) = 7.25 \text{ in.} \qquad (\textit{SDC}\ 3.1.3\text{-}2)$$

Plastic curvature, $\phi_p = 0.000747$ rad/in. (See $M - \phi$ data shown in Appendices 21.3-1 and 21.3-2).

Plastic rotation, $\theta_p = L_p \phi_p = 59.51 \times 0.000747 = 0.044454 \text{ rad.}$ (*SDC* 3.1.3-4)

Plastic displacement, $\Delta_p = \theta_p \left(L - \frac{L_p}{2} \right) = 0.044454 \left(528 - \frac{59.51}{2} \right) = 22.15$ in.

(*SDC* 3.1.3-8)

Total Displacement Capacity, $\Delta_c = \Delta_Y + \Delta_p = 7.25 + 22.15 = 29.40$ in.

(*SDC* 3.1.3-1)

Local displacement ductility capacity, $\mu_c = \frac{\Delta_c}{\Delta_Y} = \left(\frac{29.40}{7.25} \right) = 4.1 > 3$

OK. (*SDC* Section 3.1.4.1)

(2) Bent 3 Columns

Similarly, Δ_p = 24.93 in., Δ_Y = 8.27 in.

$\mu_c = \frac{\Delta_c}{\Delta_Y} = \left(\frac{33.20}{8.27} \right) = 4.0 > 3$ OK.

21.3.7.2 Displacement Ductility Demand

(1) Bent 2

The period of fundamental mode of vibration is as:

$$T_2 = 2\pi \sqrt{\frac{m_2}{k_2^e}} = 2\pi \sqrt{\frac{8.77}{87.64}} = 1.99 \text{ sec.}$$

From the Design spectrum shown in Figure 21.3-2, the value of spectral acceleration for T = 1.99 sec is read as: $a_2 = 0.36g$

Displacement demand, $\Delta_D = \frac{ma}{k_e} = \frac{8.77(0.36)(32.2)(12)}{87.64} = 13.92$ in.

Displacement Demand ductility, $\mu_D = \frac{13.92}{7.25} = 1.9 \le 5$ OK. (*SDC* Section 2.2.4)

(2) Bent 3

Similarly, for Bent 3, $T_3 = 2.17$ sec. The longer period is expected because Bent 3 columns are longer.

The corresponding value of spectral acceleration, $a_3 = 0.33g$ (Figure 21.3-2)

Displacement demand, $\Delta_D = \frac{8.56(0.33)(32.2)(12)}{71.59} = 15.25$ in.

Displacement Demand ductility, $\mu_D = \frac{15.25}{8.27} = 1.8 \le 5$ OK.

21.3.8 Step 6 – Perform Transverse Pushover Analysis

21.3.8.1 Modeling

Figure 21.3-3 shows a schematic model of the frame in the transverse direction. Data used for the soil springs are shown in Appendix 21.3-3.

The following values of column effective section properties for Bent 2 and idealized plastic moment capacity (under dead loads only) obtained from *xSECTION* output (see Appendix 21.3-1) are used as input in *wFRAME* program for pushover analysis.

P_c (kip)	M_p (kip-ft)	I_e (ft^4)	ϕ_y (rad/in.)	ϕ_p (rad/in.)
1,694	13,838	23.717	0.000078	0.000747

Appendices 21.3-4 and 21.3-5 show select portions of *xSECTION* output for the cap section for positive and negative bending, respectively. The following section properties are used for the *wFRAME* run:

$$A = 62.62\,\text{ft}^{2\,*},\ I_{eff}^{+ve} = 55.57\,\text{ft}^4,\ \ I_{eff}^{-ve} = 48.94\,\text{ft}^4$$

*Note that per *SDC* Equation 7.3.1.1-1, the value of A (effective bent cap cross sectional area) would be 66.62 ft^2. The value of 62.62 ft^4 used is based on effective bent cap overhang width of 34 in. required by *California Amendment* Article 4.6.2.6.1 (Caltrans 2014). However, any errors introduced by using A = 62.62 ft^4 instead of A = 66.62 ft^4 would result in a conservative design.

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 additional compression. The column on the backside experiences the same value in tension, reducing the net axial load. Based on their behavior, these columns are usually known as compression and tension columns, respectively.

At the instant the first plastic hinge forms (in this case at the top of the compression column), the following superstructure displacement and corresponding lateral force values are obtained from the *wFRAME* output (see Appendix 21.3-6):

$$\Delta_y = 8.49\,\text{in.}$$

Corresponding lateral force = 0.171(3,382) = 578 kips, where, 3,382 kips is the total tributary weight on the bent. At this stage, the axial forces in tension and compression columns as read from the *wFRAME* analysis output are 907 kips and 2,474 kips, respectively.

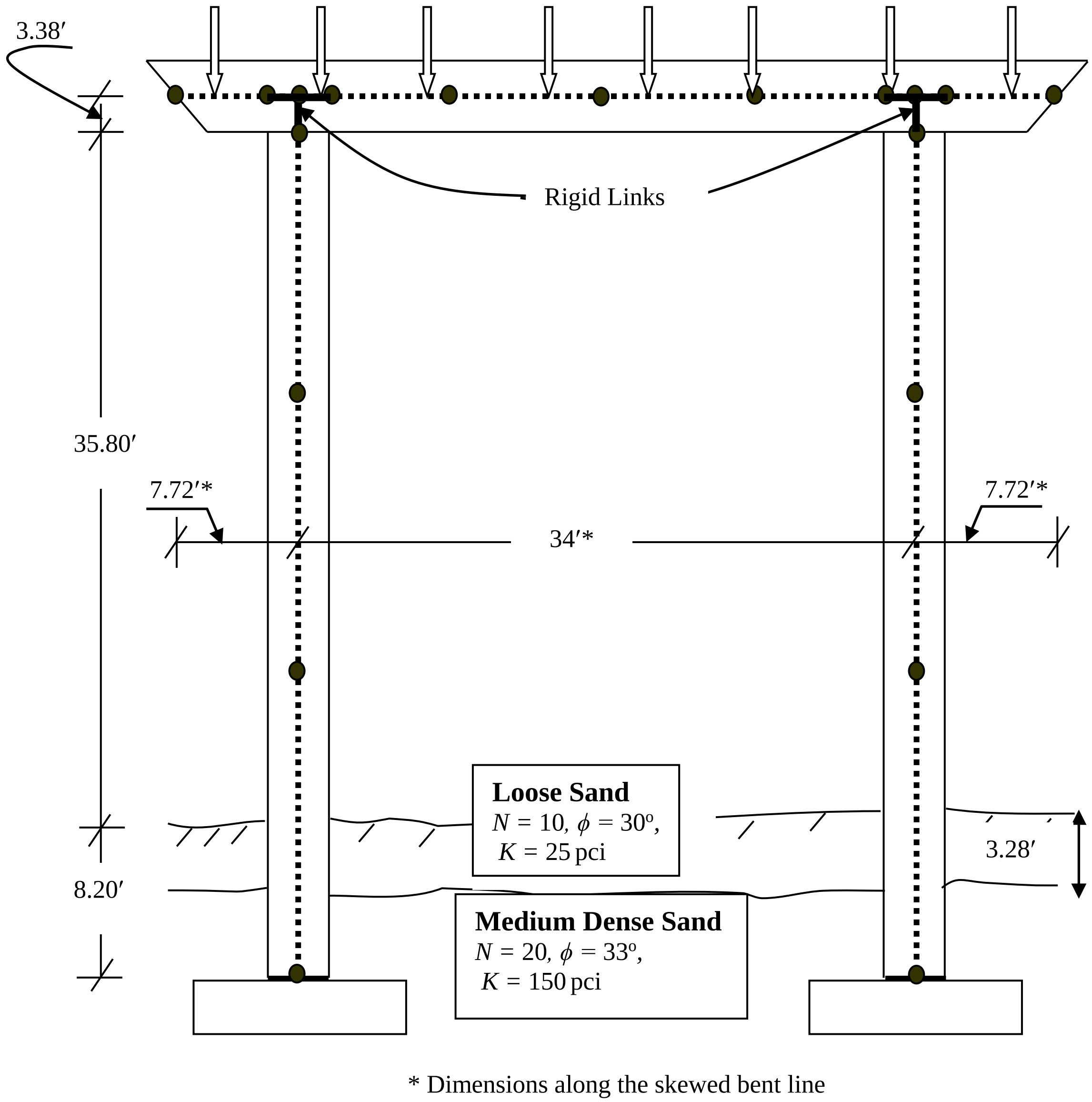


Figure 21.3-3 Transverse Pushover Analysis Model

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

$$M_{overturning} = 578(44) = 25{,}432\ \text{kip-ft}$$

Axial compression corresponding to $M_{overturning}$, $\Delta P = \pm \dfrac{25{,}432}{34} = 748\ \text{kips}$

The axial force in the compression column will increase to 1,694 + 748 = 2,242 kips. The tension column will see its axial compression drop to 1,694-748 = 946 kips. These values compare very well with the *wFRAME* results. The small differences are probably due to the presence of soil in the more realistic *wFRAME* model.

Column section properties corresponding to the updated axial forces (i.e. with overturning) are obtained from new *xSECTION* runs and summarized in the table below (see Appendices 21.3-7 and 21.3-8 for select portions of the output for the compression and tension columns, respectively).

Column Type	P_c (kip)	M_p (kip-ft)	I_e (ft^4)	ϕ_y (rad/in.)	ϕ_p (rad/in.)
Tension	907	12,636	21.496	0.000079	0.000836
Compression	2,474	14,964	25.572	0.000079	0.000682

Note that higher compression produces a higher value of M_p but a reduction in ϕ_p. This trend occurs in all columns and is a reminder that M_p is not the only indicator of column performance.

With updated values of M_p and I_e, we run a second iteration of the *wFRAME* program. As the frame is pushed laterally, the compression column yields at the top at a displacement $\Delta_{y(1)}$ = 8.79 inches. The tension column has not reached its capacity yet. See Appendix 21.3-9 for these results. At this stage, the column axial forces are read to be 880 kips and 2,502 kips for tension and compression columns, respectively. Since, the change in column axial load is now less than 5%, there is no need for further iteration.

As the frame is pushed further, the already yielded compression column is able to undergo additional displacement because of its plastic hinge rotational capacity. As the bent is pushed further, the top of the tension column yields at a displacement, $\Delta_{y(2)}$ = 10.52 in (see Appendix 21.3-9). 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. The force-displacement relationship is shown in Appendix 21.3-10.

The idealized yield Δ_y, which was calculated previously based upon the assumption that cap beam is infinitely rigid, is updated to 8.79 inches. The corresponding lateral force = 0.176×(3,382) = 595 kips.

21.3.8.2 Displacement Ductility Capacity

The main purpose of the preliminary calculation for Δ_C was to size up the members and ensure that they meet the minimum local displacement ductility capacity of 3 before proceeding with the more realistic and comprehensive pushover analysis that includes the effects of bent cap flexibility.

The displacement capacities for both columns are calculated as before (see Step 5) using updated values of ϕ_p, and summarized below:

Tension Column

L = 44 ft, Lp = 59.51 in.

ϕ_p = 0.000836 rad/in.

Δ_p = 24.79 in.

Δ_c = 10.52 + 24.79 = 35.31 in.

Compression Column

L = 44 ft, $L_p = 59.51$ in.

ϕ_p = 0.000682 rad/in.

Δ_p = 20.22 in.

Δ_c = 8.79 + 20.22 = 29.01 in.

For bents having a large number of columns or more locations for potential hinging, tabulation of these results provides a quick way to determine the critical hinge.

Hinge Location	Yield Displacement (in.)	Plastic Deformation (in.)	Total Displacement Capacity (in.)
Compression Column Top	8.79	20.22	29.01*
Tension Column Top	10.52	24.79	35.31

* Critical bent displacement capacity, Δ_C.

21.3.8.3 Displacement Ductility Demand

(1) Bent 2

$$k_2^e = \frac{F_y}{\Delta_y} = \frac{595}{8.79} = 67.69\,\frac{k}{in}$$

$$T = 2\pi\sqrt{\frac{8.77}{67.69}} = 2.26\,\text{sec}$$

From the Design Spectrum (DS) curve, the spectral acceleration a_2 is read as 0.32g. The maximum seismic displacement demand is estimated as:

$$\Delta_D = \frac{8.77\times(0.32\times 32.2\times 12)}{67.69} = 16.02\,\text{in.}$$

$$\mu_D = \frac{16.02}{8.79} = 1.82 < 5$$ OK. (*SDC* Section 2.2.4)

Also, $\Delta_D = 16.02\,\text{in.} < \Delta_C = 29.01\,\text{in.}$ OK. (*SDC* 4.1.1-1)

Note that the bent is forced well beyond its yield displacement but that collapse is prevented because of ductile capacity. This is what we expect of Caltrans "No Collapse" Performance Criteria. Based upon these checks one might conclude that the column is over designed for the anticipated seismic demand. However, as shown later, the *P-Δ* effect controls the column flexural design.

The above calculation was made assuming the bent stiffness equals the stiffness at first yield. This assumption is valid because the two hinges occurred close to each other (i.e., 8.79 in and 10.52 in). If this assumption is not valid, a more exact calculation may be carried out using idealized stiffness as follows (see Figure 21.3-4):

$$k_2^e = \frac{F_y}{\Delta_y} = \frac{640}{9.6} = 66.67\ \text{kip/in.}$$

$$T = 2\pi\sqrt{\frac{8.77}{66.67}} = 2.28\,\text{sec}.$$

a_2 = 0.3g; $\Delta_D = \dfrac{8.77(0.32)(32.2)(12)}{66.67} = 16.27\,\text{in.}$

$$\mu_D = \frac{16.27}{9.6} = 1.69 < 5 \qquad OK.$$

$\Delta_D = 16.27\,\text{in.} < \Delta_C = 29.01\,\text{in.}$ *OK.*

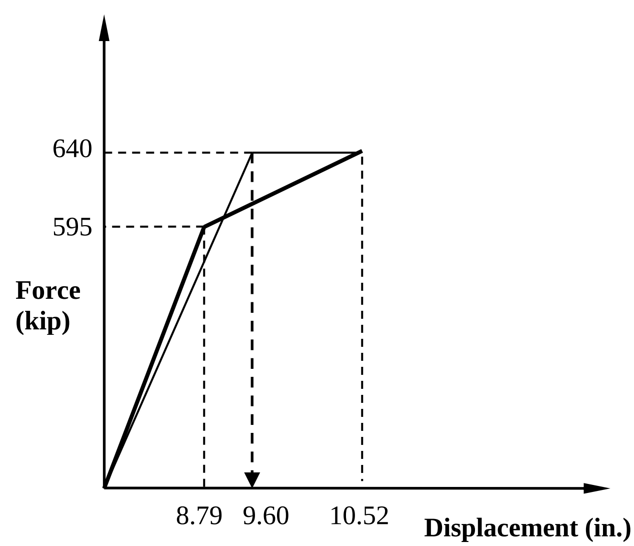


Figure 21.3-4 Force – Displacement Relations

(2) Bent 3

The same procedure is repeated to perform transverse pushover analysis for Bent 3. The results are summarized below:

Tension Column	Compression Column
$L = 47\,\text{ft},\ L_p = 62.39\,\text{in.}$	$L = 47\,\text{ft},\ L_p = 62.39\,\text{in.}$
$\phi_p = 0.000842\,\text{rad/in.}$	$\phi_p = 0.000685\,\text{rad/in.}$
$\Delta_p = 27.99\,\text{in.}$	$\Delta_p = 22.77\,\text{in.}$
$\Delta_c = 11.48 + 27.99 = 39.47\,\text{in.}$	$\Delta_c = 9.71 + 22.77 = 32.48\,\text{in.}^*$

* *Critical bent displacement capacity,* Δ_C.

Seismic Demand

$$k_e^3 = \frac{F_y}{\Delta_y} = \frac{0.180(3{,}278)}{9.71} = 60.77\,\text{kip/in.}$$

The period of vibration, $T = 2\pi\sqrt{\dfrac{8.56}{60.77}} = 2.36\,\text{sec}$

From Design Spectrum, the spectral acceleration a_3 is read as $0.31g$.

$$\Delta_D = \frac{8.56(0.31)(32.2)(12)}{60.77} = 16.87\,\text{in.}$$

$$\mu_D = \frac{16.87}{9.71} = 1.74 < 5 \qquad \text{OK.}$$

Also, $\Delta_D = 16.87\,\text{in.} < \Delta_C = 32.48\,\text{in.}$ OK.

21.3.9 Step 7- Perform Longitudinal Pushover Analysis

21.3.9.1 Abutment Soil Springs

This bridge is supported on seat type abutments (see Figure 21.3-5 for effective abutment dimensions). The effective area is calculated as:

$$A_e = h_{bw} w_{bw} = 6.75(46.46) = 313.6\,\text{ft}^2 \qquad (\textit{SDC}\ 7.8.1\text{-}4)$$

$$(\,w_{bw} = (49.83 + 43.08)/2 = 46.46\,\text{ft}\,)$$

$$P_w = A_e(5)\left(\frac{h_{bw}}{5.5}\right) = (313.6)(5)\left(\frac{6.75}{5.5}\right) = 1{,}924\,\text{kips} \qquad (\textit{SDC}\ 7.8.1\text{-}3)$$

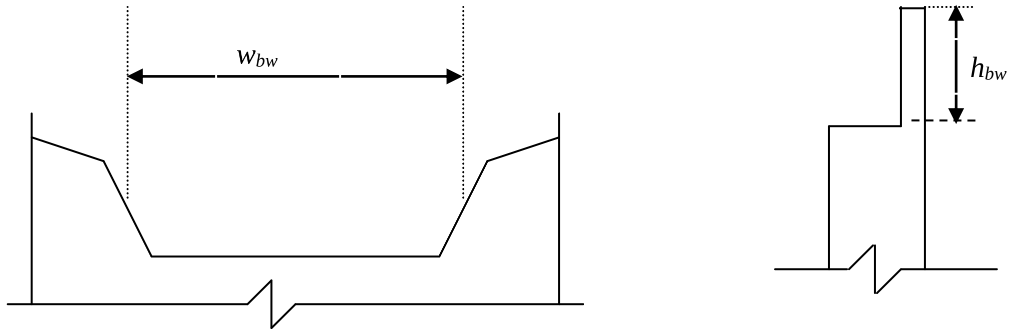


Figure 21.3-5 Effective Area of Seat Type Abutment

Using initial embankment fill stiffness,

$$K_i \approx 50\left(\frac{\text{kips/in.}}{\text{ft}}\right) \qquad (SDC\ 7.8.1\text{-}1)$$

Initial abutment stiffness

$$K_{abut} = K_i w\left(\frac{h}{5.5}\right) == 50(46.46)\left(\frac{6.75}{5.5}\right) = 2{,}851\,\text{kip/in.} \qquad (SDC\ 7.8.1\text{-}2)$$

$$\Delta = \frac{F}{K} = \frac{1{,}924}{2{,}851} = 0.67\,\text{in. (See Figure 21.3-6)}$$

$$\Delta_{effective} = \Delta + \Delta_{gap} = 0.67 + 2.60 = 3.27\,\text{in.} = 0.272\,\text{ft}$$

See Appendix 21.3-11 for calculations for Δ_{gap}, the combined effect of thermal movement and anticipated shortening. Average contributory length is used in the calculation for Δ_{gap}.

$$K_{initial}^{Abut} = \frac{1{,}924}{3.27} = 588\,\text{kip/in.} = 7{,}061\,\text{kip/ft}$$

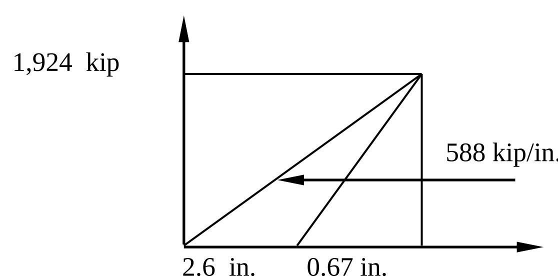


Figure 21.3-6 Initial Abutment Stiffness Iteration

This value is used as the starting abutment stiffness for the longitudinal pushover analysis. When the structure has reached its plastic limit state (i.e., when both bents 2 and 3 columns have yielded), the longitudinal bridge stiffness is calculated as follows:

$$k_{long} = \frac{0.38(8{,}430)}{9.13} = 351 \text{ kip/in.}$$

(See Appendix 21.3-12 for the force-deflection curve for Right Push).

Mass, $m = \dfrac{W}{g} = \dfrac{8{,}430}{32.2 \times 12} = 21.82 \text{ kip} - \text{s}^2/\text{in.}$

$$T = 2\pi\sqrt{\frac{m}{k_{long}}} = 2\pi\sqrt{\frac{21.82}{351}} = 1.57 \text{ sec}$$

$$S_a = 0.48g$$

$$\Delta_D = \frac{F}{K} = \frac{ma}{K} = \frac{21.82(0.48)(32.2)(12)}{351} = 11.53 \text{ in.}$$

$$R_A = \frac{\Delta_D}{\Delta_{effective}} = \frac{11.53}{3.27} = 3.53$$

$$\textit{Since } 2 < R_A < 4, \quad K_{final}^{Abut} = K_{initial}^{Abut} \times \left[1.0 - 0.45(R_A - 2)\right] \quad (\textit{SDC} \text{ Section 7.8.1})$$

$$K_{final}^{Abut} = 588(0.312) = 183 \text{ kip/in.} = 2196 \text{ kip/ft}$$

The following stiffness values as shown in Figure 21.3-7 shall be used for all subsequent *wFRAME* longitudinal pushover analyses:

K_1 = 2,196 kip/ft and Δ_1 = 0.272 ft

K_2 = 0 kip/ft and Δ_2 = 1.0 ft

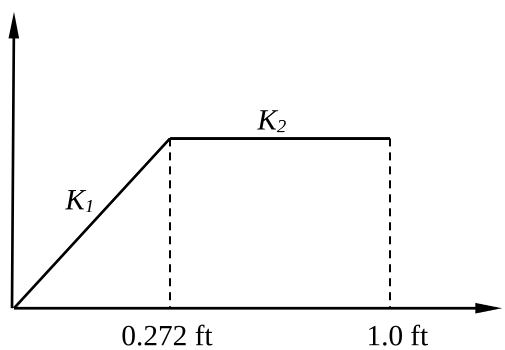


Figure 21.3-7 Final Abutment Stiffness

21.3.9.2 Displacement Ductility Capacity and Demand

From the *wFRAME* results (see Appendix 21.3-13 for the force-displacement relationship for the right push), the yield displacements of Bent 2 and Bent 3 are:

Location	Yield Displacement (Right Push) (in.)	Yield Displacement (Left Push) (in.)
Bent 2	8.86	8.36
Bent 3	9.11	9.84

The plastic deformation capacities for both Bent 2 and Bent 3 are exactly the same as calculated for the transverse bending for the case of gravity loading. This is because the longitudinal case has very little overturning to change the column axial loads.

Δ_p = 22.15 in. for Bent 2 and Δ_p = 24.93 in. for Bent 3.

(1) Bent 2

$$\text{Min} \quad \mu_c = \frac{\Delta_c}{\Delta_Y} = \left(\frac{8.86 + 22.15}{8.86}\right) = 3.5 > 3 \qquad \text{OK.} \qquad (\textit{SDC}\ \text{Section 3.1.4})$$

(2) Bent 3

$$\text{Min}\ \mu_c = \frac{\Delta_c}{\Delta_y} = \left(\frac{9.84 + 24.93}{9.84}\right) = 3.5 > 3 \qquad \text{OK.} \qquad (\textit{SDC}\ \text{Section 3.1.4})$$

From *wFRAME* force-displacement relationship of Appendix 21.3-13, the bridge longitudinal stiffness is calculated when the first bent has yielded.

$$k_{long} = \frac{0.22(8{,}430)}{8.86} = 209\ \text{kip/in.}$$

T = 2.03 sec for which S_a = 0.35g

Δ_D = 14.12 in.

This 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.

$$\text{Max}\ \mu_D = \frac{14.12}{8.36} = 1.7 < 5 \ \text{(Bent 2)} \qquad \text{OK.} \qquad (\textit{SDC}\ \text{Section 2.2.4})$$

$$\text{Max}\ \mu_D = \frac{14.12}{9.11} = 1.5 < 5 \ \text{(Bent 3)} \qquad \text{OK.} \qquad (\textit{SDC}\ \text{Section 2.2.4})$$

21.3.10 Step 8 - Check P - Δ Effects

21.3.10.1 Transverse direction

We have relatively heavily loaded tall columns. P-Δ effects could be significant for this type of situation.

(1) Bent 2 Columns

P_{dl} = 1,694 kips, M_p = 13,838 kip-ft,

Maximum Seismic Displacement Δ_r = 16.02 in.

$$\frac{P_{dl}\Delta_r}{M_p^{col}} = \frac{1{,}694(16.02)}{13{,}838(12)} = 0.16 < 0.20$$ OK. (*SDC* 4.2-1)

(2) Bent 3 Columns

P_{dl} = 1,653 kips, M_p = 13,777 kip-ft,

Maximum Seismic Displacement Δ_r = 16.87 in.

$$\frac{P_{dl}\Delta_r}{M_p^{col}} = \frac{1{,}653(16.87)}{13{,}777(12)} = 0.17 < 0.20$$ OK. (*SDC* 4.2-1)

Now we can see that although the selected column section has more than enough ductility capacity, the column sections meet the P-Δ requirements only by a small margin.

21.3.10.2 Longitudinal Direction

(1) Bent 2 Columns

$$\frac{P_{dl}\Delta_r}{M_p^{col}} = \frac{1{,}694(14.12)}{13{,}838(12)} = 0.14 < 0.20$$ OK. (*SDC* 4.2-1)

(2) Bent 3 Columns

$$\frac{P_{dl}\Delta_r}{M_p^{col}} = \frac{1{,}653(14.12)}{13{,}777(12)} = 0.14 < 0.20$$ OK. (*SDC* 4.2-1)

21.3.11 Step 9 - Check Bent Minimum Lateral Strength

21.3.11.1 Transverse direction

From the force deflection data shown in Appendix 21.3-10, Minimum lateral strength per bent =

$$0.19(3{,}383) = 643\,\text{kips} > 0.1(3{,}383) = 338\text{ kips} \qquad \text{OK.} \quad (\textit{SDC}\text{ Section 3.5})$$

2.3.11.2 Longitudinal Direction

From the force deflection data shown in Appendix 21.3-13,
Minimum lateral strength per column =

$$0.22\left(\frac{8{,}430}{2}\right) = 927\text{ kips} > 0.1\left(\frac{8{,}430}{2}\right) = 422\,\text{kips} \qquad \text{OK.} \quad (\textit{SDC}\text{ Section 3.5})$$

21.3.12 Step 10 - Perform Column Shear Design

21.3.12.1 Transverse Bending

(1) Bent 2

$$M_0 = 1.2M_p = 1.2(14{,}964) = 17{,}957\,\text{kip}-\text{ft} \text{ (includes overturning effects).}$$

Shear demand associated with overstrength moment is as:

$$V_0 = \frac{M_0}{L} = \frac{17{,}957}{44} = 408\,\text{kips}$$

Alternatively, from *wFRAME* output (see Appendix 21.3-9), the maximum column shear demand $= 1.2(349) = 419\,\text{kips}$.

The presence of soil around the footing results in a slightly shorter effective column length, which in turn causes slightly higher column shear demand in the *wFRAME* output.

Concrete Shear Capacity, V_c

For #8 hoops @ 5 in o.c.,

$$A_b = 0.79\,\text{in.}^2, \quad D' = 72 - 2 - 2 - \frac{1.13}{2} - \frac{1.13}{2} = 66.87\,\text{in.}, \quad s = 5\,\text{in.}$$

$$\rho_s = \frac{4A_b}{D's} = 0.009451 \qquad (\textit{SDC}\text{ 3.8.1-1})$$

f_{yh} = 60 ksi

$$\rho_s f_{yh} = 0.009451(60) = 0.57 > 0.35$$

$$\therefore \text{ Use } \rho_s f_{yh} = 0.35\,\text{ksi} \qquad (\textit{SDC}\text{ Section 3.6.2})$$

Using the maximum value of the displacement ductility demand, μ_d = 1.82 (see calculation for Bent 2 – Transverse pushover analysis), the shear capacity factor f_1 is calculated as:

$$f_1 = \frac{\rho_s f_{yh}}{0.150} + 3.67 - \mu_d = \frac{0.35}{0.150} + 3.67 - 1.82 = 4.18 > 3$$

$\therefore$ Use $f_1 = 3$ (*SDC* 3.6.2-5)

$$f_2 = 1 + \frac{P_c}{2{,}000 A_g} = 1 + \frac{880 \times 10^3}{2{,}000\left(\frac{\pi}{4}\right)(6 \times (12))^2} = 1.11 < 1.5 \quad \text{OK.}$$

$\therefore$ Use $f_2 = 1.11$

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 tension column.

$$v_c = f_1 f_2 \sqrt{f_c'} = 3(1.11)\sqrt{4{,}000} = 211\,\text{psi} < 4\sqrt{4{,}000} = 253\,\text{psi} \quad \text{OK.}$$

$$A_e = 0.8\left(\frac{\pi}{4}\right)(6(12))^2 = 3{,}257\,\text{in.}^2$$

$$\therefore V_c = v_c A_e = 211(3{,}257) = 687{,}227\,\text{lb} \approx 687\,\text{kips}$$

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

$$V_s = \left(\frac{n\pi A_v f_{yh} D'}{2s}\right) = \frac{\pi}{2}\left(\frac{0.79(60)(66.87)}{5}\right) = 996\,\text{kips}$$

Maximum shear strength is as:

$$V_{s,\max} = 8\sqrt{f_c'} A_e = 8\sqrt{4{,}000}(3{,}257/1{,}000) = 1{,}648 \text{ kips} > 996 \text{ kips} \quad \text{OK.} \quad (SDC\ 3.6.5.1\text{-}1)$$

Minimum shear reinforcement is as:

$$A_{v,\min} = 0.025\left(\frac{D's}{f_{yh}}\right) = 0.025\left(\frac{66.87(5)}{60}\right) = 0.14\,\text{in.}^2 < 0.79\,\text{in.}^2 \quad \text{OK.} \quad (SDC\ 3.6.5.2\text{-}1)$$

Shear capacity

$$\phi V_n = 0.9(V_c + V_s) = 0.9(687 + 996) = 1{,}515\,\text{kips} > V_0 = 419\,\text{kips} \quad \text{OK.}$$

(2) Bent 3

$$V_0 = \frac{M_0}{L} = \frac{1.2(14{,}893)}{47} = 380\,\text{kips}$$

From the *wFRAME* analysis results, the maximum column shear demand = 1.2 × 340 = 408 kips. Going through a similar calculation as was done for Bent 2 columns, we determine that

$$\phi V_n = 0.9(V_c + V_s) = 0.9(681 + 996) = 1{,}509\,\text{kips} > V_0 = 408\,\text{kips} \qquad \text{OK.}$$

21.3.12.2 Longitudinal bending

(1) Bent 2

V_0 = 1.2V_p = 1.2(645/2) = 377 kips

This corresponds to the maximum shear value of V_p = 323 kips/column obtained from the *wFRAME* pushover analysis.

For μ_D = 1.7, f_1 = 4.3 > 3. Use f_1 = 3.

For dead load axial force, factor f_2 = 1.21

v_c = 230 psi which gives V_c = 749 kips

V_s = 996 kips as calculated before.

$$\phi V_n = 0.9(749 + 996) = 1{,}571\,\text{kips} > V_0 = 307\,\text{kips} \qquad \text{OK.}$$

(2) Bent 3

V_0 = 1.2V_p = 1.2(629/2) = 378 kips

This corresponds to the maximum shear value of V_p = 315 kips/column obtained from the *wFRAME* pushover analysis.

For μ_D = 1.5, factor 1 = 4.5 > 3. Use f_1 = 3

For dead load axial force, factor f_2 = 1.20

v_c = 228 psi which gives V_c = 743 kips

V_s = 996 kips as calculated earlier.

$$\phi V_n = 0.9(743 + 996) = 1{,}565\,\text{kips} > V_0 = 378\,\text{kips} \qquad \text{OK.}$$

21.3.13 Step 11- Design Column Shear Key

21.3.13.1 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

$$A_{sk} = \frac{1.2(F_{sk} - 0.25P)}{f_y} \qquad (SDC\ 7.6.7\text{-}1)$$

P = 815 kips (for column with the lowest axial load) – see Appendix 21.3-9

Shear force associated with column overstrength moment is as:

F_{sk} = shear force associated with column overstrength moment

$$F_{sk} = \begin{cases} 1.2(349) = 419 \text{ kips} & \text{For Bent 2} \\ 1.2(340) = 408 \text{ kips} & \text{For Bent 3} \end{cases}$$

See Step 10 – Perform Column Shear Design and Appendix 21.3-9.

Therefore, F_{sk} = 419 kips

$$A_{sk} = \frac{1.2[419 - 0.25(815)]}{60} = 4.3 \text{ in.}^2 > 4 \text{ in.}^2 \qquad \text{OK.}$$

Provide 6#8 dowels in column key ($A_{sk,provided}$ = 4.74 in.^{2} > 4.3 in.^{2} OK.)

Dowel Cage diameter: Preferred spacing of #8 bars = 4.25 in. (see *BDD* 13-20)

Diameter of dowel cage = (6)(4.25)/π = 8.1 in. say 9 in. cage

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

$$A_{cv} \geq \frac{4.0F_{sk}}{f_c'} = \frac{4.0(419)}{4} = 419 \text{ in.}^2 \qquad (SDC\ 7.6.7\text{-}3)$$

$$A_{cv} \geq 0.67F_{sk} = 281 \text{ in.}^2 \qquad (SDC\ 7.6.7\text{-}4)$$

Per *SDC* Section 7.6.7, A_{cv} must not be less than that required to meet the axial resistance requirements specified in AASHTO Article 5.7.4.4 (AASHTO 2012).

$$\phi P_n = (\phi)(0.85)\left[0.85 f_c'(A_g - A_{st}) + f_y A_{st}\right] \qquad \text{(AASHTO 5.7.4.4-2)}$$

Using the largest axial load with overturning effects P = 2,567 kips (see Appendix 21.3-9) and ϕ = 1 (seismic), we have:

$$\phi P_n = (1.0)(0.85)\left[0.85(4)(A_g - 4.74) + (60)(4.74)\right] = 2{,}567 \text{ kips}$$

A_g = 809 in.^{2} > 419 in.^{2}

Therefore, $A_{cv,reqd} = 809 \text{ in.}^2$

Diameter of $A_{cv} = \sqrt{\dfrac{(809)(4)}{\pi}} = 32 \text{ in.}$

Use A_{cv} diameter = 32 in. (see Figure 21.3-8)

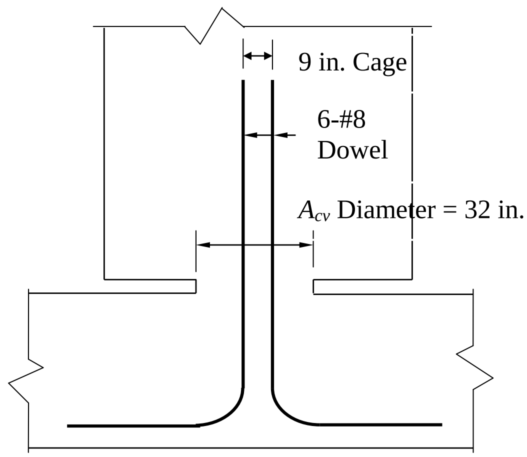


Figure 21.3-8 Column Shear Key

21.3.14 Step 12 - Check Bent Cap Flexural and Shear Capacity

21.3.14.1 Check Bent Cap Flexural Capacity

The design for strength limit states had resulted in the following main reinforcement for the bent cap:

Top Reinforcement	22 - #11 rebars
Bottom Reinforcement	24 - #11 rebars

Ignoring the side face reinforcement, the positive and negative flexural capacity of the bent cap is estimated to be M^{+ve} = 21,189 kip-ft and M^{-ve} = 19,436 kip-ft. Appendices 21.3-4 and 21.3-5 show these values, which are based on when either the concrete strain reaches 0.003 or the steel strain reaches ε_{SU}^{R} as required for capacity protected members (See *SDC* Section 3.4).

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 *wFRAME* pushover analysis of Bent 2 with column moment capacity taken as M_o. As shown in Appendix 21.3-14 (right pushover), bent cap moment demands are:

$$M_D^{+ve} = 14{,}350 \text{ kip}-\text{ft} < M^{+ve} = 21{,}189 \text{ kip}-\text{ft} \qquad \text{OK.}$$

$$M_D^{-ve} = 15{,}072 \text{ kip}-\text{ft} < M^{-ve} = 19{,}436 \text{ kip}-\text{ft} \qquad \text{OK.}$$

The associated shear demand obtained from the above pushover analysis, V_o = 2,009 kips.

21.3.14.2 Check Bent Cap Shear Capacity

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

$$V_n = V_c + V_s + V_p \qquad \text{(AASHTO 5.8.3.3-1)}$$

and

$$V_n = 0.25 f_c' b_v d_v + V_p \qquad \text{(AASHTO 5.8.3.3-2)}$$

where:

$$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v \qquad \text{(AASHTO 5.8.3.3-3)}$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \qquad \text{(AASHTO C5.8.3.3-1)}$$

V_p = 0 (bent cap is not prestressed)

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 5.8.2.9).

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

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

d_e = cap effective depth = 81-5-1.63/2 = 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 5.8.3.4 (AASHTO 2012) 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).

$\therefore$ Use β = 2.0 and θ = 45° per AASHTO Article 5.8.3.4.1.

$$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v = 0.0316(2)(\sqrt{4})(96)(67.7) = 821 \text{ kips}$$

Assuming 6-legged, #6 stirrups @ 7 in. o.c. transverse reinforcement (see Figure 21.3-19).

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} = \frac{6(0.44)(60)(67.7)(\cot 45)}{7} = 1{,}532\text{kips}$$
$$V_n = V_c + V_s = 821 + 1{,}532 = 2{,}353\text{kips}$$
$$V_n = 0.25 f_c^{'} b_v d_v = 0.25(4)(96)(67.7) = 6{,}499\text{kips} > \ 2{,}353\text{kips}$$
$$\therefore \ V_n = 2{,}353\text{kips}$$
$$\phi \times V_n = 0.9(2{,}353) = 2{,}118\text{kips} > V_0 = 2{,}009\text{kips} \qquad \text{OK.}$$

21.3.15 Step 13 - Calculate Column Seismic Load Moments

21.3.15.1 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 *CTBridge* (Table 21.3-1) 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 *wFRAME* results and to account for the bridge skew. To do so, the following coordinate transformation (see Figure 21.3-9) will be applied to the top of column moments from *CTBridge*.

Table 21.3-1 Top of Bent Column Moments (kip-ft) from *CTBridge*

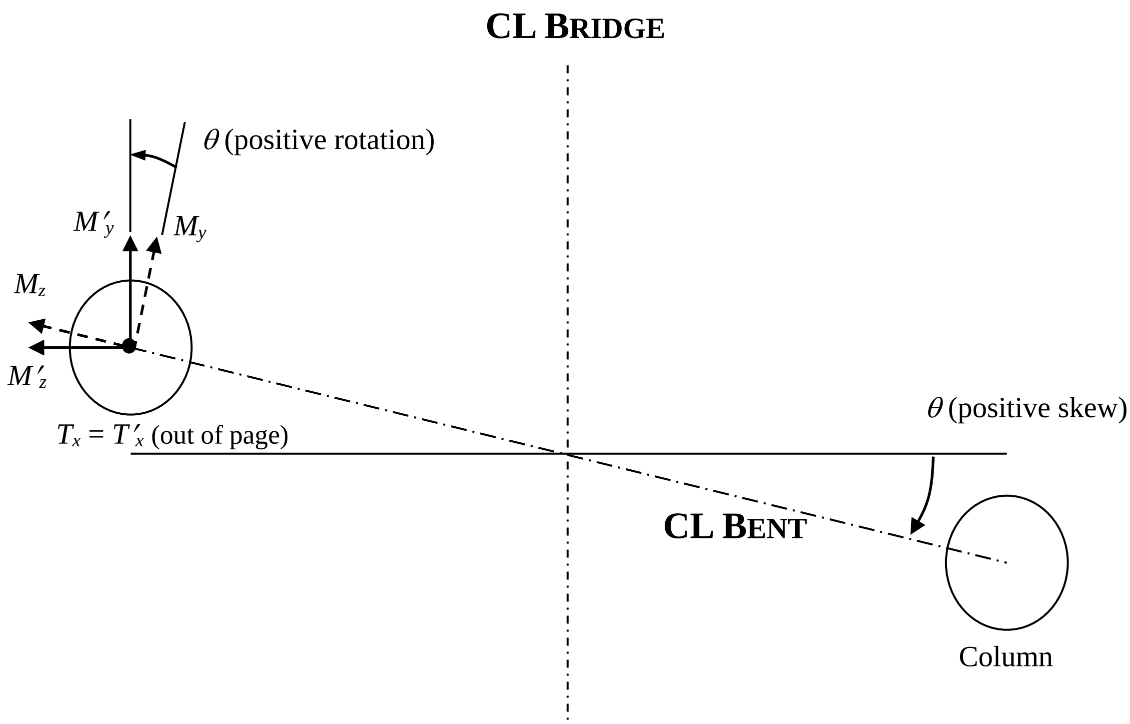
Bent	Skew (Degree)	*DL*			*ADL*			*Sec. PS*		
		M_z	M_y	M_{long}	M_z	M_y	M_{long}	M_z	M_y	M_{long}
2	20	-1,189	91	-1,148	-213	17	-207	82	-371	204
3	20	1,305	-1	1,227	234	-1	220	-127	287	-218

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 21.3-10)
  Column moment at base, $M_{dl}^{col,bottom} = 0$ kip-ft (CTBridge Output)
  Column moment at deck soffit,
  $M_{dl}^{col,top@joint} = (-1{,}148) + (-207) = -1{,}355$ kip-ft

- Secondary Prestress Moments *(*Figure *21.3-11)*
  Column moment at base, $M_{ps}^{col,bottom} = 0$ kip-ft (CTBridge Output)
  Column moment at deck soffit, $M_{ps}^{col,top@joint} = +204$ kip-ft



$$M'_z = M_z \cos\theta - M_y \sin\theta \qquad \text{(Longitudinal Moment)}$$

$$M'_y = M_z \sin\theta + M_y \cos\theta \qquad \text{(Transverse Moment)}$$

$$T'_x = T_x \qquad \text{(Torsional Moment)}$$

Figure 21.3-9 Coordinate Transformation from Skewed to Unskewed Configuration

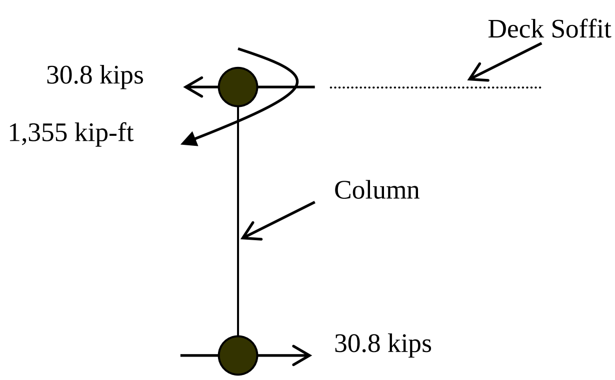


Figure 21.3-10 Free Body Diagram Showing Equilibrium of Dead Loading at Bent 2

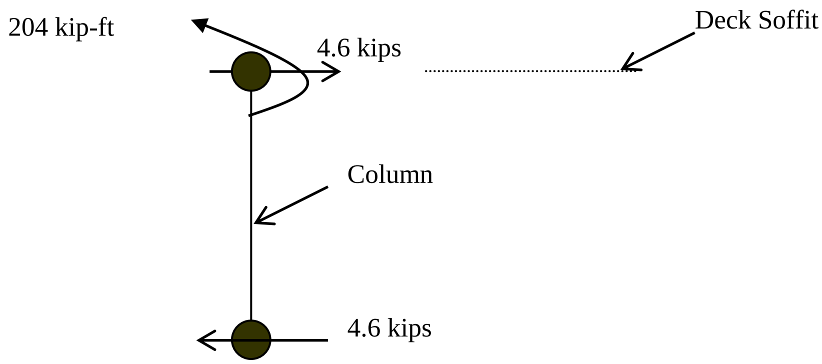


Figure 21.3-11 Free Body Diagram Showing Equilibrium of Secondary Prestress Forces at Bent 2

(2) Moment at Column Top - Bent 3

- Dead load and additional dead load moments

Column moment at base, $M_{dl}^{col,top@joint} = 0 \text{ kip-ft}$ (CTBridge Output)

Column moment at deck soffit,
$M_{dl}^{col,top@joint} = \{(+1{,}227)+(+220)\} = +1{,}447 \text{ kip-ft}$

- Secondary Prestress Moments

Column moment at base, $M_{ps}^{col,bottom} = 0 \text{ kip} - \text{ft}$ (CTBridge Output)

Column Moment at deck soffit, $M_{ps}^{col,top@joint} = -218 \text{ kip-ft}$

21.3.15.2 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 21.3-2).

(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 21.3-2).

Table 21.3-2 Dead Load and Secondary Prestress Moments from *CTBridge* Output

	Location		Whole Superstructure Width			Per Girder		
	x/L	x (ft)	M_{DL} (kip-ft)	M_{ADL} (kip-ft)	M_{PS} (kip-ft)	M_{DL} (kip-ft)	M_{ADL} (kip-ft)	M_{PS} (kip-ft)
Span 1	Support	1.5	619	114	647	124	23	129
	0.1	12.6	7110	1275	1462	1422	255	292
	0.2	25.2	12158	2178	2272	2432	436	454
	0.3	37.8	14741	2640	3096	2948	528	619
	0.4	50.4	14857	2661	3956	2971	532	791
	0.5	63	12508	2240	4705	2502	448	941
	0.6	75.6	7693	1377	5617	1539	275	1123
	0.7	88.2	412	74	6400	82	15	1280
	0.8	100.8	-9334	-1671	7911	-1867	-334	1582
	0.9	113.4	-21553	-3857	8498	-4311	-771	1700
	Support	123	-32599	-5819	8672	-6520	-1164	1734
Span 2	Support	129	-33654	-6009	8468	-6731	-1202	1694
	0.1	142.8	-17502	-3136	9516	-3500	-627	1903
	0.2	159.6	-1955	-354	9005	-391	-71	1801
	0.3	176.4	9208	1645	8318	1842	329	1664
	0.4	193.2	15989	2859	8281	3198	572	1656
	0.5	210	18388	3289	8027	3678	658	1605
	0.6	226.8	16406	2935	8072	3281	587	1614
	0.7	243.6	10043	1795	7905	2009	359	1581
	0.8	260.4	-699	-128	8355	-140	-26	1671
	0.9	277.2	-15820	-2835	8645	-3164	-567	1729
	Support	291	-31614	-5646	7554	-6323	-1129	1511
Span 3	Support	297	-30429	-5434	7482	-6086	-1087	1496
	0.1	305.8	-20789	-3723	7275	-4158	-745	1455
	0.2	317.6	-9854	-1766	6861	-1971	-353	1372
	0.3	329.4	-1093	-197	5559	-219	-39	1112
	0.4	341.2	5506	986	4870	1101	197	974
	0.5	353	9943	1781	4085	1989	356	817
	0.6	364.8	12219	2189	3417	2444	438	683
	0.7	376.6	12333	2210	2669	2467	442	534
	0.8	388.4	10286	1844	1945	2057	369	389
	0.9	400.2	6077	1091	1230	1215	218	246
	Support	410.5	637	117	529	127	23	106

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

As shown in Figure 21.3-12, such loading results in positive moments in the columns according to the sign convention used here.

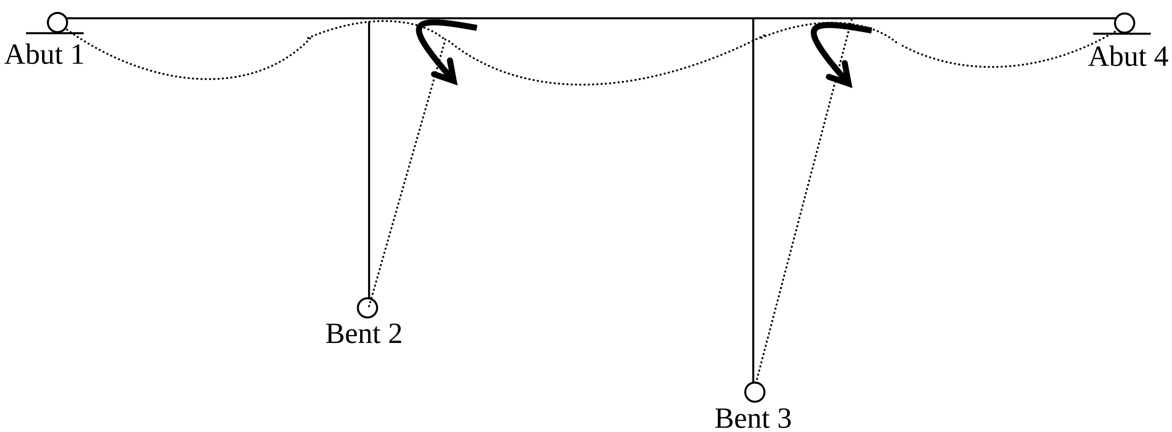


Figure 21.3-12 Seismic Loading Case "1" Producing Positive Moments in Columns

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

$$M_{dl}^{col@soffit} + M_{ps}^{col@soffit} = \{(-1{,}355) + (+204)\} = -1{,}151\,\text{kip-ft} \ \ \text{(Bent 2)}$$

$$M_{dl}^{col@soffit} + M_{ps}^{col@soffit} = \{(1{,}447) + (-218)\} = +1229\,\text{kip-ft} \ \ \text{(Bent 3)}$$

Column moment generated by seismic loading at column soffit is:

$$M_{eq}^{col@soffit} = 1.2M_{p}^{col@soffit}(M_{dl}^{col} + M_{ps}^{col@soffit})$$

$$= 1.2(2)(13{,}838) - \{(-1{,}355) + 0)\} = +34{,}566\,\text{kip-ft} \ \ \text{(Bent 2)}$$

It should be noted that the secondary prestress moment is neglected because doing so results in increased seismic demand on the column and hence in the superstructure. Figure 21.3-13 schematically explains this superposition approach.

$$M_{eq}^{col@soffit} = 1.2(2)(13{,}777) - (1{,}447 - 218) = 31{,}835\ \text{kip-ft} \ \ \ \text{(Bent 3)}$$

It should be noted that for Bent 3, the effect of secondary prestress moments is included because doing so results in increased seismic moment in the columns and hence in the superstructure.

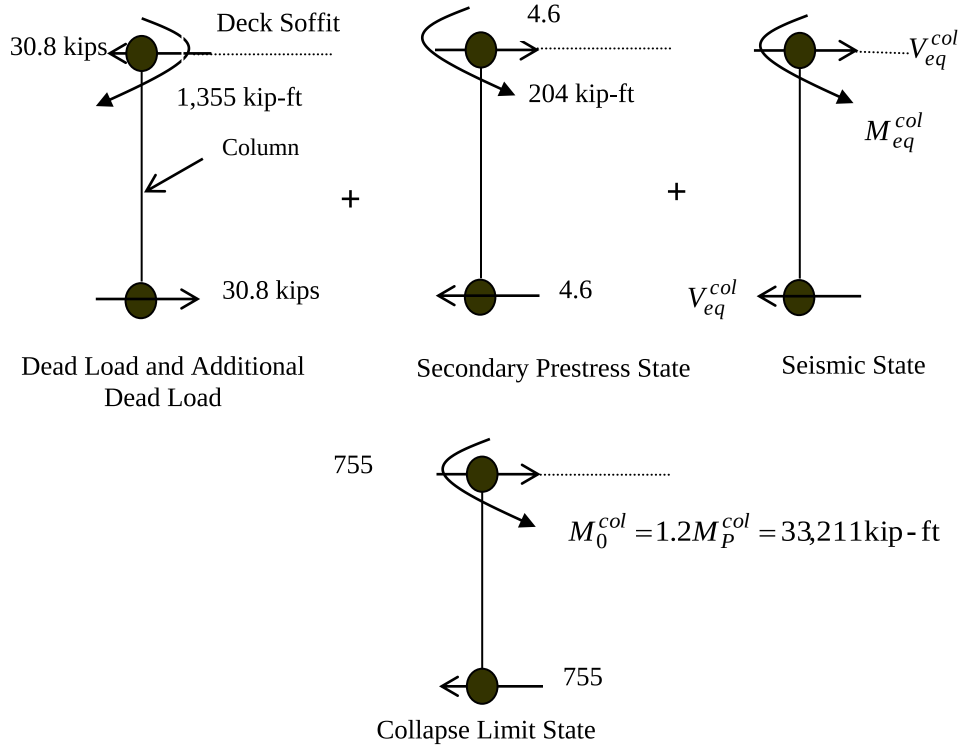


Figure 21.3-13 Superposition of Column Forces at Bent 2 for Loading Case "1"

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

As shown in Figure 21.3-14, such loading results in negative moments in the columns according to our sign convention.

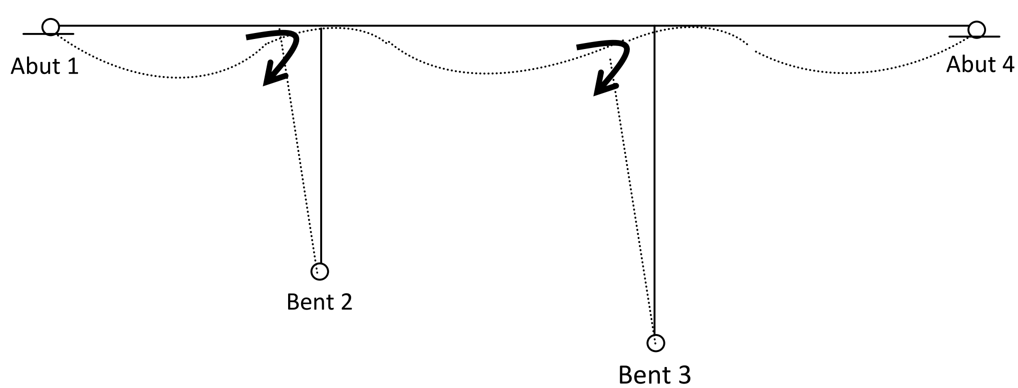


Figure 21.3-14 Seismic Loading Case "2" Producing Negative Moments in Columns

Bent 2

$$M_{eq}^{col@soffit} = 1.2M_p^{col} - (M_{dl}^{col} + M_{ps}^{col})$$

$$= 1.2(2)(-13{,}838) - (-1{,}355 + 204) = -32{,}060 \text{ kip-ft}$$

Bent 3

$$M_{eq}^{col@soffit} = 1.2M_p^{col} - (M_{dl}^{col} + M_{ps}^{col})$$

$$= 1.2(2)(-13{,}777) - (1{,}447 - 0) = -34{,}512 \text{ kip-ft}$$

Figure 21.3-15 schematically shows the Free Body Diagram at Bent 2 for this seismic loading case.

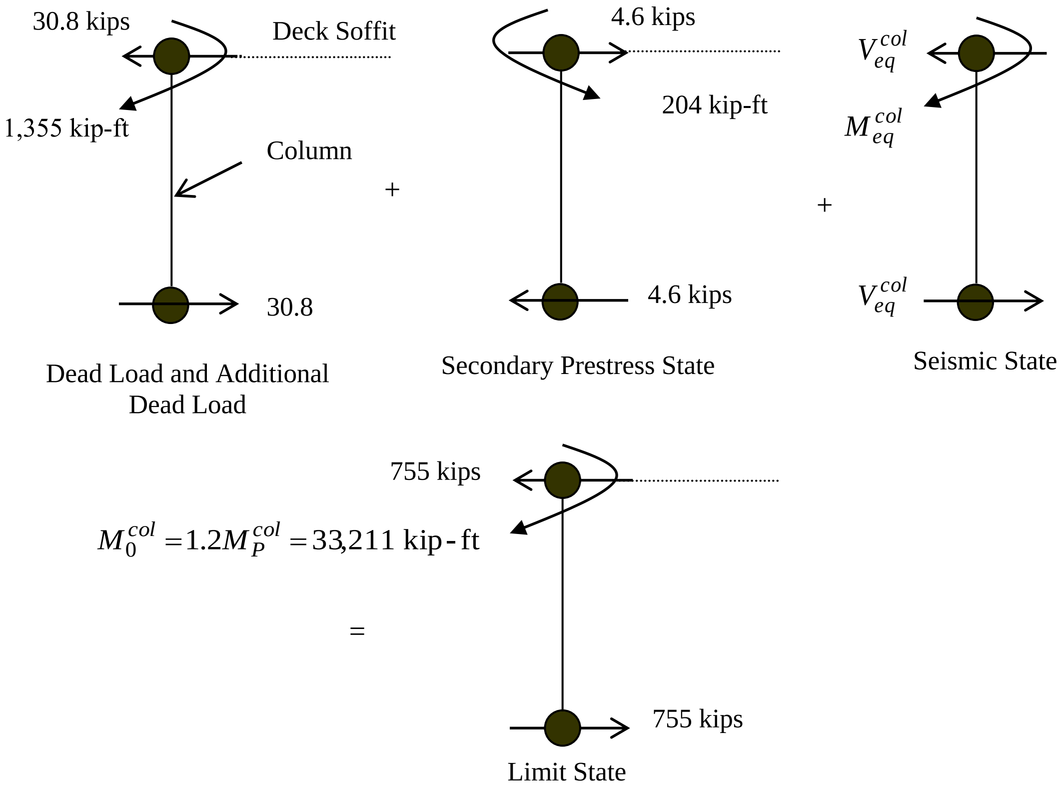


Figure 21.3-15 Superposition of Column Forces at Bent 2 for Loading Case "2"

21.3.16 Step 14 - Distribute $M_{eq}^{col@soffit}$ into the Superstructure

The static non-linear "push-over" frame analysis program *wFRAME* is used to distribute the column earthquake moments $M_{eq}^{col@soffit}$ into the superstructure.

Note the difference in sign convention between the *wFRAME* model and the one adopted here. Therefore, for the input file, the positive column earthquake moments corresponding to "Case 1" loading are used as negative column moment capacities for pushover analysis while the negative column earthquake moments corresponding to "Case 2" are modeled as positive column moment capacities. Also, the superstructure dead load is removed from the *wFRAME* model. Appendix 21.3-15 shows portions of the output file for Case 1 (i.e., right push). Table 21.3-3 lists the distribution of earthquake moments in the superstructure as obtained from these pushover analyses.

21.3.17 Step 15 - Calculate Superstructure Seismic Moment Demand at Location of Interest

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

(1) Example Calculation - Bent 2: Left and Right Faces of Bent Cap

The effective section width is:

$$b_{eff} = D_c + 2D_s = 6.00 + 2(6.75) = 19.50 \text{ ft.} \qquad (SDC\ 7.2.1.1\text{-}1)$$

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 bent cap, four girders are within the effective section. All five girders fall within the effective width for all the other tenth point locations (see Table 21.3-4). Note that the per-girder values used below have previously been listed in Table 21.3-2.

<u>Case 1</u>

$$M_{dl}^{L} = \{(-6{,}520) + (-1{,}164)\}(4) = -30{,}736 \text{ kip-ft}$$

$$M_{dl}^{R} = \{(-6{,}731) + (-1{,}202)\}(4) = -31{,}732 \text{ kip-ft}$$

$$M_{ps}^{L} = \{+1{,}734\}(4) = +6{,}936 \text{ kip-ft}$$

$$M_{ps}^{R} = \{+1{,}694\}(4) = +6{,}776 \text{ kip-ft}$$

$$M_{eq}^{L} = -15{,}015 \text{ kip-ft} \text{ (see Table 21.3-3)}$$

$$M_{eq}^{R} = +21{,}135 \text{ kip-ft} \text{ (see Table 21.3-3)}$$

The superstructure moment demand is then calculated as:

$$M_D^L = M_{dl}^L + M_{ps}^L + M_{eq}^L = (\text{-}30{,}736) + (\cancel{6{,}936}^{*}) + (\text{-}15{,}015) = \text{-}45{,}751 \text{ kip-ft}$$

$$M_D^R = M_{dl}^R + M_{ps}^R + M_{eq}^R = (\text{-}31{,}732) + (6{,}776) + (21{,}135) = \text{-}3{,}821 \text{ kip-ft}$$

Table 21.3-4 lists these superstructure seismic moment demands.

<u>Case 2</u>

$$M_{eq}^L = +13{,}201 \text{ kip-ft}; \quad M_{eq}^R = \text{-}20{,}299 \text{ kip-ft}$$

$$M_D^L = (\text{-}30{,}736) + (6{,}936) + (13{,}201) = \text{-}10{,}599 \text{ kip-ft}$$

$$M_D^R = (\text{-}31{,}732) + (\cancel{6{,}776}^{*}) + (\text{-}20{,}295) = \text{-}52{,}027 \text{ kip-ft}$$

*The prestressing secondary effect is ignored as doing so results in a conservatively higher seismic demand in the superstructure.

(2) *Bent 3*

Similarly, we obtain the following:

$$M_D^L = \begin{cases} -49{,}702 \text{ kip-ft} & \text{Case 1} \\ -3{,}001 \text{ kip-ft} & \text{Case 2} \end{cases}$$

$$M_D^R = \begin{cases} -9{,}434 \text{ kip-ft} & \text{Case 1} \\ -43{,}915 \text{ kip-ft} & \text{Case 2} \end{cases}$$

Seismic moment demands along the superstructure length have been summarized in the form of moment envelope values (see Table 21.3-4).

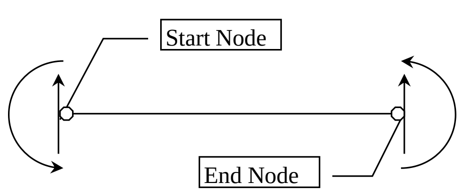
$$M_{positive} = M_{EQ,\max} + M_{DL} + M_{ADL} + M_{ps}^{*}$$

$$M_{negative} = M_{EQ,\min} + M_{DL} + M_{ADL} + M_{ps}^{**}$$

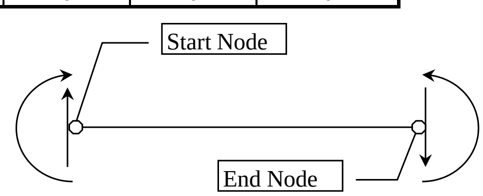
* Only include M_{ps} when it maximizes $M_{positive}$
** Only include M_{ps} when it minimizes $M_{negative}$

Table 21.3-3 Earthquake Moments from *wFRAME* Output

Location		M_{EQ} (kip-ft)			
		wFRAME Convention		Standard Convention	
		Case 1	Case 2	Case 1	Case 2
Span 1	0.0	0	0	0	0
	Support			-183	161
	0.1			-1538	1352
	0.2			-3076	2705
	0.3			-4614	4057
	0.4			-6152	5409
	0.5			-7691	6761
	0.6			-9229	8114
	0.7			-10767	9466
	0.8			-12305	10818
	0.9			-13843	12170
	Support			-15015	13201
	1.0	-15381	13523	-15381	13523
Span 2	0.0	-21895	21055	21895	-21055
	Support			21135	-20295
	0.1			17640	-16798
	0.2			13385	-12540
	0.3			9131	-8282
	0.4			4876	-4024
	0.5			621	234
	0.6			-3634	4492
	0.7			-7889	8750
	0.8			-12144	13008
	0.9			-16399	17266
	Support			-19894	20763
	1.0	-20653	21524	-20653	21524
Span 3	0.0	-13620	15621	13620	-15621
	Support			13274	-15223
	0.1			12258	-14059
	0.2			10896	-12496
	0.3			9534	-10934
	0.4			8172	-9372
	0.5			6810	-7810
	0.6			5448	-6248
	0.7			4086	-4686
	0.8			2724	-3124
	0.9			1362	-1562
	Support			173	-199
	1.0	0	0	0	0



wFRAME Positive Convention



Standard Positive Convention

Table 21.3-4 Moment Demand Envelope

Location			No. of Girders in Effective Section				Case 1	Case 2	Case 1		Case 2		Envelope	
				M_{DL}	M_{ADL}	M_{PS}	M_{EQ}	M_{EQ}	$M_{postive}$	$M_{negative}$	$M_{positive}$	$M_{negative}$	$M_{positive}$	$M_{negative}$
	x/L	x (ft)	4	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)
Span 1	Support	1.5	4	496	91	517	-183	161	921	403	1265	748	1265	403
	0.1	12.6	5	7110	1275	1462	-1538	1352	8309	6847	11199	9737	11199	6847
	0.2	25.2	5	12158	2178	2272	-3076	2705	13532	11260	19313	17041	19313	11260
	0.3	37.8	5	14741	2640	3096	-4614	4057	15862	12766	24533	21438	24533	12766
	0.4	50.4	5	14857	2661	3956	-6152	5409	15321	11365	26883	22927	26883	11365
	0.5	63.0	5	12508	2240	4705	-7691	6761	11762	7057	26213	21509	26213	7057
	0.6	75.6	5	7693	1377	5617	-9229	8114	5459	-159	22801	17184	22801	-159
	0.7	88.2	5	412	74	6400	-10767	9466	-3881	-10281	16352	9952	16352	-10281
	0.8	100.8	5	-9334	-1671	7911	-12305	10818	-15399	-23310	7724	-187	7724	-23310
	0.9	113.4	5	-21553	-3857	8498	-13843	12170	-30755	-39254	-4742	-13240	-4742	-39254
	Support	123.0	4	-26079	-4656	6937	-15015	13201	-38815	-45751	-10599	-17535	-10599	-45751
Span 2	Support	129.0	4	-26923	-4807	6774	21135	-20295	-3821	-10597	-45251	-52027	-3821	-52027
	0.1	142.8	5	-17502	-3136	9516	17640	-16798	6518	-2998	-27920	-37436	6518	-37436
	0.2	159.6	5	-1955	-354	9005	13385	-12540	20082	11077	-5843	-14848	20082	-14848
	0.3	176.4	5	9208	1645	8318	9131	-8282	28301	19984	10889	2571	28301	2571
	0.4	193.2	5	15989	2859	8281	4876	-4024	32005	23724	23105	14824	32005	14824
	0.5	210.0	5	18388	3289	8027	621	234	30324	22297	29938	21911	30324	21911
	0.6	226.8	5	16406	2935	8072	-3634	4492	23779	15707	31905	23833	31905	15707
	0.7	243.6	5	10043	1795	7905	-7889	8750	11854	3950	28493	20588	28493	3950
	0.8	260.4	5	-699	-128	8355	-12144	13008	-4616	-12970	20536	12181	20536	-12970
	0.9	277.2	5	-15820	-2835	8645	-16399	17266	-26409	-35054	7256	-1390	7256	-35054
	Support	291.0	4	-25291	-4517	6043	-19894	20763	-43658	-49702	-3001	-9045	-3001	-49702
Span 3	Support	297.0	4	-24344	-4347	5986	13274	-15223	-9434	-15418	-37931	-43915	-9434	-43915
	0.1	305.8	5	-20789	-3723	7275	12258	-14059	-4979	-12254	-31296	-38571	-4979	-38571
	0.2	317.6	5	-9854	-1766	6861	10896	-12496	6138	-724	-17255	-24116	6138	-24116
	0.3	329.4	5	-1093	-197	5559	9534	-10934	13804	8244	-6665	-12224	13804	-12224
	0.4	341.2	5	5506	986	4870	8172	-9372	19533	14663	1988	-2881	19533	-2881
	0.5	353.0	5	9943	1781	4085	6810	-7810	22619	18534	7999	3913	22619	3913
	0.6	364.8	5	12219	2189	3417	5448	-6248	23273	19856	11576	8159	23273	8159
	0.7	376.6	5	12333	2210	2669	4086	-4686	21298	18629	12526	9857	21298	9857
	0.8	388.4	5	10286	1844	1945	2724	-3124	16799	14854	10950	9006	16799	9006
	0.9	400.2	5	6077	1091	1230	1362	-1562	9760	8530	6836	5606	9760	5606
	Support	410.5	4	509	94	423	173	-199	1199	776	827	404	1199	404

21.3.18 Step 16 - Calculate Superstructure Seismic Shear Demand at Location of Interest

Values of shear forces due to dead load, additional dead load, and secondary prestress, as read from *CTBridge* output, are listed in Table 21.3-5.

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

Span 1, Case 1

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

Seismic Moment at Bent 2 $M_{eq}^{(2)} = -15{,}381 \text{ kip-ft}$

Shear force in Span 1, $V_{eq} = \dfrac{\left(M_{eq}^{(2)} + M_{eq}^{(1)}\right)}{\text{Length of Span 1}} = \dfrac{(-15{,}381 + 0)}{126} = -122 \text{ kips}$

Span 1, Case 2

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

Seismic Moment at Bent 2, $M_{eq}^{(2)} = 13{,}523 \text{ kip-ft}$

Shear force in Span 1, $V_{eq} = \dfrac{\left(M_{eq}^{(2)} + M_{eq}^{(1)}\right)}{\text{Length of Span 1}} = \dfrac{(13{,}523 + 0)}{126} = 107 \text{kips}$

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

Span 2, $V_{eq} = \begin{cases} -253 \text{ kips} & \text{Case 1} \\ +253 \text{ kips} & \text{Case 2} \end{cases}$

Span 3, $V_{eq} = \begin{cases} -115 \text{ kips} & \text{Case 1} \\ +133 \text{ kips} & \text{Case 2} \end{cases}$

Table 21.3-6 lists these values. Table 21.3-7 lists the maximum shear demands summarized as a shear envelope.

$$V_{positive} = V_{EQ,\max} + V_{DL} + V_{ADL} + V_{ps}^{*}$$

$$V_{negative} = V_{EQ,\min} + V_{DL} + V_{ADL} + V_{ps}^{**}$$

$$V_{\max} = \text{Greater of Absolute}(V_{posittive}) \text{ or Absolute}(V_{negative})$$

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

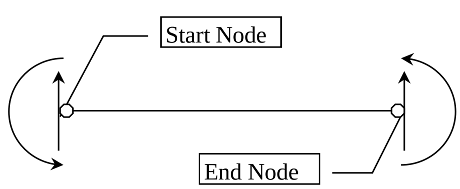
** Only include V_{PS} when it minimizes $V_{negative}$

Table 21.3-5 Dead Load and Secondary Prestress Shears Forces from *CTBridge* Output

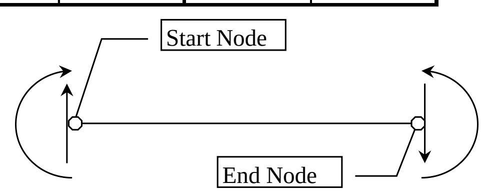
Location			Whole Superstructure Width			Per Girder		
			V_{DL}	V_{ADL}	V_{PS}	V_{DL}	V_{ADL}	V_{PS}
	x/L	*x (ft)*	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)
Span 1	Support	1.5	671	120	79	134	24	16
	0.1	12.6	498	89	78	100	18	16
	0.2	25.2	303	54	76	61	11	15
	0.3	37.8	107	19	76	21	4	15
	0.4	50.4	-89	-16	75	-18	-3	15
	0.5	63.0	-284	-51	75	-57	-10	15
	0.6	75.6	-480	-86	75	-96	-17	15
	0.7	88.2	-675	-121	75	-135	-24	15
	0.8	100.8	-871	-156	75	-174	-31	15
	0.9	113.4	-1070	-191	30	-214	-38	6
	Support	123.0	-1232	-218	134	-246	-44	27
Span 2	Support	129.0	1287	227	-44	257	45	-9
	0.1	142.8	1056	189	-22	211	38	-4
	0.2	159.6	795	142	2	159	28	0
	0.3	176.4	534	96	2	107	19	0
	0.4	193.2	273	49	2	55	10	0
	0.5	210.0	13	2	2	3	0	0
	0.6	226.8	-248	-45	1	-50	-9	0
	0.7	243.6	-509	-91	1	-102	-18	0
	0.8	260.4	-770	-138	1	-154	-28	0
	0.9	277.2	-1031	-185	-28	-206	-37	-6
	Support	291.0	-1261	-223	37	-252	-45	7
Span 3	Support	297.0	1171	207	-118	234	41	-24
	0.1	305.8	1021	182	-69	204	36	-14
	0.2	317.6	834	149	-48	167	30	-10
	0.3	329.4	651	117	-48	130	23	-10
	0.4	341.2	468	84	-48	94	17	-10
	0.5	353.0	284	51	-49	57	10	-10
	0.6	364.8	101	18	-48	20	4	-10
	0.7	376.6	-82	-15	-48	-16	-3	-10
	0.8	388.4	-265	-47	-48	-53	-9	-10
	0.9	400.2	-448	-80	-48	-90	-16	-10
	Support	410.5	-608	-109	-68	-122	-22	-14

Table 21.3-6 Earthquake Shear Forces from *wFRAME* Output

	Location		V_{EQ} (kip)			
			wFRAME Convention		Standard Convention	
			Case 1	Case 2	Case 1	Case 2
Span 1	0	0.0	-122	107	-122	107
	Support	1.5	0	0	-122	107
	0.1	12.6	0	0	-122	107
	0.2	25.2	0	0	-122	107
	0.3	37.8	0	0	-122	107
	0.4	50.4	0	0	-122	107
	0.5	63.0	0	0	-122	107
	0.6	75.6	0	0	-122	107
	0.7	88.2	0	0	-122	107
	0.8	100.8	0	0	-122	107
	0.9	113.4	0	0	-122	107
	Support	123.0	0	0	-122	107
	1	126.0	-122	107	-122	107
Span 2	0	126.0	-253	253	-253	253
	Support	129.0	0	0	-253	253
	0.1	142.8	0	0	-253	253
	0.2	159.6	0	0	-253	253
	0.3	176.4	0	0	-253	253
	0.4	193.2	0	0	-253	253
	0.5	210.0	0	0	-253	253
	0.6	226.8	0	0	-253	253
	0.7	243.6	0	0	-253	253
	0.8	260.4	0	0	-253	253
	0.9	277.2	0	0	-253	253
	Support	291.0	0	0	-253	253
	1	294.0	-253	253	-253	253
Span 3	0	294.0	-115	133	-115	133
	Support	297.0	0	0	-115	133
	0.1	305.8	0	0	-115	133
	0.2	317.6	0	0	-115	133
	0.3	329.4	0	0	-115	133
	0.4	341.2	0	0	-115	133
	0.5	353.0	0	0	-115	133
	0.6	364.8	0	0	-115	133
	0.7	376.6	0	0	-115	133
	0.8	388.4	0	0	-115	133
	0.9	400.2	0	0	-115	133
	Support	410.5	0	0	-115	133
	1	412.0	-115	133	-115	133



wFRAME Positive Convention



Standard Positive Convention

Table 21.3-7 Shear Demand Envelope

Location			No. of Girders in Effective Section				Case 1	Case 2	Case 1		Case 2		Envelope		
				V_{DL}	V_{ADL}	V_{PS}	V_{EQ}	V_{EQ}	$V_{positive}$	$V_{negative}$	$V_{positive}$	$V_{negative}$	$V_{positive}$	$V_{negative}$	V_{max}
	x/L	x (ft)		(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)
Span 1	Support	12.6	5	498	89	78	-122	107	543	465	772	694	772	465	772
	0.1	12.6	5	498	89	78	-122	107	543	465	772	694	772	465	772
	0.2	25.2	5	303	54	76	-122	107	311	235	540	464	540	235	540
	0.3	37.8	5	107	19	76	-122	107	80	4	309	233	309	4	309
	0.4	50.4	5	-89	-16	75	-122	107	-151	-227	78	3	78	-227	227
	0.5	63.0	5	-284	-51	75	-122	107	-382	-457	-153	-228	-153	-457	457
	0.6	75.6	5	-480	-86	75	-122	107	-613	-688	-384	-459	-384	-688	688
	0.7	88.2	5	-675	-121	75	-122	107	-843	-918	-614	-689	-614	-918	918
	0.8	100.8	5	-871	-156	75	-122	107	-1075	-1149	-846	-920	-846	-1149	1149
	0.9	113.4	5	-1070	-191	30	-122	107	-1354	-1383	-1125	-1154	-1125	-1383	1383
	Support	123.0	4	-986	-174	107	-122	107	-1175	-1282	-946	-1053	-946	-1282	1282
Span 2	Support	129.0	4	1029	182	-35	-253	253	958	923	1464	1429	1464	923	1464
	0.1	142.8	5	1056	189	-22	-253	253	992	971	1498	1477	1498	971	1498
	0.2	159.6	5	795	142	2	-253	253	686	684	1192	1190	1192	684	1192
	0.3	176.4	5	534	96	2	-253	253	378	377	884	883	884	377	884
	0.4	193.2	5	273	49	2	-253	253	71	69	577	575	577	69	577
	0.5	210.0	5	13	2	2	-253	253	-237	-238	269	268	269	-238	269
	0.6	226.8	5	-248	-45	1	-253	253	-544	-546	-38	-40	-38	-546	546
	0.7	243.6	5	-509	-91	1	-253	253	-852	-853	-346	-347	-346	-853	853
	0.8	260.4	5	-770	-138	1	-253	253	-1160	-1161	-654	-655	-654	-1161	1161
	0.9	277.2	5	-1031	-185	-28	-253	253	-1469	-1496	-963	-990	-963	-1496	1496
	Support	291.0	4	-1009	-178	30	-253	253	-1411	-1440	-905	-934	-905	-1440	1440
Span 3	Support	297.0	4	937	165	-94	-115	133	987	893	1234	1141	1234	893	1234
	0.1	305.8	5	1021	182	-69	-115	133	1088	1020	1336	1267	1336	1020	1336
	0.2	317.6	5	834	149	-48	-115	133	868	820	1116	1068	1116	820	1116
	0.3	329.4	5	651	117	-48	-115	133	652	604	900	852	900	604	900
	0.4	341.2	5	468	84	-48	-115	133	436	388	684	636	684	388	684
	0.5	353.0	5	284	51	-49	-115	133	220	172	468	420	468	172	468
	0.6	364.8	5	101	18	-48	-115	133	4	-44	252	204	252	-44	252
	0.7	376.6	5	-82	-15	-48	-115	133	-212	-259	36	-11	36	-259	259
	0.8	388.4	5	-265	-47	-48	-115	133	-428	-476	-180	-228	-180	-476	476
	0.9	400.2	5	-448	-80	-48	-115	133	-644	-692	-396	-444	-396	-692	692
	Support	410.5	4	-486	-87	-54	-115	133	-689	-743	-441	-495	-441	-743	743

21.3.19 Step 17 - Perform Vertical Acceleration Analysis

Since the site PRA = 0.5g < 0.6g, vertical acceleration analysis is not required.

21.3.20 Step 18 - Calculate Superstructure Flexural and Shear Capacity

21.3.20.1 Superstructure Flexural Capacity

Table 21.3-8 lists the data that will be used to calculate the flexural section capacity using the computer program *PSSECx*. Symbols in Table 21.3-8 are shown in Figure 21.3-16. Appendix 21.3-16 lists the *PSSECx* input for the superstructure section that lies just to the left of Bent 2. The model is shown in Appendix 21.3-17. The results for negative capacity calculations are shown in Appendix 21.3-18. The limiting condition for flexural capacity in this case was the steel reaching its maximum allowable strain.

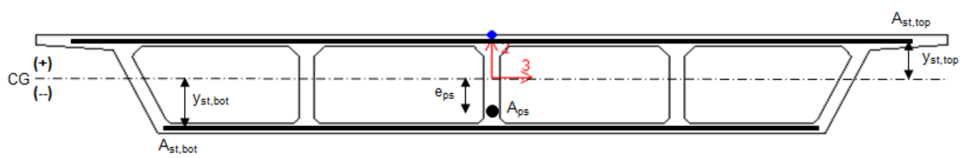


Figure 21.3-16 Typical Superstructure Cross Section

PSSECx was run repeatedly to calculate superstructure flexural capacities at various points along the span length. Table 21.3-9 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. The worst *D/C* ratio of 0.63 suggests overdesign. If such case is found across a broad spectrum of various Caltrans bridges, perhaps the requirement of #8 spaced 12 in. may be revised in the future.

Table 21.3-8 Section Flexural Capacity Calculation Data

Location	x/L	x (ft)	No. Girders	No. Girders in Effective Section	Eccentricity e_{ps} (in.)	PS Force After All Losses (kip)	For Effective Section: PS Force After All Losses (kip)	For Effective Section: Area of PS A_{ps} (in.^{2})	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 Bottom Mild Steel $y_{st,bot}$ (in.)
Span 1	Support	1.5	5	4	-2.6628	7439	5952	38.28	8.00	31.80	6.00	-42.13
	0.1	12.6	5	5	-14.9760	7508	7508	47.85	8.00	31.80	6.00	-42.13
	0.2	25.2	5	5	-25.1328	7582	7582	47.85	8.00	31.80	6.00	-42.13
	0.3	37.8	5	5	-31.2264	7650	7650	47.85	8.00	31.80	6.00	-42.13
	0.4	50.4	5	5	-33.2568	7712	7712	47.85	8.00	31.80	6.00	-42.13
	0.5	63.0	5	5	-31.4076	7766	7766	47.85	47.40	31.80	34.76	-42.13
	0.6	75.6	5	5	-25.8576	7814	7814	47.85	47.40	31.80	34.76	-42.13
	0.7	88.2	5	5	-16.6068	7859	7859	47.85	47.40	31.80	34.76	-42.13
	0.8	100.8	5	5	-3.6576	7839	7839	47.85	47.40	31.80	34.76	-42.13
	0.9	113.4	5	5	14.9160	7765	7765	47.85	47.40	31.80	34.76	-42.13
	Support	123.0	5	4	25.4412	7697	6157	38.28	47.40	31.80	34.76	-42.13
Span 2	Support	129.0	5	4	25.6116	7595	6076	38.28	47.40	31.80	34.76	-42.13
	0.1	142.8	5	5	12.0432	7413	7413	47.85	47.40	31.80	34.76	-42.13
	0.2	159.6	5	5	-8.2824	7370	7370	47.85	47.40	31.80	34.76	-42.13
	0.3	176.4	5	5	-22.1568	7327	7327	47.85	47.40	31.80	34.76	-42.13
	0.4	193.2	5	5	-30.4824	7272	7272	47.85	8.00	31.80	6.00	-42.13
	0.5	210.0	5	5	-33.2568	7212	7212	47.85	8.00	31.80	6.00	-42.13
	0.6	226.8	5	5	-30.4824	7148	7148	47.85	8.00	31.80	6.00	-42.13
	0.7	243.6	5	5	-22.1568	7079	7079	47.85	47.40	31.80	34.76	-42.13
	0.8	260.4	5	5	-8.2824	6999	6999	47.85	47.40	31.80	34.76	-42.13
	0.9	277.2	5	5	12.0432	6922	6922	47.85	47.40	31.80	34.76	-42.13
	Support	291.0	5	4	25.6116	6844	5475	38.28	47.40	31.80	34.76	-42.13
Span 3	Support	297.0	5	4	25.3668	6742	5393	38.28	47.40	31.80	34.76	-42.13
	0.1	305.8	5	5	15.1068	6572	6572	47.85	47.40	31.80	34.76	-42.13
	0.2	317.6	5	5	-3.6576	6545	6545	47.85	47.40	31.80	34.76	-42.13
	0.3	329.4	5	5	-16.6068	6522	6522	47.85	47.40	31.80	34.76	-42.13
	0.4	341.2	5	5	-25.8576	6484	6484	47.85	47.40	31.80	34.76	-42.13
	0.5	353.0	5	5	-31.4076	6443	6443	47.85	47.40	31.80	34.76	-42.13
	0.6	364.8	5	5	-33.2568	6398	6398	47.85	8.00	31.80	6.00	-42.13
	0.7	376.6	5	5	-31.2264	6345	6345	47.85	8.00	31.80	6.00	-42.13
	0.8	388.4	5	5	-25.1328	6287	6287	47.85	8.00	31.80	6.00	-42.13
	0.9	400.2	5	5	-14.9760	6225	6225	47.85	8.00	31.80	6.00	-42.13
	Support	410.5	5	4	-2.7900	6174	4940	38.28	8.00	31.80	6.00	-42.13

P_{jack} = 9,689 kips

* Area of mild steel based on minimum seismic requirement only

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

Table 21.3-9 Section Flexural Capacity Calculation Data

Location			Moment Demand		Moment Capacity		*D/C* Ratio	
			$M_{positive}$	$M_{negative}$	$M_{positive}$	$M_{negative}$	Postive Moment	Negative Moment
	x/L	x (ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)		
Span 1	Support	1.5	1265	403	34530	-39444	0.04	0.00
	0.1	12.6	11199	6847	54933	-34040	0.20	0.00
	0.2	25.2	19313	11260	65503	-23133	0.29	0.00
	0.3	37.8	24533	12766	72025	-16558	0.34	0.00
	0.4	50.4	26883	11365	73892	-14352	0.36	0.00
	0.5	63.0	26213	7057	86109	-36067	0.30	0.00
	0.6	75.6	22801	-159	81016	-41879	0.28	0.00
	0.7	88.2	16352	-10281	71684	-51573	0.23	0.20
	0.8	100.8	7724	-23310	58705	-65648	0.13	0.36
	0.9	113.4	-4742	-39254	38646	-85898	0.00	0.46
	Support	123.0	-10599	-45751	26587	-81787	0.00	0.56
Span 2	Support	129.0	-3821	-52027	26432	-81933	0.00	0.63
	0.1	142.8	6518	-37436	41672	-82802	0.16	0.45
	0.2	159.6	20082	-14848	63619	-60763	0.32	0.24
	0.3	176.4	28301	2571	77024	-45653	0.37	0.00
	0.4	193.2	32005	14824	71217	-17311	0.45	0.00
	0.5	210.0	30324	21911	73881	-14256	0.41	0.00
	0.6	226.8	31905	15707	71218	-17293	0.45	0.00
	0.7	243.6	28493	3950	77020	-45591	0.37	0.00
	0.8	260.4	20536	-12970	63615	-60698	0.32	0.21
	0.9	277.2	7256	-35054	41623	-82794	0.17	0.42
	Support	291.0	-3001	-49702	26344	-81924	0.00	0.61
Span 3	Support	297.0	-9434	-43915	26540	-81708	0.00	0.54
	0.1	305.8	-4979	-38571	38316	-86086	0.00	0.45
	0.2	317.6	6138	-24116	58628	-65419	0.10	0.37
	0.3	329.4	13804	-12224	71666	-51881	0.19	0.24
	0.4	341.2	19533	-2881	80998	-41529	0.24	0.07
	0.5	353.0	22619	3913	86086	-35585	0.26	0.00
	0.6	364.8	23273	8159	74193	-13993	0.31	0.00
	0.7	376.6	21298	9857	72006	-16314	0.30	0.00
	0.8	388.4	16799	9006	65484	-22996	0.26	0.00
	0.9	400.2	9760	5606	54917	-34070	0.18	0.00
	Support	410.5	1199	404	34611	-39346	0.03	0.00

21.3.20.2 Superstructure Shear Capacity

As shown in Table 21.3-10, 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 21.3-10 Shear Demand vs. Capacity

	Location		Shear Demand	Shear Capacity = Strength Shear Demand	*D/C* Ratio
			V_{max}	$\phi V_n = V_{u,strength}$	*D/C*
Span 1	Support	1.5	803	2851	0.28
	0.1	12.6	772	2317	0.33
	0.2	25.2	540	1687	0.32
	0.3	37.8	309	1101	0.28
	0.4	50.4	227	681	0.33
	0.5	63.0	457	1207	0.38
	0.6	75.6	688	1782	0.39
	0.7	88.2	918	2341	0.39
	0.8	100.8	1149	2901	0.40
	0.9	113.4	1383	3596	0.38
	Support	123.0	1282	3966	0.32
Span 2	Support	129.0	1464	4378	0.33
	0.1	142.8	1498	3759	0.40
	0.2	159.6	1192	2961	0.40
	0.3	176.4	884	2160	0.41
	0.4	193.2	577	1399	0.41
	0.5	210.0	269	686	0.39
	0.6	226.8	546	1375	0.40
	0.7	243.6	853	2139	0.40
	0.8	260.4	1161	2942	0.39
	0.9	277.2	1496	3792	0.39
	Support	291.0	1440	4367	0.33
Span 3	Support	297.0	1234	3760	0.33
	0.1	305.8	1336	3388	0.39
	0.2	317.6	1116	2817	0.40
	0.3	329.4	900	2312	0.39
	0.4	341.2	684	1774	0.39
	0.5	353.0	468	1238	0.38
	0.6	364.8	252	738	0.34
	0.7	376.6	259	1000	0.26
	0.8	388.4	476	1548	0.31
	0.9	400.2	692	2138	0.32
	Support	410.5	743	2653	0.28

21.3.21 Step 19 - Design Joint Shear Reinforcement

Figure 21.3-17 shows the bent cap-to-column joint.

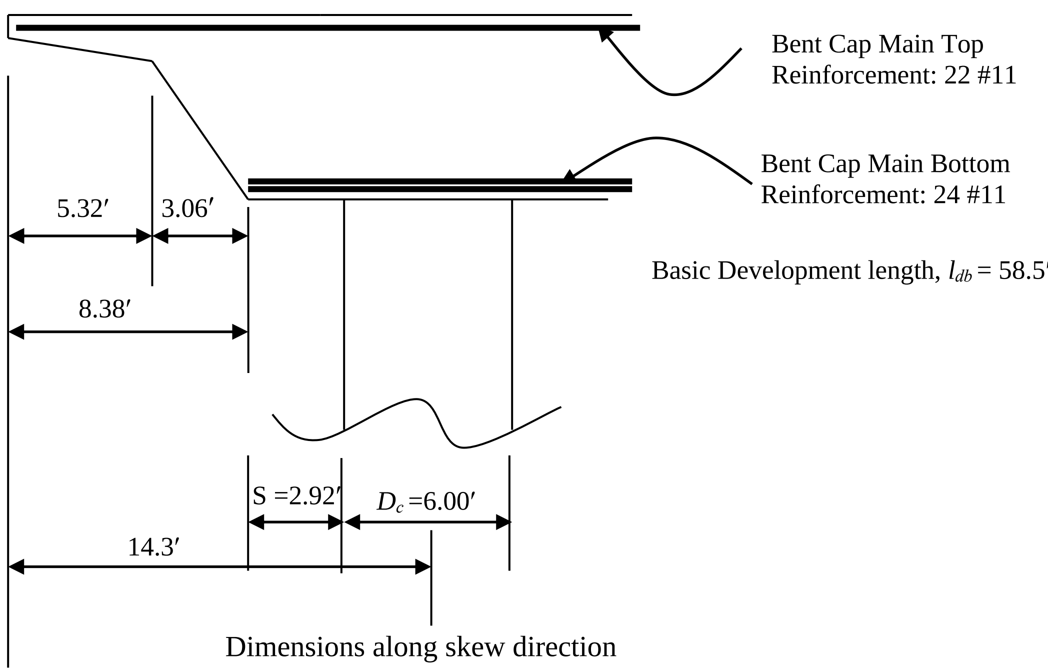


Figure 21.3-17 Bent Cap-to-Column Joint

Cap beam short stub length, S = 14.3 – 8.38 – 3 = 2.92 ft < D_c = 6 ft (*SDC* 7.4.3-1). Therefore the joint will be designed as a knee joint in the transverse direction and a T joint in the longitudinal direction.

21.3.21.1 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* 7.4.5.1-1)

(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: #8 hoops spaced at 5 in. c/c.

Column main reinforcement embedment length into the bent cap,

$$l_{ac,\,provided} = 66\,\text{in.}$$

From the *xSECTION* analysis of Bent 2 with overturning effects (see Appendix 21.3-7):
Column plastic moment, M_p = 14,964 kip-ft
Column axial force (including the effect of overturning), P_c = 2,474 kips
Cap Beam main reinforcement: top: #11 bars, total 2 and bottom: #11 bars, total 24.

<u>Calculate principal stresses,</u> p_t and p_c

Vertical Shear Stress, v_{jv}

$T_c \approx$ 1.2 (2862) kips = 3,434 kips
(Using *xSECTION* results of Appendix 21.3-7)

$$A_{jv} = l_{ac} B_{cap} = (66)(96) = 6{,}336\ \text{in.}^2 \qquad (SDC\ 7.4.4.1\text{-}4)$$

$$v_{jv} = \frac{T_c}{A_{jv}} = \frac{3{,}434}{6336} = 0.542\,\text{ksi} \qquad (SDC\ 7.4.4.1\text{-}3)$$

Normal Stress (Vertical), f_v

$$f_v = \frac{P_c}{A_{jh}} = \frac{P_c}{(D_c + D_s)B_{cap}} = \frac{2{,}474}{(6.00 + 6.75)(8.00)(144)} = 0.168\,\text{ksi}$$

(*SDC* 7.4.4.1-5)

Normal Stress (Horizontal)

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

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

$$= \frac{(0.00 + 0.168)}{2} - \sqrt{\left(\frac{0.00 - 0.168}{2}\right)^2 + 0.542^2} \text{ (+ for joint in tension)}$$

$$= -0.464 \text{ ksi}$$

(*SDC* 7.4.4.1-1)

$$p_c = \frac{(0.00+0.168)}{2} + \sqrt{\left(\frac{0.00-0.168}{2}\right)^2 + 0.542^2} \text{ (+ for joint in compression)}$$

$$= 0.632 \text{ ksi}$$

(*SDC* 7.4.4.1-2)

Check Joint Size Adequacy

Principal compression, p_c = 0.632 ksi ≤ [0.25 $f_c^{'}$ = 0.25 (4.0) = 1 ksi]

OK (*SDC* 7.4.2-1)

Principal tension, p_t = 0.464 ksi ≤ [12 $\sqrt{f_c^{'}} = 12\sqrt{4000}/1000$= 0.76 ksi]

OK (*SDC* 7.4.2-2)

Check the Need for Additional Joint Reinforcement

Since p_t = 0.464 ksi > [3.5 $\sqrt{f_c^{'}} = 3.5\sqrt{4000}/1000$= 0.221 ksi], additional joint reinforcement is required (see *SDC* Section 7.4.4.2).

Similar calculations can be performed for Bent 3.

(2) Opening Failure Mode - Bent 2 Knee Joint

From *wFRAME* push-over analysis results (see Appendix 21.3-6),
Column axial force (including the effect of overturning), P_c = 907 kips*
Column plastic moment, M_p = 12,636 kip-ft*

* These values were obtained from *xSECTION* analysis of Bent 2 with overturning effects (see Appendix 21.3-8)

T_c ≈ 1.2 (3,148) kips = 3,778 kips using *xSECTION* results.

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

$$v_{jv} = \frac{3{,}778}{6{,}336} = 0.596 \text{ ksi}$$

$$f_v = \frac{P_c}{A_{jh}} = \frac{P_c}{(D_c + D_s) \times B_{cap}} = \frac{907}{(6.00 + 6.75) \times 8.00 \times 144} = 0.062\,\text{ksi}$$

$f_h = 0$ (since $P_b = 0$)

$$p_t = \frac{(0.00 + 0.062)}{2} - \sqrt{\left(\frac{0.00 - 0.062}{2}\right)^2 + 0.596^2} = -0.566\,\text{ksi}$$

$$p_c = \frac{(0.00 + 0.062)}{2} + \sqrt{\left(\frac{0.00 - 0.062}{2}\right)^2 + 0.596^2} = 0.628\,\text{ksi}$$

Check Joint Size Adequacy

Principal compression, $p_c = 0.628$ ksi < [0.25 × 4.0 = 1 ksi] OK

Principal tension, [$p_t = 0.566$ ksi] < [$12\sqrt{4000}/1000 = 0.760$ ksi] OK

Check the Need for Additional Joint Reinforcement

Since p_t = 0.566 ksi > [$3.5\sqrt{4000}/1000 = 0.221$ ksi], additional joint reinforcement is required.

Based upon joint stress condition evaluation for both closing and opening modes of failure, the joint needs additional joint reinforcement. Refer to Figure 21.3-18 for regions of additional joint shear reinforcement.

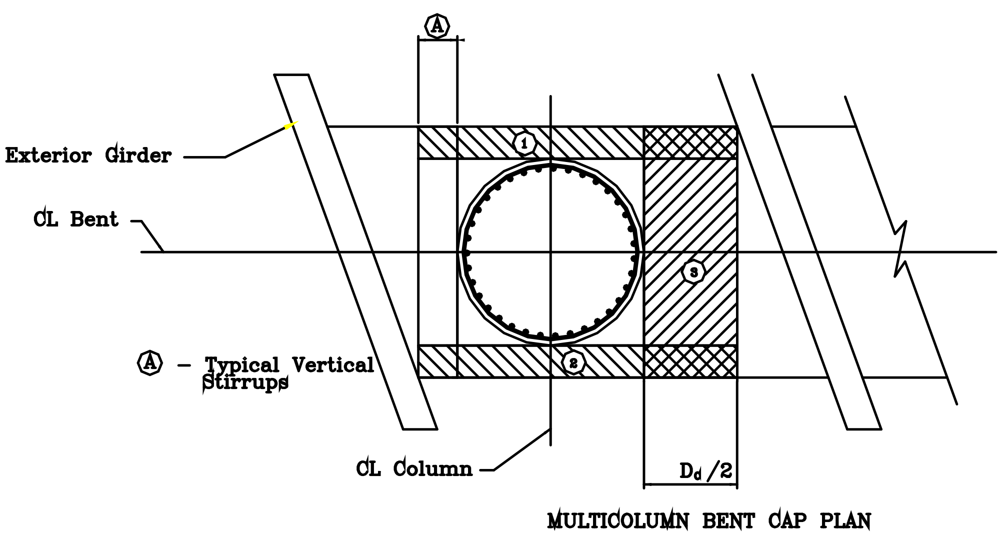


Figure 21.3-18 Regions of Additional Joint Shear Reinforcement

Joint Shear Requirement

- Bent Cap Top and Bottom Flexural Reinforcement, A_s^{U-Bar} (Refer to Figure 21.3-19)

  $A_s^{U-Bar}{}_{required}$ = 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 21.3-19 for the rebar layout.

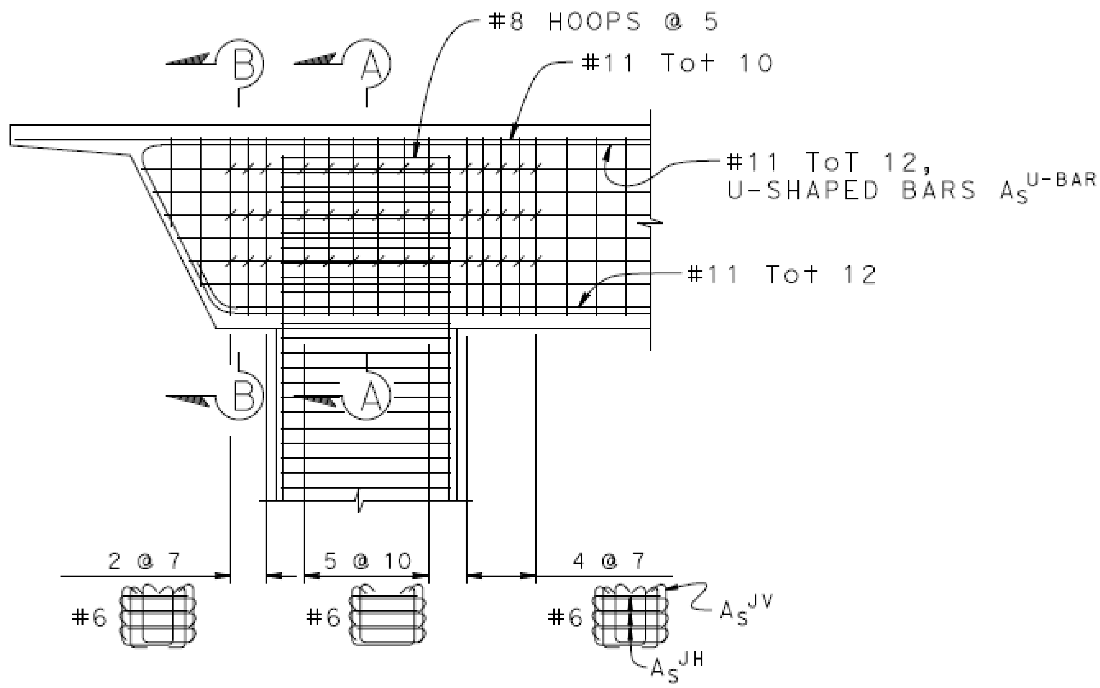


Figure 21.3-19 Location of Joint Shear Reinforcement (Elevation View)

- Vertical Stirrups in Joint Region

  $A_s^{jv}{}_{required}$ = 0.2 A_{st} = 0.20 (58.5) = 11.7 in.^{2} (*SDC* 7.4.5.1-4)

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

  $A_s^{jv}{}_{provided}$ = (6 legs)(5 sets)(0.44) = 13.2 in.^{2} > 11.7 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 21.3-19 and also in Figure 21.3-20.

- Horizontal Stirrups in Joint Region

$A_{s\ required}^{jh} = 0.1\ A_{st} = 0.1\ (58.5) = 5.85$ in.^{2} (*SDC* 7.4.5.1-5)

As shown in Figure 21.3-19, provide 3-legged #6 stirrups, total 14 sets

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

Placed within a distance D_c = 72 in. extending from either side of the column centerline as shown in Figure 21.3-19.

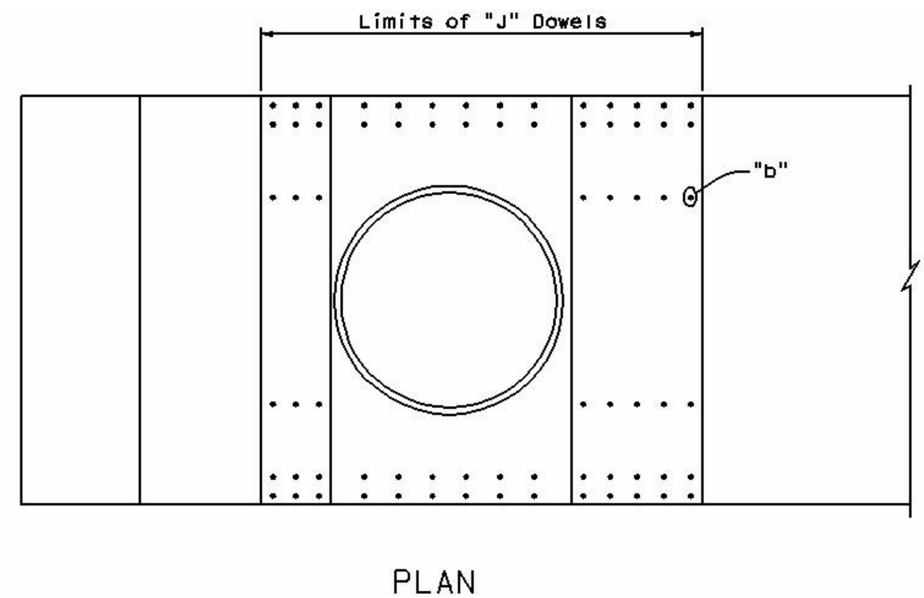


Figure 21.3-20 Location of Vertical Stirrups, A_s^{jv}

- Horizontal Side Reinforcement

$$A_s^{sf} \geq \begin{cases} 0.1 \times A_{cap}^{top} \\ \quad or \\ 0.1 \times A_{cap}^{bot} \end{cases}$$ (*SDC* 7.4.5.1-6)

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

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

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

As shown in Figures 21.3-21 and 21.3-22, provide #6, total 5 (“U” shaped), giving:

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

- Horizontal Cap End Ties

$$A_s^{jhc} = 0.33 A_s^{u-bar} = 0.33(19.3) = 6.37\,\text{in.}^2 \qquad (SDC\ 7.4.5.1\text{-}7)$$

Provide #8, total 10 ($A_{s,provided}^{jhc} = 10(0.79) = 7.9\,\text{in.}^2 > 6.37\,\text{in.}^2$) 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* guidelines, there is no need for J-Dowels for this bridge. Let us provide it anyway.

$$A_s^{j-bar} = 0.08\,A_{st} = 0.08\,(58.5) = 4.68\text{ in.}^2 \qquad (SDC\ 7.4.5.1\text{-}8)$$

Use 16, #5 J-Dowels.

$$A_{s\;\;provided}^{j-bar} = (16\text{ bars})(0.31) = 4.96\text{ in.}^2 > 4.68\text{ in.}^2$$

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 21.3-21 and 21.3-22.

- Check Transverse Reinforcement

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* 7.4.5.1-9)

Column transverse reinforcement that extends into the joint region consists of #8 hoops at 5 in. spacing.

$$\rho_{s,\ \ provided} = \frac{4A_b}{D's} = \frac{4(0.79)}{\left(72-2(2)-2\left[\frac{1.13}{2}\right]\right)(5)} = 0.0095 > \ 0.00936 \quad \text{OK}$$

- Check Anchorage for Main Column Reinforcement

$l_{ac,\ required} = 24d_{bl} = 24\ (1.69) = 40.6$ in. < [$l_{ac,\ provided} = 66$ in.]
OK (*SDC* 8.2.1-1)

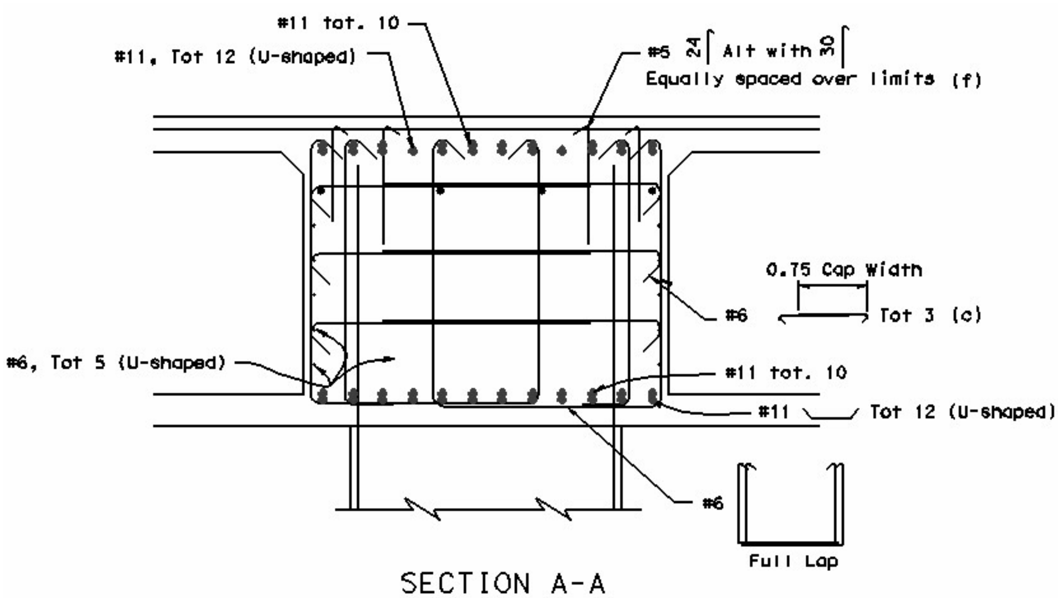


Figure 21.3-21 Joint Reinforcement Within the Column Region

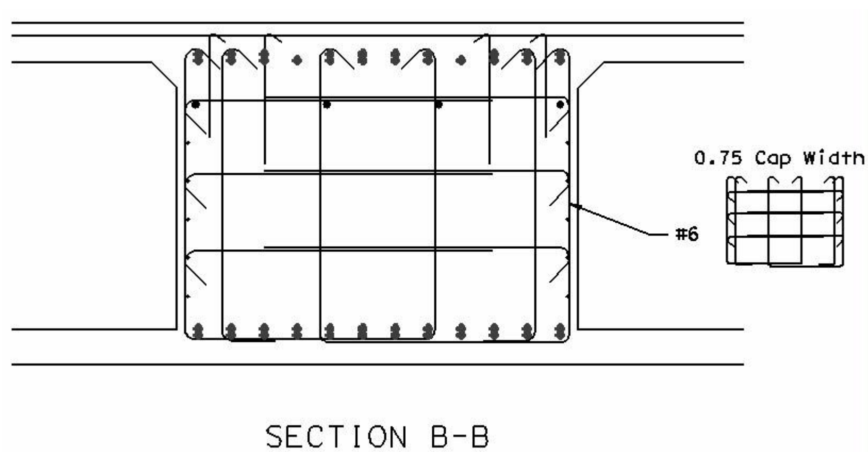


Figure 21.3-22 Joint Reinforcement Outside the Column Region

21.3.21.2 Longitudinal Direction (T-Joint)

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

Column plastic moment, M_p = 13,838 kip-ft*
Column axial force, P_c = 1,694 kips*
Cap beam main reinforcement: top: #11 bars, total 22 and bottom: #11 bars, total 24.
* These values were obtained from the *xSECTION* analysis of Bent 2 columns without overturning effects (see Appendix 21.3-1).

(1) Calculate Principal Stresses, p_t *and* p_c

$T_c \approx$ 1.2(2,948) kips = 3,538 kips using *xSECTION* results

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

Vertical Shear Stress

$$v_{jv} = \frac{T_c}{A_{jv}} = \frac{3{,}538}{6{,}336} = 0.558\,\text{ksi}$$

Normal Stress (Vertical)

$$f_v = \frac{P_c}{A_{jh}} = \frac{P_c}{(D_c + D_s)B_{cap}} = \frac{1{,}694}{(6.00 + 6.75)(8.00)(144)} = 0.115\,\text{ksi}$$

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

$$p_t = \frac{(0.00 + 0.115)}{2} - \sqrt{\left(\frac{0.00 - 0.115}{2}\right)^2 + 0.558^2} = -0.503\,\text{ksi} \quad \text{(i.e., tension)}$$

$$p_c = \frac{(0.00 + 0.115)}{2} + \sqrt{\left(\frac{0.00 - 0.115}{2}\right)^2 + 0.558^2} = 0.618\,\text{ksi} \quad \text{(i.e., compression)}$$

Check Joint Size Adequacy

Principal compression, p_c = 0.618 ksi ≤ [0.25 (4.0) = 1 ksi] OK

Principal tension, p_t = 0.503 ksi ≤ [$12\sqrt{4000}/1000$ = 0.760 ksi] OK

Check the Need for Additional Joint Reinforcement

p_t = 0.503 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 21.3-18.

- Vertical Stirrups in Joint Region – Regions 1 and 2 of Figure 21.3-18

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

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

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

  As shown in Figures 21.3-19 and 21.3-20, 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.

21.3.22 Step 20 - Determine Minimum Hinge Seat Width

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

21.3.23 Step 21 - Determine Minimum Abutment Seat Width

Minimum required abutment seat width, N_A = 30 in. (*SDC* Section 7.8.3)

$$N_A \geq \Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 4 \text{ (in.)} \quad (SDC\ 7.8.3\text{-}1)$$

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 21.3-11).

The maximum seismic 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.,

$\Delta_{eq,longitudinal}$ = 14.12 + 16.87 sin (20°) = 19.89 in.

$N_{A,\ required}$ (normal to centerline of bearing) = (19.89 + 2.6) cos (20°) + 4
= 25.13 in. < 30 in.

Provide abutment seat width N_A = 36 in. > 30 in. OK

21.3.24 Step 22 - Design Abutment Shear Key Reinforcement

Shear key force capacity, $F_{sk} = \alpha\,(0.75V_{piles} + V_{ww})$ (*SDC* 7.8.4-1)

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 (see MTD 5-1)
- 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 Method 1 of AASHTO Article 5.8.3.4, 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 = $[6.75 + \frac{1}{3}(7.25)](12)$ *in* = 110 in.

$V_{ww} = 0.0316\beta\sqrt{f'_c}\,b_v d_v = 0.0316(2)\sqrt{3.6}(110)(8.64)$ = 114 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:

$$A_{sk} = \frac{F_{sk}}{1.8 f_{ye}} = \frac{401}{1.8(68)} = 3.28 \text{ in.}^2$$ (*SDC* 7.8.4.1A-1)

Provide 8 #6 – bundle bars as shown in Figure *SDC* 7.8.4.1-1A
($A_{sk,\,provided}$ = 3.52 in.^{2} > 3.28 in.^{2}) OK

Hanger bars,

$A_{sh} = 2.0\,A^{Iso}_{sk(provided)}$ = 2 (3.52) = 7.04 in.^{2} (*SDC* 7.8.4.1B-1)

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

$$L_{\min,hooked} = 0.6(a+b) + l_{dh} \qquad (SDC\ 7.8.4.1B\text{-}3)$$

Assuming 5-inch thick bearing pads and 12 in. vertical height of expansion joint filler (see *SDC* Figure 7.8.4.1-1A),

a = (Bearing pad thichness + 6 in.) = 11 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 7.8.4.1-1A for definition of dimension "b")

$$l_{dh} = \frac{38d_b}{\sqrt{f_c'}} = \frac{38(1.41)}{\sqrt{3.6}} = 28.2\,\text{in.}$$

$$L_{\min,hooked} = 0.6(a+b) + l_{dh} = 0.6(11+3.4) + 28.2 = 37\,\text{in.}$$

Place vertical shear key bars A_{sk} 40 in. from the hooked ends of the hanger bars A_{sh}.

21.3.25 Step 23 - Check Requirements for No-splice Zone

For this bridge, only columns have been designated as "seismic critical" elements.

Maximum length of column rebar can be estimated as

L_{max} = 47.00 + 5.5 = 52.5 ft < 60.00 ft

Therefore, we will specify on the plans that no splices will be permitted for column main reinforcement. The superstructure rebars, however, will need to be spliced with Service Splice per *MTD* 20-9 (Caltrans 2001b).

APPENDIX 21.3-1 Output from *xSECTION*

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************************************************************
*                                                          *
*                       xSECTION                           *
*                                                          *
*               DUCTILITY and STRENGTH of                  *
*     Circular, Semi-Circular, full and partial Rings,     *
*  Rectangular, T-, I-, Hammer head, Octagonal, Polygons   *
*      or any combination of above shapes forming          *
*         Concrete Sections using Fiber Models             *
*                                                          *
* VER._2.40,_MAR-14-99                                     *
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* implied warranty of any kind with regard to this program.*
* In no event shall the author be held liable for          *
* incidental or consequential damages arising out of the   *
* use of this program.                                     *
*                                                          *
************************************************************
This output was generated by running:
xSECTION
VER._2.40,_MAR-14-99
LICENSE    (choices: LIMITED/UNLIMITED)
UNLIMITED
ENTITY     (choices: GOVERNMENT/CONSULTANT)
Government
NAME_OF_FIRM
Caltrans
BRIDGE_NAME
EXAMPLE
BRIDGE_NUMBER
99-9999
JOB_TITLE
PROTYPE BRIDGE - BRIDGE DESIGN ACADEMY



Concrete Type Information:

     ----------strains--------  --------strength--------
Type   e0    e2    ecc     eu     f0    f2    fcc    fu    E    W
 1 0.0020 0.0040 0.0055 0.0145  5.28  6.98  7.15  6.11  4313   148
 2 0.0020 0.0040 0.0020 0.0050  5.28  3.61  5.28  2.64  4313   148


Steel Type Information:

     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0150 0.0900 68.00 95.00 29000
 2 0.0023 0.0075 0.0600 68.00 95.00 29000


Steel Fiber Information:

Fiber          xc        yc       area
No.   type     in        in       in^2
  1    2    31.93     0.00    2.25
  2    2    31.00     7.64    2.25
  3    2    28.27    14.84    2.25
  4    2    23.90    21.17    2.25
  5    2    18.14    26.28    2.25
  6    2    11.32    29.86    2.25
```

```
  7    2     3.85   31.70    2.25
...
...

 25    2    28.27  -14.84    2.25
 26    2    31.00   -7.64    2.25


Force Equilibrium Condition of the x-section:

     Max.             Max.
     Conc.   Neutral Steel          Steel
     Strain   Axis   Strain Conc.    force       P/S     Net  Curvature Moment
step epscmax  in.    Tens.  Comp.  Comp. Tens.   force   force  rad/in  (K-ft)
 0   0.00000   0.00  0.0000     0     0     0     0      0.00 0.000000      0
 1   0.00029 -12.30 -0.0001  1570   174   -49     0      1.52 0.000006   2588
 2   0.00032  -9.09 -0.0002  1585   182   -73     0      0.95 0.000007   2843
...
...
25   0.00322  16.99 -0.0083  3210   889 -2406     0     -0.96 0.000170  12508
26   0.00356  17.39 -0.0094  3249   929 -2483     0      0.66 0.000192  12718
27   0.00394  17.67 -0.0106  3309   952 -2568     0     -1.69 0.000215  12926
28   0.00435  17.91 -0.0119  3361   978 -2646     0     -1.26 0.000241  13129
29   0.00481  18.07 -0.0134  3388  1008 -2703     0     -0.57 0.000269  13267
30   0.00532  18.11 -0.0148  3413  1037 -2756     0      0.59 0.000298  13362
31   0.00588  18.15 -0.0164  3461  1048 -2816     0     -0.56 0.000330  13495
32   0.00650  18.21 -0.0183  3515  1060 -2881     0     -0.42 0.000366  13660
33   0.00718  18.27 -0.0203  3570  1072 -2948     0     -0.93 0.000406  13834
34   0.00794  18.30 -0.0225  3630  1087 -3021     0      1.38 0.000449  14017
35   0.00878  18.33 -0.0249  3686  1103 -3096     0     -1.20 0.000497  14194
36   0.00971  18.34 -0.0275  3743  1122 -3171     0     -0.61 0.000550  14368
37   0.01073  18.34 -0.0304  3792  1148 -3246     0      0.07 0.000608  14536
38   0.01186  18.34 -0.0336  3834  1181 -3321     0     -0.67 0.000672  14695
39   0.01312  18.38 -0.0373  3847  1217 -3371     0     -0.48 0.000745  14841
40   0.01450  18.41 -0.0414  3857  1256 -3420     0     -1.66 0.000825  14976


First Yield of Rebar Information (not Idealized):

     Rebar Number 20
     Coordinates X and Y (global in.)  -3.85, -31.70
     Yield strain =  0.00230
     Curvature (rad/in)=  0.000054
     Moment (ft-k) =    9537

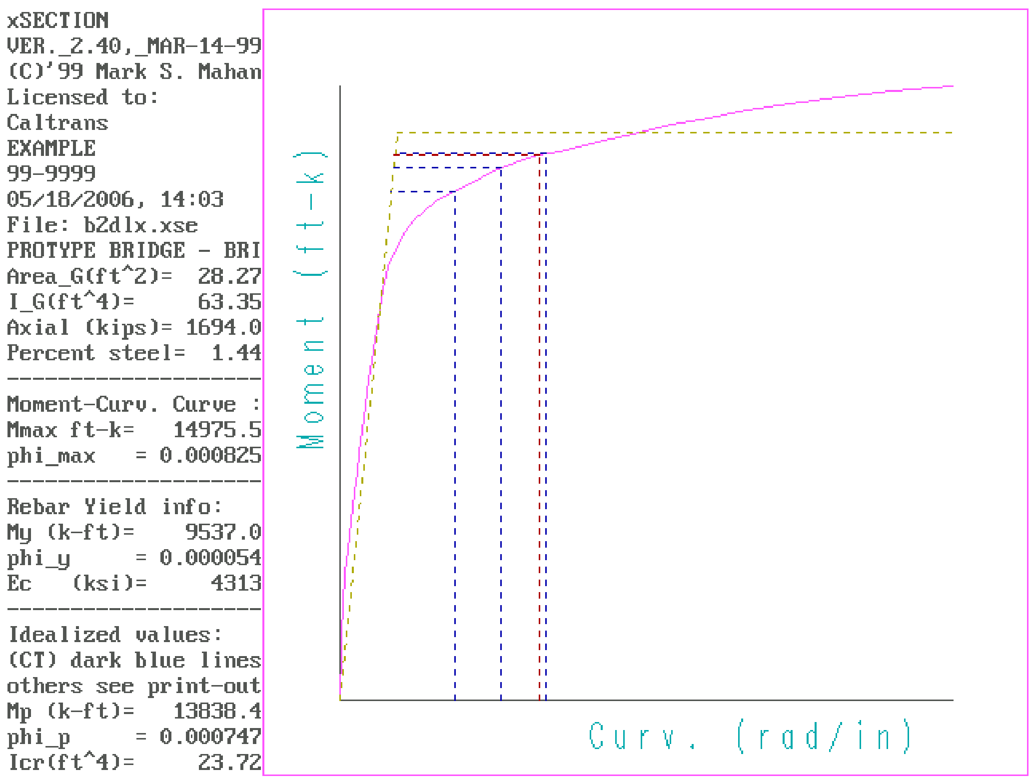
Cross Section Information:

     Axial Load on Section (kips) =  1694
     Percentage of Main steel in Cross Section =   1.44
     Concrete modulus used in Idealization (ksi) =  4313
     Cracked Moment of Inertia (ft^4) =    23.717

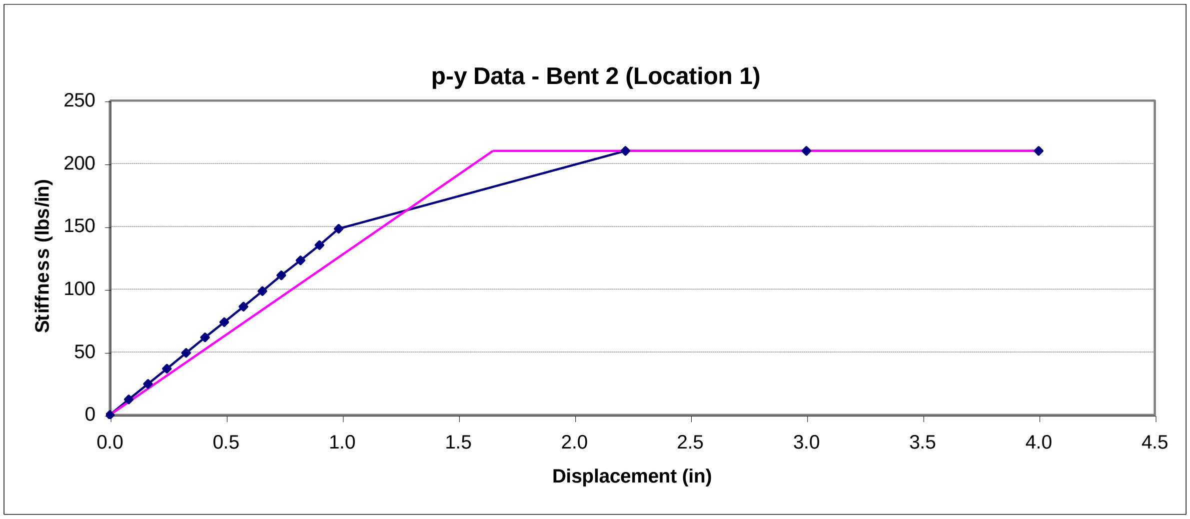

Idealization of Moment-Curvature Curve by Various Methods:

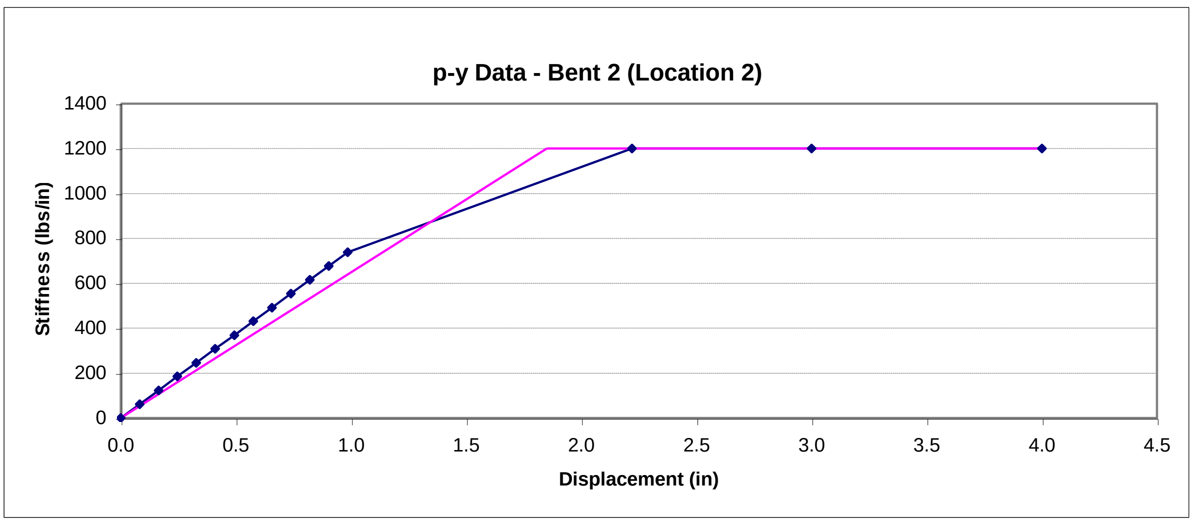
                  Points on Curve        Idealized Values
                  ===============   =============================
Method     Conc.                    Yield          symbol  Plastic
 ID      | Strain  Curv.    Moment | Curv.  Moment   for   Curv.
         | in/in   rad/in   (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000155   12388 0.000070   12388   Mn   0.000755
Strain @ 0.004   0.000219   12957 0.000073   12957   Mn   0.000752
Strain @ 0.005   0.000279   13302 0.000075   13302   Mn   0.000750
CALTRANS 0.00720 0.000407   13838 0.000078   13838   Mp   0.000747
UCSD@5phy0.00483 0.000270   13271 0.000075   13271   Mn   0.000750
```

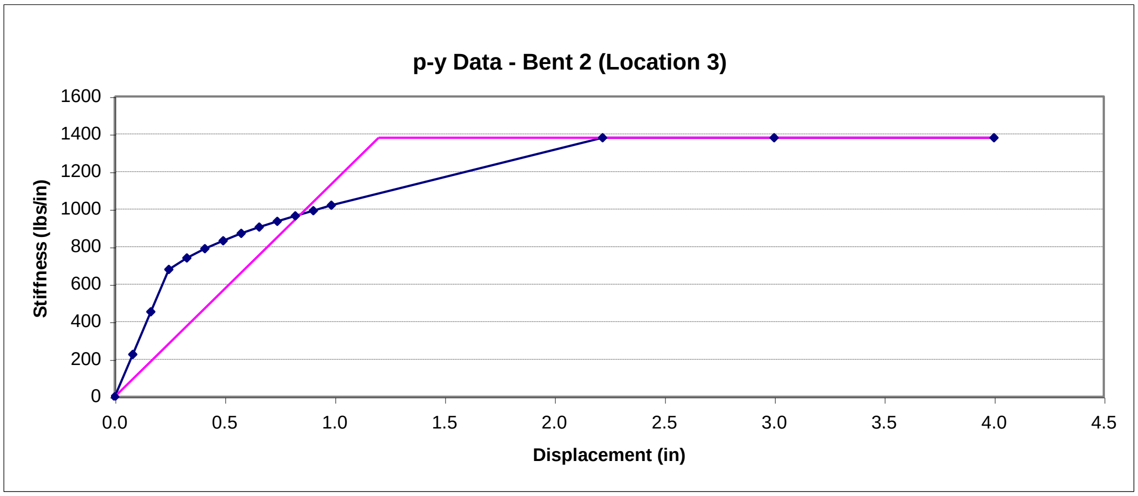
APPENDIX 21.3-2 Moment – Curvature Relationship

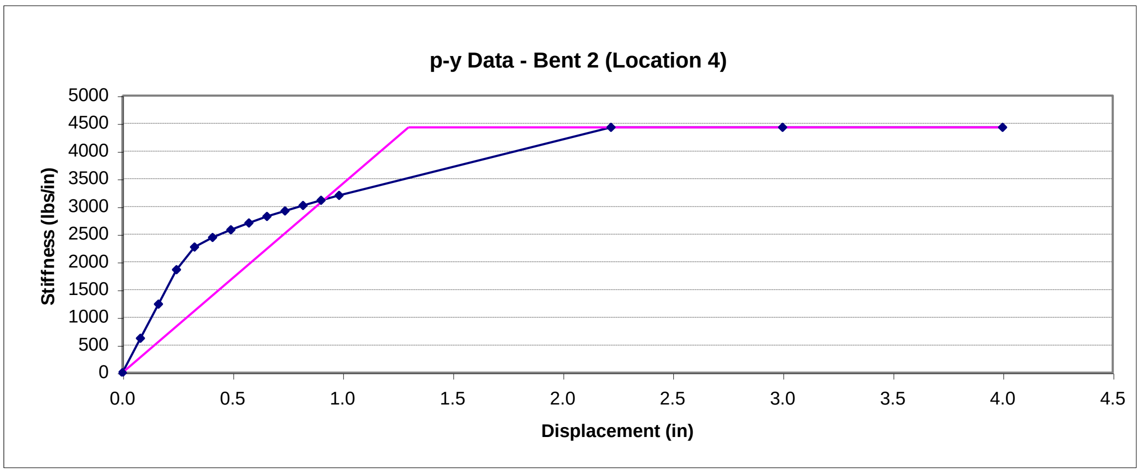


APPENDIX 21.3-3 Soil Spring Data






p-y Data - Bent 2 (Location 3)
Stiffness (lbs/in)
0
200
400
600
800
1000
1200
1400
1600
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
Displacement (in)


p-y Data - Bent 2 (Location 4)
Stiffness (lbs/in)
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
Displacement (in)

APPENDIX 21.3-4 Bent Cap – Positive Bending Section Capacities – Select Output

```
05/16/2006, 10:17
***********************************************************
*                      xSECTION                           *
*                                                         *
*              DUCTILITY and STRENGTH of                  *
*     Circular, Semi-Circular, full and partial Rings,    *
*  Rectangular, T-, I-, Hammer head, Octagonal, Polygons  *
*      or any combination of above shapes forming         *
*         Concrete Sections using Fiber Models            *
*                                                         *
* VER._2.40,_MAR-14-99                                    *
*                                                         *
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* use of this program.                                    *
*                                                         *
***********************************************************
This output was generated by running:
xSECTION
VER._2.40,_MAR-14-99
........................................
........................................
Concrete Type Information:
     ----------strains--------  --------strength--------
Type   e0     e2     ecc     eu     f0    f2    fcc    fu    E     W
 1 0.0020 0.0040 0.0027 0.0115  5.00  5.01  5.35  2.63  4200   148
 2 0.0020 0.0040 0.0020 0.0050  5.00  3.52  5.00  2.50  4200   148

Steel Type Information:
     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0150 0.0900 68.00 95.00 29000
 2 0.0023 0.0075 0.0600 68.00 95.00 29000
........................................

First Yield of Rebar Information (not Idealized):

     Rebar Number 1
     Coordinates X and Y (global in.) -44.80, -35.49
     Yield strain =  0.00230
     Curvature (rad/in)=  0.000037
     Moment (ft-k) =   14873

Cross Section Information:
     Axial Load on Section (kips) =     1
     Percentage of Main steel in Cross Section =   0.80
     Concrete modulus used in Idealization (ksi) =  4200
     Cracked Moment of Inertia (ft^4) =    55.568

Idealization of Moment-Curvature Curve by Various Methods:
                  Points on Curve        Idealized Values
                  ===============   =============================
Method     Conc.                    Yield          symbol  Plastic
 ID       | Strain  Curv.   Moment | Curv.  Moment   for   Curv.
          | in/in   rad/in  (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000520   21189 0.000053   21189   Mn  0.000665
Strain @ 0.004   0.000684   21635 0.000054   21635   Mn  0.000664
Strain @ 0.005   0.000000       0 0.000000       0   Mn  0.000718
CALTRANS 0.00187 0.000306   19484 0.000048   19484   Mp  0.000669
UCSD@5phy0.00126 0.000184   17426 0.000043   17426   Mn  0.000674
```

APPENDIX 21.3-5 Bent Cap – Negative Bending Section Capacities – Select Output

```
05/15/2006, 08:26
************************************************************
*                      xSECTION                            *
*                                                          *
*              DUCTILITY and STRENGTH of                   *
*     Circular, Semi-Circular, full and partial Rings,     *
*  Rectangular, T-, I-, Hammer head, Octagonal, Polygons   *
*      or any combination of above shapes forming          *
*         Concrete Sections using Fiber Models             *
*                                                          *
* VER._2.40,_MAR-14-99                                     *
*                                                          *
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* In no event shall the author be held liable for          *
* incidental or consequential damages arising out of the   *
* use of this program.                                     *
*                                                          *
************************************************************
This output was generated by running:
xSECTION
VER._2.40,_MAR-14-99
…………………………………………………………………………………………………………………………………….
……………………………………………………………………………………………………………………………………..

Concrete Type Information:
     ----------strains--------  --------strength--------
Type   e0     e2     ecc     eu      f0    f2    fcc    fu    E    W
 1 0.0020 0.0040 0.0027 0.0115  5.00  5.01  5.35  2.63  4200   148
 2 0.0020 0.0040 0.0020 0.0050  5.00  3.52  5.00  2.50  4200   148

Steel Type Information:
     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0150 0.0900 68.00 95.00 29000
 2 0.0023 0.0075 0.0600 68.00 95.00 29000

First Yield of Rebar Information (not Idealized):
     Rebar Number 25
            Coordinates X and Y (global in.)  44.80, -34.49
     Yield strain =  0.00230
     Curvature (rad/in)=  0.000037
     Moment (ft-k) =   13030

Cross Section Information:
     Axial Load on Section (kips) =     1
     Percentage of Main steel in Cross Section =   0.80
     Concrete modulus used in Idealization (ksi) =  4200
     Cracked Moment of Inertia (ft^4) =    48.938

Idealization of Moment-Curvature Curve by Various Methods:
                  Points on Curve        Idealized Values
                  ===============   =============================
Method     Conc.                    Yield         symbol  Plastic
 ID      | Strain  Curv.   Moment | Curv.  Moment   for   Curv.
         | in/in   rad/in  (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000593   19436 0.000055   19436   Mn  0.000563
Strain @ 0.004   0.000000       0 0.000000       0   Mn  0.000618
Strain @ 0.005   0.000000       0 0.000000       0   Mn  0.000618
CALTRANS 0.00159 0.000282   17307 0.000049   17307   Mp  0.000569
UCSD@5phy0.00117 0.000183   15735 0.000044   15735   Mn  0.000573
```

APPENDIX 21.3-6 *wFRAME* Output File

```
05/15/2006, 07:47
Design Academy Example No: 1 (Bent 2)

************************************************************
*                                                          *
*                       wFRAME                             *
*                                                          *
*          PUSH ANALYSIS of BRIDGE BENTS and FRAMES.       *
*                                                          *
*       Indicates formation of successive plastic hinges.  *
*                                                          *
* VER._1.12,_JAN-14-95                                     *
*                                                          *
* Copyright (C) 1994 By Mark Seyed.                        *
*                                                          *
* This program should not be distributed under any         *
* condition. This release is for demo ONLY (beta testing   *
* is not complete). The author makes no expressed or       *
* implied warranty of any kind with regard to this program.*
* In no event shall the author be held liable for          *
* incidental or consequential damages arising out of the   *
* use of this program.                                     *
*                                                          *
************************************************************
Node Point Information:

Fixity condition definitions:
    s=spring and value
    r=complete release
    f=complete fixity with imposed displacement

node    name    coordinates    -----------fixity --------
  #               X       Y     X-dir.     Y-dir.   Rotation
    1 S01.00     0.00     0.00   r          r          r
    2 S01.01     4.72     0.00   r          r          r
    3 S01.02     7.72     0.00   r          r          r
    4 C01.01     7.72    -3.38   r          r          r
    5 C01.02     7.72   -15.31   r          r          r
    6 C01.03     7.72   -27.24   r          r          r
    7 C01.04     7.72   -39.17   r          r          r
    8 P01.01     7.72   -41.22   s 1.4e+002  r          r
    9 P01.02     7.72   -43.27   s 4.1e+002  r          r
   10 P01.03     7.72   -45.32   s 6.7e+002  r          r
   11 P01.04     7.72   -47.37   f 0.0000   f 0.0000   r
   12 S02.01    10.72     0.00   r          r          r
   13 S02.02    17.72     0.00   r          r          r
   14 S02.03    24.72     0.00   r          r          r
   15 S02.04    31.72     0.00   r          r          r
   16 S02.05    38.72     0.00   r          r          r
   17 S02.06    41.72     0.00   r          r          r
   18 C02.01    41.72    -3.38   r          r          r
   19 C02.02    41.72   -15.31   r          r          r
   20 C02.03    41.72   -27.24   r          r          r
   21 C02.04    41.72   -39.17   r          r          r
   22 P02.01    41.72   -41.22   s 1.4e+002  r          r
   23 P02.02    41.72   -43.27   s 4.1e+002  r          r
   24 P02.03    41.72   -45.32   s 6.7e+002  r          r
   25 P02.04    41.72   -47.37   f 0.0000   f 0.0000   r
   26 S03.01    44.72     0.00   r          r          r
   27 S03.02    49.44     0.00   r          r          r


Spring Information at node points:

k's = k/ft or ft-k/rad.;  d's = ft or rad.
node   spring      k1      d1      k2      d2
```

```
    #    name
    8  P01X01       136.37  0.149         0.00  1.000         0.00 1000.000
    9  P01X02       414.83  0.105         0.00  1.000         0.00 1000.000
   10  P01X03       665.70  0.106         0.00  1.000         0.00 1000.000
   22  P02X01       136.37  0.149         0.00  1.000         0.00 1000.000
   23  P02X02       414.83  0.105         0.00  1.000         0.00 1000.000
   24  P02X03       665.70  0.106         0.00  1.000         0.00 1000.000

Structural Setup:
Spans= 3, Columns= 2, Piles= 2, Link Beams= 0

Element Information:
   element     nodes      depth
 # name fix    i  j     L   d   area    Ei     Ef     Icr     q     Mpp    Mpn  tol status
  1 S01-01 rn   1   2   4.72 6.8 62.6 629528  62953  52.25 -68.40  27676  27676 0.02 e
  2 S01-02 rn   2   3   3.00 6.8 62.6 629528  62953  52.25 -68.40  27676  27676 0.02 e
  3 C01-01 rn   3   4   3.38 6.0 28.3 629528  62953  47.44   0.00  27676  27676 0.02 e
  4 C01-02 rn   4   5  11.93 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
  5 C01-03 rn   5   6  11.93 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
  6 C01-04 rn   6   7  11.93 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
  7 P01-01 rn   7   8   2.05 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
  8 P01-02 rn   8   9   2.05 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
  9 P01-03 rn   9  10   2.05 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
 10 P01-04 rn  10  11   2.05 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
 11 S02-01 rn   3  12   3.00 6.8 62.6 629528  62953  52.25 -68.40  27676  27676 0.02 e
 12 S02-02 rn  12  13   7.00 6.8 62.6 629528  62953  52.25 -68.40  27676  27676 0.02 e
 13 S02-03 rn  13  14   7.00 6.8 62.6 629528  62953  52.25 -68.40  27676  27676 0.02 e
 14 S02-04 rn  14  15   7.00 6.8 62.6 629528  62953  52.25 -68.40  27676  27676 0.02 e
 15 S02-05 rn  15  16   7.00 6.8 62.6 629528  62953  52.25 -68.40  27676  27676 0.02 e
 16 S02-06 rn  16  17   3.00 6.8 62.6 629528  62953  52.25 -68.40  27676  27676 0.02 e
 17 C02-01 rn  17  18   3.38 6.0 28.3 629528  62953  47.44   0.00  27676  27676 0.02 e
 18 C02-02 rn  18  19  11.93 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
 19 C02-03 rn  19  20  11.93 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
 20 C02-04 rn  20  21  11.93 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
 21 P02-01 rn  21  22   2.05 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
 22 P02-02 rn  22  23   2.05 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
 23 P02-03 rn  23  24   2.05 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
 24 P02-04 rn  24  25   2.05 6.0 28.3 629528  62953  23.72   0.00  13838  13838 0.02 e
 25 S03-01 rn  17  26   3.00 6.8 62.6 629528  62953  52.25 -68.40  27676  27676 0.02 e
 26 S03-02 rn  26  27   4.72 6.8 62.6 629528  62953  52.25 -68.40  27676  27676 0.02 e
bandwidth of the problem = 10
Number of rows and columns in strage = 81 x 30


Cumulative Results of analysis at end of stage   0

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL= 3381.7)  /    (in)

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00  0.00001  0.00633 -0.00136
    2 S01.01  0.00001 -0.00014 -0.00140
    3 S01.02  0.00001 -0.00450 -0.00152
    4 C01.01 -0.00484 -0.00418 -0.00135
    5 C01.02 -0.01446 -0.00304 -0.00032
    6 C01.03 -0.01376 -0.00191  0.00038
    7 C01.04 -0.00662 -0.00078  0.00076
    8 P01.01 -0.00503 -0.00058  0.00079
    9 P01.02 -0.00338 -0.00039  0.00081
   10 P01.03 -0.00170 -0.00019  0.00083
   11 P01.04  0.00000  0.00000  0.00083
   12 S02.01  0.00001 -0.00941 -0.00170
   13 S02.02  0.00000 -0.02023 -0.00121
```

```
   14 S02.03 -0.00001 -0.02467  0.00000
   15 S02.04 -0.00001 -0.02023  0.00121
   16 S02.05 -0.00002 -0.00941  0.00170
   17 S02.06 -0.00002 -0.00450  0.00152
   18 C02.01  0.00483 -0.00417  0.00135
   19 C02.02  0.01445 -0.00304  0.00032
   20 C02.03  0.01375 -0.00191 -0.00038
   21 C02.04  0.00662 -0.00078 -0.00076
   22 P02.01  0.00503 -0.00058 -0.00079
   23 P02.02  0.00338 -0.00039 -0.00081
   24 P02.03  0.00170 -0.00019 -0.00083
   25 P02.04  0.00000  0.00000 -0.00083
   26 S03.01 -0.00002 -0.00014  0.00140
   27 S03.02 -0.00002  0.00633  0.00136

element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1  0.00001  0.00633 -0.00136      0.00      0.00      0.00
                2  0.00001 -0.00014 -0.00140      0.00    322.85   -761.94
  2 S01-02 rn   2  0.00001 -0.00014 -0.00140      0.00   -322.85    761.93
                3  0.00001 -0.00450 -0.00152      0.00    528.05  -2038.28
  3 C01-01 rn   3  0.00450  0.00001 -0.00152   1690.85    -34.15  -1605.41
                4  0.00418 -0.00484 -0.00135  -1690.85     34.15   1489.98
  4 C01-02 rn   4  0.00418 -0.00484 -0.00135   1690.85    -34.15  -1489.98
                5  0.00304 -0.01446 -0.00032  -1690.85     34.15   1082.57
  5 C01-03 rn   5  0.00304 -0.01446 -0.00032   1690.85    -34.15  -1082.57
                6  0.00191 -0.01376  0.00038  -1690.85     34.15    675.15
  6 C01-04 rn   6  0.00191 -0.01376  0.00038   1690.85    -34.15   -675.15
                7  0.00078 -0.00662  0.00076  -1690.85     34.15    267.73
  7 P01-01 rn   7  0.00078 -0.00662  0.00076   1690.85    -34.14   -267.71
                8  0.00058 -0.00503  0.00079  -1690.85     34.14    197.70
  8 P01-02 rn   8  0.00058 -0.00503  0.00079   1690.85    -33.46   -197.70
                9  0.00039 -0.00338  0.00081  -1690.85     33.46    129.10
  9 P01-03 rn   9  0.00039 -0.00338  0.00081   1690.85    -32.05   -129.10
               10  0.00019 -0.00170  0.00083  -1690.85     32.05     63.39
 10 P01-04 rn  10  0.00019 -0.00170  0.00083   1690.85    -30.92    -63.40
               11  0.00000  0.00000  0.00083  -1690.85     30.92      0.00
 11 S02-01 rn   3  0.00001 -0.00450 -0.00152     34.15   1162.80   3643.68
               12  0.00001 -0.00941 -0.00170    -34.15   -957.60   -463.07
 12 S02-02 rn  12  0.00001 -0.00941 -0.00170     34.15    957.61    463.06
               13  0.00000 -0.02023 -0.00121    -34.15   -478.81   4564.38
 13 S02-03 rn  13  0.00000 -0.02023 -0.00121     34.15    478.81  -4564.38
               14 -0.00001 -0.02467  0.00000    -34.15     -0.01   6240.23
 14 S02-04 rn  14 -0.00001 -0.02467  0.00000     34.15      0.01  -6240.23
               15 -0.00001 -0.02023  0.00121    -34.15    478.79   4564.47
 15 S02-05 rn  15 -0.00001 -0.02023  0.00121     34.15   -478.80  -4564.46
               16 -0.00002 -0.00941  0.00170    -34.15    957.60   -462.91
 16 S02-06 rn  16 -0.00002 -0.00941  0.00170     34.15   -957.59    462.89
               17 -0.00002 -0.00450  0.00152    -34.15   1162.79  -3643.48
 17 C02-01 rn  17  0.00450 -0.00002  0.00152   1690.83     34.15   1605.20
               18  0.00417  0.00483  0.00135  -1690.83    -34.15  -1489.77
 18 C02-02 rn  18  0.00417  0.00483  0.00135   1690.83     34.15   1489.77
               19  0.00304  0.01445  0.00032  -1690.83    -34.15  -1082.42
 19 C02-03 rn  19  0.00304  0.01445  0.00032   1690.83     34.15   1082.42
               20  0.00191  0.01375 -0.00038  -1690.83    -34.15   -675.06
 20 C02-04 rn  20  0.00191  0.01375 -0.00038   1690.83     34.15    675.06
               21  0.00078  0.00662 -0.00076  -1690.83    -34.15   -267.70
 21 P02-01 rn  21  0.00078  0.00662 -0.00076   1690.83     34.14    267.71
               22  0.00058  0.00503 -0.00079  -1690.83    -34.14   -197.71
 22 P02-02 rn  22  0.00058  0.00503 -0.00079   1690.83     33.46    197.71
               23  0.00039  0.00338 -0.00081  -1690.83    -33.46   -129.11
 23 P02-03 rn  23  0.00039  0.00338 -0.00081   1690.83     32.06    129.12
               24  0.00019  0.00170 -0.00083  -1690.83    -32.06    -63.39
 24 P02-04 rn  24  0.00019  0.00170 -0.00083   1690.83     30.93     63.40
               25  0.00000  0.00000 -0.00083  -1690.83    -30.93      0.00
 25 S03-01 rn  17 -0.00002 -0.00450  0.00152      0.00    528.05   2038.28
```

```
             26 -0.00002 -0.00014  0.00140      0.00   -322.85   -761.92
 26 S03-02 rn  26 -0.00002 -0.00014  0.00140      0.00    322.85    761.93
             27 -0.00002  0.00633  0.00136      0.00      0.00      0.00


Cumulative Results of analysis at end of stage   1

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL= 3381.7)  /     (in)
C02-02     1     rs       0.1712         8.4898

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00  0.70748  0.02708 -0.00378
    2 S01.01  0.70748  0.00919 -0.00382
    3 S01.02  0.70747 -0.00241 -0.00394
    4 C01.01  0.69197 -0.00224 -0.00522
    5 C01.02  0.58279 -0.00163 -0.01268
    6 C01.03  0.39898 -0.00103 -0.01773
    7 C01.04  0.16941 -0.00042 -0.02035
    8 P01.01  0.12746 -0.00031 -0.02056
    9 P01.02  0.08515 -0.00021 -0.02070
   10 P01.03  0.04263 -0.00010 -0.02078
   11 P01.04  0.00000  0.00000 -0.02080
   12 S02.01  0.70749 -0.01286 -0.00301
   13 S02.02  0.70750 -0.02604 -0.00077
   14 S02.03  0.70750 -0.02467  0.00103
   15 S02.04  0.70749 -0.01442  0.00165
   16 S02.05  0.70746 -0.00595  0.00039
   17 S02.06  0.70744 -0.00658 -0.00090
   18 C02.01  0.70163 -0.00611 -0.00252
   19 C02.02  0.61168 -0.00445 -0.01204
   20 C02.03  0.42648 -0.00280 -0.01849
   21 C02.04  0.18265 -0.00114 -0.02187
   22 P02.01  0.13751 -0.00085 -0.02214
   23 P02.02  0.09192 -0.00057 -0.02233
   24 P02.03  0.04603 -0.00028 -0.02243
   25 P02.04  0.00000  0.00000 -0.02246
   26 S03.01  0.70745 -0.00948 -0.00102
   27 S03.02  0.70745 -0.01442 -0.00106

element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1  0.70748  0.02708 -0.00378     27.69     -0.01     -0.01
                2  0.70748  0.00919 -0.00382    -27.69    322.85   -761.97
  2 S01-02 rn   2  0.70748  0.00919 -0.00382     71.78   -322.85    761.95
                3  0.70747 -0.00241 -0.00394    -71.78    528.05  -2038.29
  3 C01-01 rn   3  0.00241  0.70747 -0.00394    907.25    253.50  11715.99
                4  0.00224  0.69197 -0.00522   -907.25   -253.50 -10859.41
  4 C01-02 rn   4  0.00224  0.69197 -0.00522    907.18    253.90  10859.10
                5  0.00163  0.58279 -0.01268   -907.18   -253.90  -7830.08
  5 C01-03 rn   5  0.00163  0.58279 -0.01268    907.18    253.90   7830.12
                6  0.00103  0.39898 -0.01773   -907.18   -253.90  -4801.02
  6 C01-04 rn   6  0.00103  0.39898 -0.01773    907.18    253.90   4801.03
                7  0.00042  0.16941 -0.02035   -907.18   -253.90  -1771.96
  7 P01-01 rn   7  0.00042  0.16941 -0.02035    907.18    253.53   1771.40
                8  0.00031  0.12746 -0.02056   -907.18   -253.53  -1250.91
  8 P01-02 rn   8  0.00031  0.12746 -0.02056    907.18    236.71   1251.16
                9  0.00021  0.08515 -0.02070   -907.18   -236.71   -765.72
  9 P01-03 rn   9  0.00021  0.08515 -0.02070    907.18    201.31    766.32
               10  0.00010  0.04263 -0.02078   -907.18   -201.31   -353.62
 10 P01-04 rn  10  0.00010  0.04263 -0.02078    907.18    172.67    354.11
               11  0.00000  0.00000 -0.02080   -907.18   -172.67      0.01
 11 S02-01 rn   3  0.70747 -0.00241 -0.00394   -147.11    379.20  -9677.96
               12  0.70749 -0.01286 -0.00301    147.11   -174.00  10507.76
 12 S02-02 rn  12  0.70749 -0.01286 -0.00301    -88.03    173.93 -10507.76
```

```
                13  0.70750 -0.02604 -0.00077     88.03    304.87  10049.47
 13 S02-03 rn   13  0.70750 -0.02604 -0.00077     -5.78   -304.87 -10049.47
                14  0.70750 -0.02467  0.00103      5.78    783.67   6239.57
 14 S02-04 rn   14  0.70750 -0.02467  0.00103     76.09   -783.67  -6239.57
                15  0.70749 -0.01442  0.00165    -76.09   1262.47   -921.93
 15 S02-05 rn   15  0.70749 -0.01442  0.00165    158.08  -1262.47    921.94
                16  0.70746 -0.00595  0.00039   -158.08   1741.27 -11435.05
 16 S02-06 rn   16  0.70746 -0.00595  0.00039    216.23  -1741.27  11435.04
                17  0.70744 -0.00658 -0.00090   -216.23   1946.47 -16966.67
 17 C02-01 rn   17  0.00658  0.70744 -0.00090   2474.51    322.57  14926.95
                18  0.00611  0.70163 -0.00252  -2474.51   -322.57 -13838.61
 18 C02-02 rs   18  0.00611  0.70163 -0.00252   2474.46    322.17  13838.00
                19  0.00445  0.61168 -0.01204  -2474.46   -322.17  -9994.56
 19 C02-03 rn   19  0.00445  0.61168 -0.01204   2474.46    322.18   9994.59
                20  0.00280  0.42648 -0.01849  -2474.46   -322.18  -6150.91
 20 C02-04 rn   20  0.00280  0.42648 -0.01849   2474.46    322.18   6150.92
                21  0.00114  0.18265 -0.02187  -2474.46   -322.18  -2307.32
 21 P02-01 rn   21  0.00114  0.18265 -0.02187   2474.46    322.17   2307.40
                22  0.00085  0.13751 -0.02214  -2474.46   -322.17  -1646.88
 22 P02-02 rn   22  0.00085  0.13751 -0.02214   2474.46    303.41   1647.21
                23  0.00057  0.09192 -0.02233  -2474.46   -303.41  -1024.71
 23 P02-03 rn   23  0.00057  0.09192 -0.02233   2474.46    265.35   1024.96
                24  0.00028  0.04603 -0.02243  -2474.46   -265.35   -481.13
 24 P02-04 rn   24  0.00028  0.04603 -0.02243   2474.46    234.74    481.23
                25  0.00000  0.00000 -0.02246  -2474.46   -234.74      0.07
 25 S03-01 rn   17  0.70744 -0.00658 -0.00090    -72.95    528.06   2038.30
                26  0.70745 -0.00948 -0.00102     72.95   -322.86   -761.92
 26 S03-02 rn   26  0.70745 -0.00948 -0.00102    -28.11    322.85    761.91
                27  0.70745 -0.01442 -0.00106     28.11      0.00      0.00
```

APPENDIX 21.3-7 Select Output from *xSECTION*, Compression Column

```
**********************************************************
*                                                        *
*                      xSECTION                          *
*                                                        *
*              DUCTILITY and STRENGTH of                 *
*     Circular, Semi-Circular, full and partial Rings,   *
*  Rectangular, T-, I-, Hammer head, Octagonal, Polygons *
*      or any combination of above shapes forming        *
*         Concrete Sections using Fiber Models           *
*                                                        *
* VER._2.40,_MAR-14-99                                   *
*                                                        *
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*                                                        *
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* In no event shall the author be held liable for        *
* incidental or consequential damages arising out of the *
* use of this program.                                   *
*                                                        *
**********************************************************
This output was generated by running:
xSECTION
VER._2.40,_MAR-14-99
LICENSE    (choices: LIMITED/UNLIMITED)
UNLIMITED
ENTITY     (choices: GOVERNMENT/CONSULTANT)
Government
NAME_OF_FIRM
Caltrans
BRIDGE_NAME
EXAMPLE
BRIDGE_NUMBER
99-9999
JOB_TITLE
PROTYPE BRIDGE - BRIDGE DESIGN ACADEMY

Concrete Type Information:

     ----------strains--------  --------strength--------
Type   e0     e2     ecc     eu     f0    f2    fcc    fu    E     W
 1 0.0020 0.0040 0.0055 0.0145  5.28  6.98  7.15  6.11  4313   148
 2 0.0020 0.0040 0.0020 0.0050  5.28  3.61  5.28  2.64  4313   148

Steel Type Information:

     -----strains------  --strength-
Type  ey     eh     eu     fy    fu    E
 1 0.0023 0.0150 0.0900 68.00 95.00 29000
 2 0.0023 0.0075 0.0600 68.00 95.00 29000

Steel Fiber Information:

Fiber          xc       yc      area
No.   type     in       in      in^2
  1    2    31.93    0.00    2.25
  2    2    31.00    7.64    2.25
...
25    2    28.27  -14.84    2.25
 26    2    31.00   -7.64    2.25
```

```
Force Equilibrium Condition of the x-section:

     Max.             Max.
     Conc.    Neutral Steel            Steel
     Strain    Axis   Strain Conc.     force        P/S     Net  Curvature Moment
step epscmax   in.    Tens.  Comp.   Comp. Tens.   force   force  rad/in  (K-ft)
 0   0.00000    0.00  0.0000     0      0     0      0      0.00 0.000000       0
 1   0.00029  -29.19  0.0000  2256    222    -2      0      1.88 0.000004    2346
...
24   0.00291   14.11 -0.0061  3742    926 -2194      0      0.52 0.000133   13492
25   0.00322   14.74 -0.0070  3778    963 -2268      0     -1.28 0.000152   13683
26   0.00356   15.28 -0.0081  3813    991 -2330      0      0.34 0.000172   13834
27   0.00394   15.73 -0.0092  3856   1018 -2399      0      0.71 0.000194   14012
28   0.00435   16.07 -0.0104  3904   1049 -2478      0      0.63 0.000219   14204
29   0.00481   16.24 -0.0117  3950   1075 -2552      0     -0.48 0.000244   14332
30   0.00532   16.23 -0.0129  4008   1092 -2623      0      1.90 0.000269   14424
31   0.00588   16.38 -0.0144  4043   1106 -2675      0     -0.34 0.000300   14544
32   0.00650   16.52 -0.0161  4089   1121 -2734      0      1.91 0.000334   14706
33   0.00718   16.66 -0.0180  4135   1137 -2797      0      0.76 0.000372   14879
34   0.00794   16.77 -0.0200  4180   1156 -2862      0      0.35 0.000414   15055
35   0.00878   16.86 -0.0223  4226   1177 -2928      0      1.07 0.000459   15231
36   0.00971   16.91 -0.0248  4271   1201 -2997      0      0.93 0.000509   15403
37   0.01073   16.97 -0.0275  4310   1231 -3069      0     -2.02 0.000565   15573
38   0.01186   16.96 -0.0304  4366   1242 -3132      0      1.47 0.000624   15730
39   0.01312   16.95 -0.0335  4415   1255 -3195      0      0.47 0.000689   15869
40   0.01450   16.91 -0.0370  4458   1269 -3255      0     -1.79 0.000761   15987


First Yield of Rebar Information (not Idealized):

     Rebar Number 20
     Coordinates X and Y (global in.)  -3.85, -31.70
     Yield strain =  0.00230
     Curvature (rad/in)=  0.000057
     Moment (ft-k) =   10802


Cross Section Information:

     Axial Load on Section (kips) =  2474
     Percentage of Main steel in Cross Section =   1.44
     Concrete modulus used in Idealization (ksi) =  4313
     Cracked Moment of Inertia (ft^4) =    25.572


Idealization of Moment-Curvature Curve by Various Methods:


                  Points on Curve        Idealized Values
                  ===============   ============================
Method     Conc.                    Yield         symbol  Plastic
 ID      | Strain  Curv.    Moment | Curv.  Moment   for    Curv.
         | in/in   rad/in  (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000138   13546 0.000071   13546   Mn   0.000689
Strain @ 0.004   0.000198   14042 0.000074   14042   Mn   0.000687
Strain @ 0.005   0.000253   14366 0.000075   14366   Mn   0.000685
CALTRANS 0.00755 0.000392   14964 0.000079   14964   Mp   0.000682
UCSD@5phy0.00558 0.000283   14479 0.000076   14479   Mn   0.000685
```

APPENDIX 21.3-8 Select Output from *xSECTION*, Tension Column

```
05/10/2006, 07:43
************************************************************
*                                                          *
*                       xSECTION                           *
*                                                          *
*               DUCTILITY and STRENGTH of                  *
*      Circular, Semi-Circular, full and partial Rings,    *
*   Rectangular, T-, I-, Hammer head, Octagonal, Polygons  *
*       or any combination of above shapes forming         *
*          Concrete Sections using Fiber Models            *
*                                                          *
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* In no event shall the author be held liable for          *
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* use of this program.                                     *
*                                                          *
************************************************************
This output was generated by running:
xSECTION
VER._2.40,_MAR-14-99
LICENSE     (choices: LIMITED/UNLIMITED)
UNLIMITED
ENTITY      (choices: GOVERNMENT/CONSULTANT)
Government
NAME_OF_FIRM
Caltrans
BRIDGE_NAME
EXAMPLE
BRIDGE_NUMBER
99-9999
JOB_TITLE
PROTYPE BRIDGE - BRIDGE DESIGN ACADEMY

Concrete Type Information:
     ----------strains--------  --------strength--------
Type   e0     e2     ecc     eu      f0     f2    fcc    fu    E     W
 1 0.0020 0.0040 0.0055 0.0145  5.28  6.98  7.15  6.11  4313   148
 2 0.0020 0.0040 0.0020 0.0050  5.28  3.61  5.28  2.64  4313   148

Steel Type Information:

     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0150 0.0900 68.00 95.00 29000
 2 0.0023 0.0075 0.0600 68.00 95.00 29000

Steel Fiber Information:

Fiber          xc       yc      area
No.   type     in       in      in^2
  1    2    31.93    0.00    2.25
  2    2    31.00    7.64    2.25
  ................................
  ................................
 14    2   -31.93    0.00    2.25
 15    2   -31.00   -7.64    2.25
 16    2   -28.27  -14.84    2.25
 17    2   -23.90  -21.17    2.25
 18    2   -18.14  -26.28    2.25
 19    2   -11.32  -29.86    2.25
```

```
 20    2    -3.85  -31.70    2.25
 21    2     3.85  -31.70    2.25
 22    2    11.32  -29.85    2.25
 23    2    18.14  -26.28    2.25
 24    2    23.90  -21.17    2.25
 25    2    28.27  -14.84    2.25
 26    2    31.00   -7.64    2.25

Force Equilibrium Condition of the x-section:

     Max.             Max.
     Conc.    Neutral Steel           Steel
     Strain    Axis    Strain Conc.    force        P/S     Net  Curvature Moment
step epscmax   in.     Tens.  Comp.   Comp. Tens.   force   force  rad/in  (K-ft)
 0   0.00000    0.00  0.0000     0      0     0      0      0.00 0.000000      0
 1   0.00029    2.87 -0.0003   949    131  -173      0     -0.82 0.000009   2393
.............................................................................
.............................................................................
27   0.00394   19.79 -0.0125  2770    862 -2726      0     -0.83 0.000243  11770
28   0.00435   19.97 -0.0140  2820    878 -2792      0     -0.67 0.000272  11964
29   0.00481   20.02 -0.0156  2862    903 -2859      0     -0.64 0.000301  12123
30   0.00532   19.99 -0.0172  2893    936 -2922      0     -0.21 0.000333  12243
31   0.00588   20.03 -0.0191  2927    973 -2993      0      0.00 0.000368  12404
32   0.00650   20.00 -0.0210  2993    980 -3067      0     -0.17 0.000407  12576
33   0.00718   19.97 -0.0232  3066    989 -3148      0      0.19 0.000449  12758
34   0.00794   20.02 -0.0257  3114    993 -3201      0     -0.80 0.000498  12933
35   0.00878   20.05 -0.0285  3160    999 -3252      0     -0.36 0.000551  13102
36   0.00971   20.08 -0.0316  3203   1005 -3302      0     -0.67 0.000611  13262
37   0.01073   20.10 -0.0350  3245   1016 -3355      0     -0.91 0.000676  13418
38   0.01186   20.10 -0.0387  3280   1032 -3405      0      0.08 0.000747  13563
39   0.01312   20.12 -0.0429  3308   1052 -3453      0     -0.24 0.000827  13700
40   0.01450   20.13 -0.0474  3326   1077 -3496      0      0.08 0.000915  13815

First Yield of Rebar Information (not Idealized):

     Rebar Number 20
     Coordinates X and Y (global in.)  -3.85, -31.70
     Yield strain =  0.00230
     Curvature (rad/in)=  0.000051
     Moment (ft-k) =    8190

Cross Section Information:

     Axial Load on Section (kips) =   907
     Percentage of Main steel in Cross Section =   1.44
     Concrete modulus used in Idealization (ksi) =  4313
     Cracked Moment of Inertia (ft^4) =    21.496

Idealization of Moment-Curvature Curve by Various Methods:

                  Points on Curve       Idealized Values
                  ===============   =============================
Method     Conc.                    Yield         symbol  Plastic
 ID      | Strain  Curv.   Moment | Curv.  Moment   for   Curv.
         | in/in   rad/in  (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000176   11159 0.000070   11159   Mn  0.000845
Strain @ 0.004   0.000248   11800 0.000074   11800   Mn  0.000841
Strain @ 0.005   0.000313   12168 0.000076   12168   Mn  0.000839
CALTRANS 0.00673 0.000421   12636 0.000079   12636   Mp  0.000836
UCSD@5phy0.00412 0.000256   11855 0.000074   11855   Mn  0.000841
```

APPENDIX 21.3-9 *wFRAME*, Output File

```
05/15/2006, 08:02
Design Academy Example No: 1 (Bent 2)

************************************************************
*                                                          *
*                       wFRAME                             *
*                                                          *
*         PUSH ANALYSIS of BRIDGE BENTS and FRAMES.        *
*                                                          *
*      Indicates formation of successive plastic hinges.   *
*                                                          *
* VER._1.12,_JAN-14-95                                     *
*                                                          *
* Copyright (C) 1994 By Mark Seyed.                        *
*                                                          *
* This program should not be distributed under any         *
* condition. This release is for demo ONLY (beta testing   *
* is not complete). The author makes no expressed or       *
* implied warranty of any kind with regard to this program.*
* In no event shall the author be held liable for          *
* incidental or consequential damages arising out of the   *
* use of this program.                                     *
*                                                          *
************************************************************

Node Point Information:

Fixity condition definitions:
    s=spring and value
    r=complete release
    f=complete fixity with imposed displacement

node    name    coordinates    -----------fixity --------
  #              X       Y     X-dir.     Y-dir.   Rotation
    1 S01.00     0.00     0.00   r          r          r
    2 S01.01     4.72     0.00   r          r          r
    3 S01.02     7.72     0.00   r          r          r
    4 C01.01     7.72    -3.38   r          r          r
    5 C01.02     7.72   -15.31   r          r          r
    6 C01.03     7.72   -27.24   r          r          r
    7 C01.04     7.72   -39.17   r          r          r
    8 P01.01     7.72   -41.22   s 1.4e+002  r          r
    9 P01.02     7.72   -43.27   s 4.1e+002  r          r
   10 P01.03     7.72   -45.32   s 6.7e+002  r          r
   11 P01.04     7.72   -47.37   f 0.0000   f 0.0000   r
   12 S02.01    10.72     0.00   r          r          r
   13 S02.02    17.72     0.00   r          r          r
   14 S02.03    24.72     0.00   r          r          r
   15 S02.04    31.72     0.00   r          r          r
   16 S02.05    38.72     0.00   r          r          r
   17 S02.06    41.72     0.00   r          r          r
   18 C02.01    41.72    -3.38   r          r          r
   19 C02.02    41.72   -15.31   r          r          r
   20 C02.03    41.72   -27.24   r          r          r
   21 C02.04    41.72   -39.17   r          r          r
   22 P02.01    41.72   -41.22   s 1.4e+002  r          r
   23 P02.02    41.72   -43.27   s 4.1e+002  r          r
   24 P02.03    41.72   -45.32   s 6.7e+002  r          r
   25 P02.04    41.72   -47.37   f 0.0000   f 0.0000   r
   26 S03.01    44.72     0.00   r          r          r
   27 S03.02    49.44     0.00   r          r          r


Spring Information at node points:

k's = k/ft or ft-k/rad.;  d's = ft or rad.
```

```
node   spring      k1      d1      k2      d2
  #    name
    8  P01X01       136.37  0.149         0.00  1.000         0.00 1000.000
    9  P01X02       414.83  0.105         0.00  1.000         0.00 1000.000
   10  P01X03       665.70  0.106         0.00  1.000         0.00 1000.000
   22  P02X01       136.37  0.149         0.00  1.000         0.00 1000.000
   23  P02X02       414.83  0.105         0.00  1.000         0.00 1000.000
   24  P02X03       665.70  0.106         0.00  1.000         0.00 1000.000

Structural Setup:
Spans= 3, Columns= 2, Piles= 2, Link Beams= 0

Element Information:
   element     nodes      depth
 # name fix    i  j     L   d   area    Ei     Ef     Icr     q     Mpp    Mpn  tol status
  1 S01-01 rn   1   2   4.72 6.8 62.6 629528  62953  52.25 -68.40  29928  29928 0.02 e
  2 S01-02 rn   2   3   3.00 6.8 62.6 629528  62953  52.25 -68.40  29928  29928 0.02 e
  3 C01-01 rn   3   4   3.38 6.0 28.3 629528  62953  43.00   0.00  29928  29928 0.02 e
  4 C01-02 rn   4   5  11.93 6.0 28.3 629528  62953  21.50   0.00  12636  12636 0.02 e
  5 C01-03 rn   5   6  11.93 6.0 28.3 629528  62953  21.50   0.00  12636  12636 0.02 e
  6 C01-04 rn   6   7  11.93 6.0 28.3 629528  62953  21.50   0.00  12636  12636 0.02 e
  7 P01-01 rn   7   8   2.05 6.0 28.3 629528  62953  21.50   0.00  12636  12636 0.02 e
  8 P01-02 rn   8   9   2.05 6.0 28.3 629528  62953  21.50   0.00  12636  12636 0.02 e
  9 P01-03 rn   9  10   2.05 6.0 28.3 629528  62953  21.50   0.00  12636  12636 0.02 e
 10 P01-04 rn  10  11   2.05 6.0 28.3 629528  62953  21.50   0.00  12636  12636 0.02 e
 11 S02-01 rn   3  12   3.00 6.8 62.6 629528  62953  52.25 -68.40  29928  29928 0.02 e
 12 S02-02 rn  12  13   7.00 6.8 62.6 629528  62953  52.25 -68.40  29928  29928 0.02 e
 13 S02-03 rn  13  14   7.00 6.8 62.6 629528  62953  52.25 -68.40  29928  29928 0.02 e
 14 S02-04 rn  14  15   7.00 6.8 62.6 629528  62953  52.25 -68.40  29928  29928 0.02 e
 15 S02-05 rn  15  16   7.00 6.8 62.6 629528  62953  52.25 -68.40  29928  29928 0.02 e
 16 S02-06 rn  16  17   3.00 6.8 62.6 629528  62953  52.25 -68.40  29928  29928 0.02 e
 17 C02-01 rn  17  18   3.38 6.0 28.3 629528  62953  51.14   0.00  29928  29928 0.02 e
 18 C02-02 rn  18  19  11.93 6.0 28.3 629528  62953  25.57   0.00  14964  14964 0.02 e
 19 C02-03 rn  19  20  11.93 6.0 28.3 629528  62953  25.57   0.00  14964  14964 0.02 e
 20 C02-04 rn  20  21  11.93 6.0 28.3 629528  62953  25.57   0.00  14964  14964 0.02 e
 21 P02-01 rn  21  22   2.05 6.0 28.3 629528  62953  25.57   0.00  14964  14964 0.02 e
 22 P02-02 rn  22  23   2.05 6.0 28.3 629528  62953  25.57   0.00  14964  14964 0.02 e
 23 P02-03 rn  23  24   2.05 6.0 28.3 629528  62953  25.57   0.00  14964  14964 0.02 e
 24 P02-04 rn  24  25   2.05 6.0 28.3 629528  62953  25.57   0.00  14964  14964 0.02 e
 25 S03-01 rn  17  26   3.00 6.8 62.6 629528  62953  52.25 -68.40  29928  29928 0.02 e
 26 S03-02 rn  26  27   4.72 6.8 62.6 629528  62953  52.25 -68.40  29928  29928 0.02 e
bandwidth of the problem = 10
Number of rows and columns in strage = 81 x 30

Cumulative Results of analysis at end of stage   0

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL= 3381.7)  /    (in)

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00 -0.00603  0.00640 -0.00137
    2 S01.01 -0.00603 -0.00012 -0.00141
  ...........................................
  ............................................
   25 P02.04  0.00000  0.00000 -0.00064
   26 S03.01 -0.00606 -0.00012  0.00141
   27 S03.02 -0.00606  0.00639  0.00137

element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1 -0.00603  0.00640 -0.00137      0.00      0.00      0.00
                2 -0.00603 -0.00012 -0.00141      0.00    322.85   -761.94
  2 S01-02 rn   2 -0.00603 -0.00012 -0.00141      0.01   -322.84    761.93
                3 -0.00603 -0.00450 -0.00153     -0.01    528.04  -2038.28
```

```
 25 S03-01 rn  17 -0.00606 -0.00450  0.00153      0.00    528.05   2038.27
               26 -0.00606 -0.00012  0.00141      0.00   -322.85   -761.93
 26 S03-02 rn  26 -0.00606 -0.00012  0.00141      0.00    322.85    761.93
               27 -0.00606  0.00639  0.00137      0.00      0.00      0.00

Cumulative Results of analysis at end of stage   1

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL= 3381.7)  /    (in)
C02-02     1     rs       0.1760          8.7862

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00  0.73219  0.02482 -0.00348
    2 S01.01  0.73218  0.00836 -0.00351
    3 S01.02  0.73218 -0.00234 -0.00364
    4 C01.01  0.71753 -0.00217 -0.00501
    5 C01.02  0.60724 -0.00158 -0.01304
    6 C01.03  0.41672 -0.00099 -0.01846
    7 C01.04  0.17709 -0.00040 -0.02127
    8 P01.01  0.13324 -0.00030 -0.02149
    9 P01.02  0.08902 -0.00020 -0.02164
   10 P01.03  0.04456 -0.00010 -0.02172
   11 P01.04  0.00000  0.00000 -0.02175
   12 S02.01  0.73219 -0.01192 -0.00274
   13 S02.02  0.73220 -0.02352 -0.00059
   14 S02.03  0.73220 -0.02138  0.00106
   15 S02.04  0.73218 -0.01148  0.00151
   16 S02.05  0.73215 -0.00478  0.00003
   17 S02.06  0.73213 -0.00665 -0.00137
   18 C02.01  0.72472 -0.00618 -0.00300
   19 C02.02  0.62890 -0.00450 -0.01255
   20 C02.03  0.43749 -0.00283 -0.01903
   21 C02.04  0.18720 -0.00115 -0.02242
   22 P02.01  0.14093 -0.00086 -0.02270
   23 P02.02  0.09420 -0.00058 -0.02288
   24 P02.03  0.04717 -0.00029 -0.02299
   25 P02.04  0.00000  0.00000 -0.02302
   26 S03.01  0.73214 -0.01097 -0.00149
   27 S03.02  0.73214 -0.01813 -0.00153

element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1  0.73219  0.02482 -0.00348     29.06     -0.01      0.01
                2  0.73218  0.00836 -0.00351    -29.06    322.86   -761.97
  2 S01-02 rn   2  0.73218  0.00836 -0.00351     74.05   -322.85    761.94
                3  0.73218 -0.00234 -0.00364    -74.05    528.05  -2038.29
  3 C01-01 rn   3  0.00234  0.73218 -0.00364    879.83    248.18  11431.10
                4  0.00217  0.71753 -0.00501   -879.83   -248.18 -10591.82
  4 C01-02 rn   4  0.00217  0.71753 -0.00501    879.86    248.19  10591.19
                5  0.00158  0.60724 -0.01304   -879.86   -248.19  -7630.29
  5 C01-03 rn   5  0.00158  0.60724 -0.01304    879.86    248.21   7630.31
                6  0.00099  0.41672 -0.01846   -879.86   -248.21  -4669.24
  6 C01-04 rn   6  0.00099  0.41672 -0.01846    879.86    248.20   4669.30
                7  0.00040  0.17709 -0.02127   -879.86   -248.20  -1708.25
  7 P01-01 rn   7  0.00040  0.17709 -0.02127    879.86    248.12   1708.95
                8  0.00030  0.13324 -0.02149   -879.86   -248.12  -1200.10
  8 P01-02 rn   8  0.00030  0.13324 -0.02149    879.86    230.01   1200.16
                9  0.00020  0.08902 -0.02164   -879.86   -230.01   -728.78
  9 P01-03 rn   9  0.00020  0.08902 -0.02164    879.86    192.70    729.12
               10  0.00010  0.04456 -0.02172   -879.86   -192.70   -334.12
 10 P01-04 rn  10  0.00010  0.04456 -0.02172    879.86    163.00    334.16
               11  0.00000  0.00000 -0.02175   -879.86   -163.00     -0.02
 11 S02-01 rn   3  0.73218 -0.00234 -0.00364   -137.94    351.79  -9393.48
               12  0.73219 -0.01192 -0.00274    137.94   -146.59  10141.05
```

```
 12 S02-02 rn  12  0.73219 -0.01192 -0.00274    -78.08    146.67 -10141.02
               13  0.73220 -0.02352 -0.00059     78.08    332.13   9491.90
 13 S02-03 rn  13  0.73220 -0.02352 -0.00059      6.47   -332.13  -9491.90
               14  0.73220 -0.02138  0.00106     -6.47    810.93   5491.19
 14 S02-04 rn  14  0.73220 -0.02138  0.00106     91.08   -810.93  -5491.19
               15  0.73218 -0.01148  0.00151    -91.08   1289.73  -1861.15
 15 S02-05 rn  15  0.73218 -0.01148  0.00151    175.49  -1289.73   1861.14
               16  0.73215 -0.00478  0.00003   -175.49   1768.53 -12565.08
 16 S02-06 rn  16  0.73215 -0.00478  0.00003    236.81  -1768.53  12565.06
               17  0.73213 -0.00665 -0.00137   -236.81   1973.73 -18178.47
 17 C02-01 rn  17  0.00665  0.73213 -0.00137   2501.90    348.38  16141.41
               18  0.00618  0.72472 -0.00300  -2501.90   -348.38 -14963.38
 18 C02-02 rs  18  0.00618  0.72472 -0.00300   2501.84    348.05  14964.00
               19  0.00450  0.62890 -0.01255  -2501.84   -348.05 -10811.87
 19 C02-03 rn  19  0.00450  0.62890 -0.01255   2501.84    348.04  10811.93
               20  0.00283  0.43749 -0.01903  -2501.84   -348.04  -6659.75
 20 C02-04 rn  20  0.00283  0.43749 -0.01903   2501.84    348.03   6659.80
               21  0.00115  0.18720 -0.02242  -2501.84   -348.03  -2507.76
 21 P02-01 rn  21  0.00115  0.18720 -0.02242   2501.84    347.72   2507.71
               22  0.00086  0.14093 -0.02270  -2501.84   -347.72  -1795.38
 22 P02-02 rn  22  0.00086  0.14093 -0.02270   2501.84    328.53   1795.20
               23  0.00058  0.09420 -0.02288  -2501.84   -328.53  -1121.56
 23 P02-03 rn  23  0.00058  0.09420 -0.02288   2501.84    289.56   1122.16
               24  0.00029  0.04717 -0.02299  -2501.84   -289.56   -528.43
 24 P02-04 rn  24  0.00029  0.04717 -0.02299   2501.84    257.88    528.77
               25  0.00000  0.00000 -0.02302  -2501.84   -257.88     -0.07
 25 S03-01 rn  17  0.73213 -0.00665 -0.00137    -74.72    528.17   2038.64
               26  0.73214 -0.01097 -0.00149     74.72   -322.97   -761.94
 26 S03-02 rn  26  0.73214 -0.01097 -0.00149    -28.84    322.85    761.91
               27  0.73214 -0.01813 -0.00153     28.84      0.00      0.01

.................................................................
.................................................................
.................................................................
.................................................................

Cumulative Results of analysis at end of stage   6

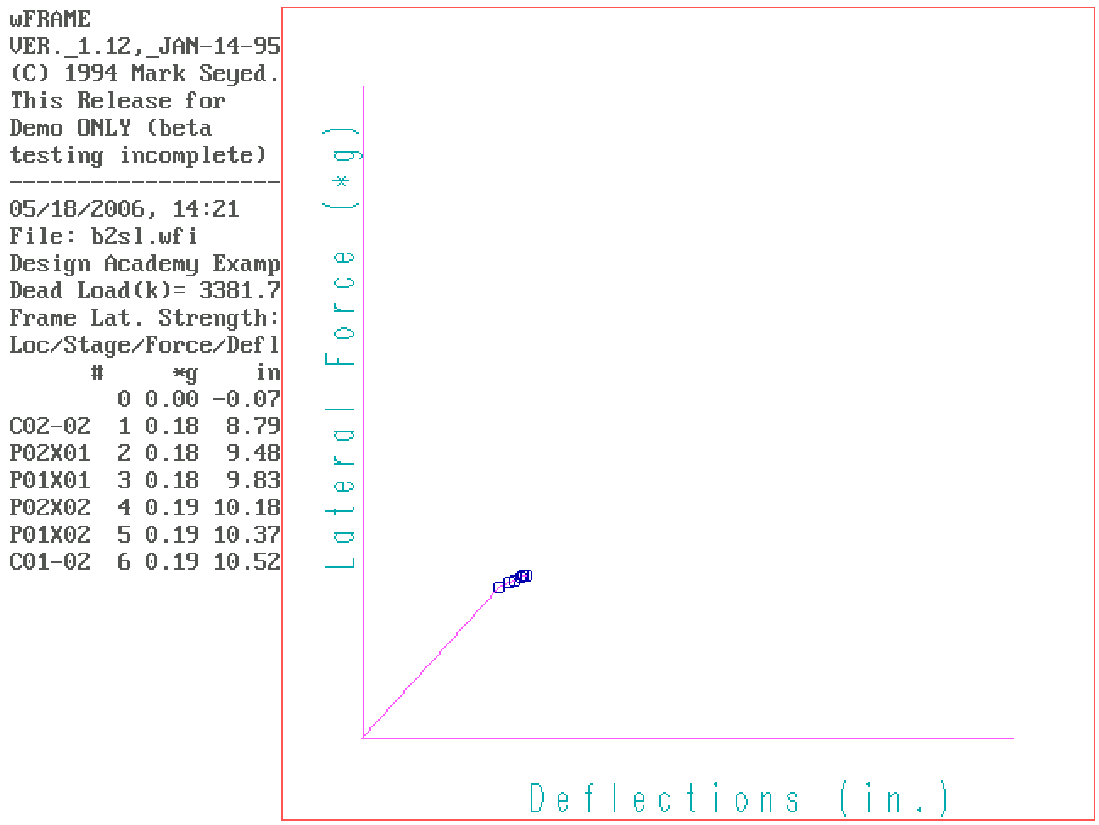
Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL= 3381.7)  /    (in)
C02-02     1     rs       0.1760          8.7862
P02X01     2      2       0.1818          9.4798
P01X01     3      2       0.1847          9.8322
P02X02     4      2       0.1875         10.1774
P01X02     5      2       0.1891         10.3724
C01-02     6     rs       0.1903         10.5239

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00  0.87699  0.03093 -0.00425
    2 S01.01  0.87698  0.01084 -0.00428
    3 S01.02  0.87698 -0.00217 -0.00441
    4 C01.01  0.85928 -0.00201 -0.00605
    5 C01.02  0.72688 -0.00147 -0.01563
    6 C01.03  0.49873 -0.00092 -0.02210
    7 C01.04  0.21194 -0.00038 -0.02546
    8 P01.01  0.15947 -0.00028 -0.02572
    9 P01.02  0.10654 -0.00019 -0.02590
   10 P01.03  0.05333 -0.00009 -0.02600
   11 P01.04  0.00000  0.00000 -0.02603
   12 S02.01  0.87699 -0.01377 -0.00332
   13 S02.02  0.87701 -0.02802 -0.00079
   14 S02.03  0.87702 -0.02622  0.00115
   15 S02.04  0.87700 -0.01502  0.00178
   16 S02.05  0.87697 -0.00605  0.00039
```

```
   17 S02.06  0.87696 -0.00682 -0.00100
   18 C02.01  0.87079 -0.00634 -0.00263
   19 C02.02  0.73519 -0.00462 -0.01588
   20 C02.03  0.50407 -0.00290 -0.02235
   21 C02.04  0.21424 -0.00118 -0.02573
   22 P02.01  0.16121 -0.00089 -0.02600
   23 P02.02  0.10771 -0.00059 -0.02618
   24 P02.03  0.05392 -0.00030 -0.02628
   25 P02.04  0.00000  0.00000 -0.02631
   26 S03.01  0.87696 -0.01003 -0.00112
   27 S03.02  0.87697 -0.01545 -0.00116

element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1  0.87699  0.03093 -0.00425     31.37     -0.01      0.00
                2  0.87698  0.01084 -0.00428    -31.37    322.86   -761.98
  2 S01-02 rn   2  0.87698  0.01084 -0.00428     79.93   -322.85    761.94
                3  0.87698 -0.00217 -0.00441    -79.93    528.05  -2038.30
  3 C01-01 rn   3  0.00217  0.87698 -0.00441    814.86    295.87  13637.29
                4  0.00201  0.85928 -0.00605   -814.86   -295.87 -12636.74
  4 C01-02 rs   4  0.00201  0.85928 -0.00605    814.88    295.88  12636.00
                5  0.00147  0.72688 -0.01563   -814.88   -295.88  -9106.20
  5 C01-03 rn   5  0.00147  0.72688 -0.01563    814.88    295.89   9106.22
                6  0.00092  0.49873 -0.02210   -814.88   -295.89  -5576.25
  6 C01-04 rn   6  0.00092  0.49873 -0.02210    814.88    295.88   5576.32
                7  0.00038  0.21194 -0.02546   -814.88   -295.88  -2046.37
  7 P01-01 rn   7  0.00038  0.21194 -0.02546    814.88    295.73   2047.11
                8  0.00028  0.15947 -0.02572   -814.88   -295.73  -1440.69
  8 P01-02 rn   8  0.00028  0.15947 -0.02572    814.88    275.51   1440.64
                9  0.00019  0.10654 -0.02590   -814.88   -275.51   -876.00
  9 P01-03 rn   9  0.00019  0.10654 -0.02590    814.88    231.57    876.37
               10  0.00009  0.05333 -0.02600   -814.88   -231.57   -401.75
 10 P01-04 rn  10  0.00009  0.05333 -0.02600    814.88    196.02    401.81
               11  0.00000  0.00000 -0.02603   -814.88   -196.02     -0.02
 11 S02-01 rn   3  0.87698 -0.00217 -0.00441   -176.75    286.81 -11599.76
               12  0.87699 -0.01377 -0.00332    176.75    -81.61  12152.41
 12 S02-02 rn  12  0.87699 -0.01377 -0.00332   -112.03     81.70 -12152.37
               13  0.87701 -0.02802 -0.00079    112.03    397.10  11048.49
 13 S02-03 rn  13  0.87701 -0.02802 -0.00079    -20.60   -397.10 -11048.48
               14  0.87702 -0.02622  0.00115     20.60    875.90   6593.00
 14 S02-04 rn  14  0.87702 -0.02622  0.00115     70.88   -875.90  -6593.00
               15  0.87700 -0.01502  0.00178    -70.88   1354.70  -1214.09
 15 S02-05 rn  15  0.87700 -0.01502  0.00178    162.24  -1354.70   1214.09
               16  0.87697 -0.00605  0.00039   -162.24   1833.50 -12372.80
 16 S02-06 rn  16  0.87697 -0.00605  0.00039    228.60  -1833.50  12372.78
               17  0.87696 -0.00682 -0.00100   -228.60   2038.70 -18181.08
 17 C02-01 rn  17  0.00682  0.87696 -0.00100   2566.87    349.18  16143.98
               18  0.00634  0.87079 -0.00263  -2566.87   -349.18 -14963.33
 18 C02-02 rs  18  0.00634  0.87079 -0.00634   2566.81    348.82  14964.00
               19  0.00462  0.73519 -0.01588  -2566.81   -348.82 -10802.60
 19 C02-03 rn  19  0.00462  0.73519 -0.01588   2566.81    348.82  10802.66
               20  0.00290  0.50407 -0.02235  -2566.81   -348.82  -6641.19
 20 C02-04 rn  20  0.00290  0.50407 -0.02235   2566.81    348.81   6641.24
               21  0.00118  0.21424 -0.02573  -2566.81   -348.81  -2479.92
 21 P02-01 rn  21  0.00118  0.21424 -0.02573   2566.81    348.49   2479.88
               22  0.00089  0.16121 -0.02600  -2566.81   -348.49  -1765.94
 22 P02-02 rn  22  0.00089  0.16121 -0.02600   2566.81    328.24   1765.77
               23  0.00059  0.10771 -0.02618  -2566.81   -328.24  -1092.73
 23 P02-03 rn  23  0.00059  0.10771 -0.02618   2566.81    284.77   1093.33
               24  0.00030  0.05392 -0.02628  -2566.81   -284.77   -509.41
 24 P02-04 rn  24  0.00030  0.05392 -0.02628   2566.81    248.60    509.76
               25  0.00000  0.00000 -0.02631  -2566.81   -248.60     -0.07
 25 S03-01 rn  17  0.87696 -0.00682 -0.00100    -80.75    528.17   2038.65
               26  0.87696 -0.01003 -0.00112     80.75   -322.97   -761.94
 26 S03-02 rn  26  0.87696 -0.01003 -0.00112    -31.13    322.85    761.91
               27  0.87697 -0.01545 -0.00116     31.13      0.00      0.01
```

APPENDIX 21.3-10 Force – Displacement Relationship, Bent 2, Right Push with Overturning



APPENDIX 21.3-11 Joint Movement Calculation

STATE OF CALIFORNIA.DEPARTMENT OF TRANSPORTATION

JOINT MOVEMENTS CALCULATIONS [a]

DS-D-0129(Rev.5/93)

Note: Specific instructions are included as footnotes.

EA	DISTRICT	COUNTY	ROUTE	PM (KP)	BRIDGE NAME AND NUMBER
910076	59	ES	999	99	Prototype Bridge

TYPE OF STRUCTURE	TYPE ABUTMENT	TYPE EXPANSION(2" elasto pads, etc.)
CIP/PS BOX GIRDER	Seat	Elastomeric Bearing Pads

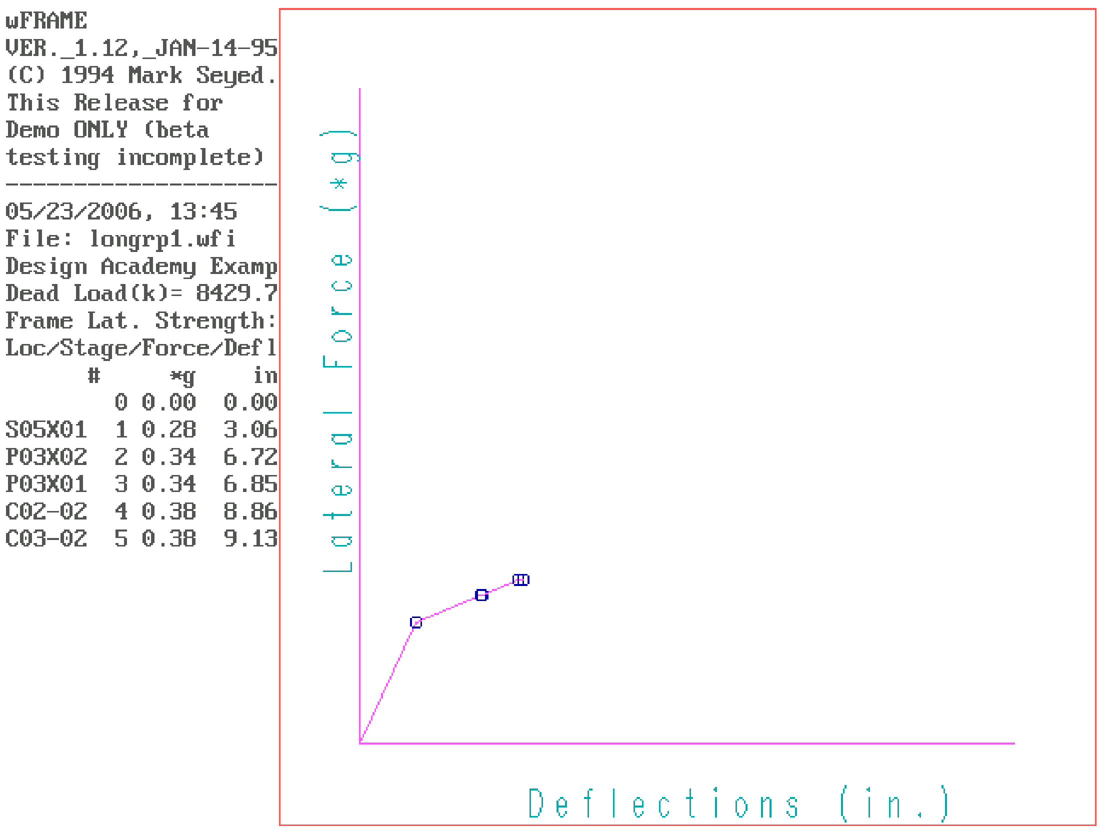
(1) TEMPERATURE EXTREMES(from Preliminary Rerport)		**Type Of Structure**		(2)THERMAL MOVEMENT (inches/100 feet)		ANTICIPATED SHORTENING (inches/100 feet)		(3)MOVEMENT FACTOR (inches/100 feet)
MAXIMUM	110 °F	Steel	Range(°F)(0.0000065X1200) =		+	0	=	
- MINIMUM	23 °F	Concrete (Conventional)	Range(°F)(0.0000060X1200) =		+	0.06	=	
		Concrete(Pretensioned)	Range(°F)(0.0000060X1200) =		+	0.12 [g]	=	
= Range	87 °F	Concrete(Post Tensioned)	Range(87 °F)(0.0000060X1200) =	0.6264	+	0.63 [g]	=	1.26

ITEM(1) DESIGNER	DATE	ITEM(2)CHECKED BY	DATE
DESIGNER		CHECKER	
To be filled in by Office of Structures Design [b]		To be filled in by SR [c]	Date:

Location	Skew (degrees) Do not use in calculation	(4) Contributing Length (feet)	Calculated Movement (inches) (3)X(4)/100	M.R. (inches) (Round up to 1/2")	Seal Type A,B, (Others) or Open Joint	Seal Width Limits [d]: Catalog Number	W1 (inches) Maximum	(5) W2 (inches) Min.@Max. Temperature	Groove (saw cut) Width or Installation Width [e]: Structure Temperature (°F) [f]	(6)Adjust from Maximum Temp. (inches) Δ°/(1)X(2)X(4)/100	Width at Temp. Listed (inches) w=(5)+(6)
Abut 1	0	202	2.53	2.50	Joint Seal Assembly(strip seal)						
Abut 4	0	210	2.64	3.00	Joint Seal Assembly(strip seal)						
					see XS-12-59						

$$\text{Anticipated Shortening} = \frac{1.26}{100} \times \left(\frac{202+210}{2} \right) = 2.60\,\text{in.}$$

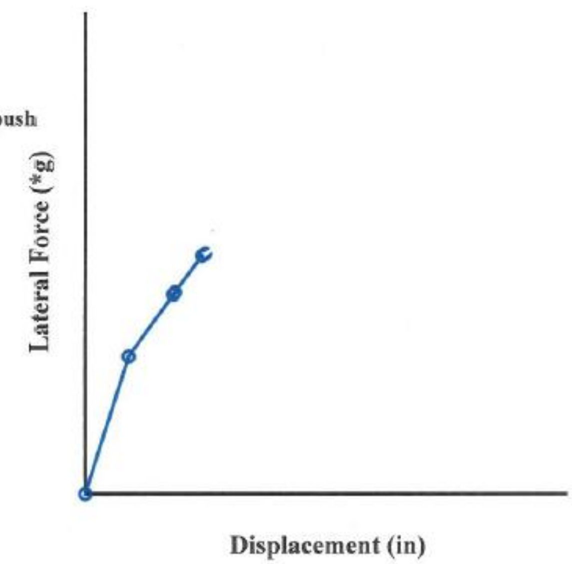
APPENDIX 21.3-12 *wFRAME* Longitudinal Push Over – Force/Displacement Relationship, Right Push



APPENDIX 21.3-13 *wFRAME* Longitudinal Push Over – Force vs. Displacement Relationship

wFRAME
VER._3.00,_JUN-16-05
(C)2005 Mark Mahan
Licensed to:
ZIPPY_ENGINEERING
ROCKET_AVE.
123-456
Sat Oct 26 12:01:50 2013
File: lrp2.wfi
Design Academy Example No: 1 (Superstructure right Push) 2nd push

Force-Deflection Curve:
Last Point on Curve:
Displacement (in) = 9.11
Lateral Force (*g) = 0.22



APPENDIX 21.3-14 Cap Beam – Seismic Moment and Shear Demands

```
05/15/2006, 15:50
Design Academy Example No: 1 (Bent 2)

***********************************************************
*                                                         *
*                      wFRAME                             *
*                                                         *
*         PUSH ANALYSIS of BRIDGE BENTS and FRAMES.       *
*                                                         *
*      Indicates formation of successive plastic hinges.  *
*                                                         *
* VER._1.12,_JAN-14-95                                    *
*                                                         *
* Copyright (C) 1994 By Mark Seyed.                       *
*                                                         *
* This program should not be distributed under any        *
* condition. This release is for demo ONLY (beta testing  *
* is not complete). The author makes no expressed or      *
* implied warranty of any kind with regard to this program.*
* In no event shall the author be held liable for         *
* incidental or consequential damages arising out of the  *
* use of this program.                                    *
*                                                         *
***********************************************************

Node Point Information:

Fixity condition definitions:
    s=spring and value
    r=complete release
    f=complete fixity with imposed displacement

node    name    coordinates    -----------fixity --------
  #              X       Y     X-dir.     Y-dir.   Rotation
    1 S01.00     0.00     0.00   r          r          r
    2 S01.01     4.72     0.00   r          r          r
    3 S01.02     7.72     0.00   r          r          r
    4 C01.01     7.72    -3.38   r          r          r
    5 C01.02     7.72   -15.31   r          r          r
    6 C01.03     7.72   -27.24   r          r          r
    7 C01.04     7.72   -39.17   r          r          r
    8 P01.01     7.72   -41.22   s 1.4e+002  r          r
    9 P01.02     7.72   -43.27   s 4.1e+002  r          r
   10 P01.03     7.72   -45.32   s 6.7e+002  r          r
   11 P01.04     7.72   -47.37   f 0.0000   f 0.0000   r
   12 S02.01    10.72     0.00   r          r          r
   13 S02.02    17.72     0.00   r          r          r
   14 S02.03    24.72     0.00   r          r          r
   15 S02.04    31.72     0.00   r          r          r
   16 S02.05    38.72     0.00   r          r          r
   17 S02.06    41.72     0.00   r          r          r
   18 C02.01    41.72    -3.38   r          r          r
   19 C02.02    41.72   -15.31   r          r          r
   20 C02.03    41.72   -27.24   r          r          r
   21 C02.04    41.72   -39.17   r          r          r


Cumulative Results of analysis at end of stage   6

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL= 3381.7)  /     (in)
P02X01     1      2       0.1863          9.3076
P01X01     2      2       0.1958          9.7870
P02X02     3      2       0.1966          9.8292
P01X02     4      2       0.2059         10.3036
C02-02     5     rs       0.2147         10.7599
```

```
C01-02     6     rs       0.2275        12.3779

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00  1.03149  0.03456 -0.00466
    2 S01.01  1.03149  0.01254 -0.00469
    3 S01.02  1.03148 -0.00170 -0.00482
    4 C01.01  1.01183 -0.00158 -0.00679
    5 C01.02  0.85856 -0.00115 -0.01829
    6 C01.03  0.59020 -0.00072 -0.02608
    7 C01.04  0.25113 -0.00029 -0.03015
    8 P01.01  0.18897 -0.00022 -0.03047
    9 P01.02  0.12627 -0.00015 -0.03069
   10 P01.03  0.06321 -0.00007 -0.03081
   11 P01.04  0.00000  0.00000 -0.03085
   12 S02.01  1.03150 -0.01419 -0.00350
   13 S02.02  1.03152 -0.02840 -0.00064
   14 S02.03  1.03153 -0.02512  0.00137
   15 S02.04  1.03151 -0.01281  0.00182
   16 S02.05  1.03148 -0.00492 -0.00001
   17 S02.06  1.03145 -0.00729 -0.00167
   18 C02.01  1.02247 -0.00677 -0.00363
   19 C02.02  0.86615 -0.00493 -0.01853
   20 C02.03  0.59507 -0.00310 -0.02630
   21 C02.04  0.25323 -0.00126 -0.03039
   22 P02.01  0.19057 -0.00095 -0.03072
   23 P02.02  0.12734 -0.00063 -0.03095
   24 P02.03  0.06376 -0.00032 -0.03107
   25 P02.04  0.00000  0.00000 -0.03111
   26 S03.01  1.03146 -0.01250 -0.00179
   27 S03.02  1.03147 -0.02108 -0.00183

element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1  1.03149  0.03456 -0.00466    37.40     -0.01       0.01
                2  1.03149  0.01254 -0.00469   -37.40    322.86    -761.98
  2 S01-02 rn   2  1.03149  0.01254 -0.00469    95.50   -322.85     761.94
                3  1.03148 -0.00170 -0.00482   -95.50    528.05   -2038.30
  3 C01-01 rn   3  0.00170  1.03148 -0.00482   639.94    353.65   16359.82
                4  0.00158  1.01183 -0.00679  -639.94   -353.65  -15163.66
  4 C01-02 rs   4  0.00158  1.01183 -0.00679   639.98    353.64   15163.00
                5  0.00115  0.85856 -0.01829  -639.98   -353.64  -10944.11
  5 C01-03 rn   5  0.00115  0.85856 -0.01829   639.98    353.66   10944.14
                6  0.00072  0.59020 -0.02608  -639.98   -353.66   -6725.06
  6 C01-04 rn   6  0.00072  0.59020 -0.02608   639.98    353.64    6725.13
                7  0.00029  0.25113 -0.03015  -639.98   -353.64   -2506.10
  7 P01-01 rn   7  0.00029  0.25113 -0.03015   639.98    353.51    2506.99
                8  0.00022  0.18897 -0.03047  -639.98   -353.51   -1782.15
  8 P01-02 rn   8  0.00022  0.18897 -0.03047   639.98    333.29    1782.11
                9  0.00015  0.12627 -0.03069  -639.98   -333.29   -1099.10
  9 P01-03 rn   9  0.00015  0.12627 -0.03069   639.98    289.29    1099.54
               10  0.00007  0.06321 -0.03081  -639.98   -289.29    -506.62
 10 P01-04 rn  10  0.00007  0.06321 -0.03081   639.98    247.19     506.68
               11  0.00000  0.00000 -0.03085  -639.98   -247.19       0.00
 11 S02-01 rn   3  1.03148 -0.00170 -0.00482  -211.27    111.89  -14322.39
               12  1.03150 -0.01419 -0.00350   211.27     93.31   14350.28
 12 S02-02 rn  12  1.03150 -0.01419 -0.00350  -133.82    -93.18  -14350.24
               13  1.03152 -0.02840 -0.00064   133.82    571.98   12022.15
 13 S02-03 rn  13  1.03152 -0.02840 -0.00064   -24.49   -571.98  -12022.14
               14  1.03153 -0.02512  0.00137    24.49   1050.78    6342.45
 14 S02-04 rn  14  1.03153 -0.02512  0.00137    84.84  -1050.79   -6342.45
               15  1.03151 -0.01281  0.00182   -84.84   1529.59   -2688.86
 15 S02-05 rn  15  1.03151 -0.01281  0.00182   194.14  -1529.59    2688.86
               16  1.03148 -0.00492 -0.00001  -194.14   2008.39  -15071.78
 16 S02-06 rn  16  1.03148 -0.00492 -0.00001   273.35  -2008.39   15071.76
               17  1.03145 -0.00729 -0.00167  -273.35   2213.59  -21404.73
```

```
17 C02-01 rn  17  0.00729  1.03145 -0.00167   2741.74    417.33  19367.77
              18  0.00677  1.02247 -0.00363  -2741.74   -417.33 -17956.42

18 C02-02 rs  18  0.00677  1.02247 -0.00706   2741.69    417.18  17957.00
              19  0.00493  0.86615 -0.01853  -2741.69   -417.18 -12980.05
19 C02-03 rn  19  0.00493  0.86615 -0.01853   2741.69    417.18  12980.12
              20  0.00310  0.59507 -0.02630  -2741.69   -417.18  -8003.12
20 C02-04 rn  20  0.00310  0.59507 -0.02630   2741.69    417.17   8003.17
              21  0.00126  0.25323 -0.03039  -2741.69   -417.17  -3026.31
21 P02-01 rn  21  0.00126  0.25323 -0.03039   2741.69    416.81   3026.30
              22  0.00095  0.19057 -0.03072  -2741.69   -416.81  -2172.40
22 P02-02 rn  22  0.00095  0.19057 -0.03072   2741.69    396.55   2172.30
              23  0.00063  0.12734 -0.03095  -2741.69   -396.55  -1359.19
23 P02-03 rn  23  0.00063  0.12734 -0.03095   2741.69    353.00   1359.84
              24  0.00032  0.06376 -0.03107  -2741.69   -353.00   -635.98
24 P02-04 rn  24  0.00032  0.06376 -0.03107   2741.69    310.33    636.35
              25  0.00000  0.00000 -0.03111  -2741.69   -310.33     -0.06
25 S03-01 rn  17  1.03145 -0.00729 -0.00167    -96.61    528.16   2038.61
              26  1.03146 -0.01250 -0.00179     96.61   -322.96   -761.93
26 S03-02 rn  26  1.03146 -0.01250 -0.00179    -37.19    322.85    761.91
              27  1.03147 -0.02108 -0.00183     37.19      0.00      0.01
```

APPENDIX 21.3-15 *wFRAME* Select Output File – To Determine Superstructure Forces due to Column Hinging, Case 1

```
10/26/2013, 09:39
Design Academy Example No: 1 (Superstructure Right Push)

************************************************************
*                                                          *
*                       wFRAME                             *
*                                                          *
*         PUSH ANALYSIS of BRIDGE BENTS and FRAMES.        *
*                                                          *
*      Indicates formation of successive plastic hinges.   *
*                                                          *
* VER._1.12,_JAN-14-95                                     *
*                                                          *
* Copyright (C) 1994 By Mark Seyed.                        *
*                                                          *
* This program should not be distributed under any         *
* condition. This release is for demo ONLY (beta testing   *
* is not complete). The author makes no expressed or       *
* implied warranty of any kind with regard to this program.*
* In no event shall the author be held liable for          *
* incidental or consequential damages arising out of the   *
* use of this program.                                     *
*                                                          *
************************************************************

Node Point Information:

Fixity condition definitions:
    s=spring and value
    r=complete release
    f=complete fixity with imposed displacement

node    name    coordinates    -----------fixity --------
  #              X       Y     X-dir.      Y-dir.   Rotation
    1 S01.00     0.00     0.00   r           r           r
    2 S01.01     2.00     0.00   r           r           r
    3 C01.01     2.00    -1.00   r           r           r
    4 P01.01     2.00    -2.00   f 0.0000    f 0.0000    r
    5 S02.01    12.57     0.00   r           r           r
    6 S02.02    23.14     0.00   r           r           r
    7 S02.03    33.71     0.00   r           r           r
    8 S02.04    44.28     0.00   r           r           r
    9 S02.05    54.85     0.00   r           r           r
   10 S02.06    65.42     0.00   r           r           r
   11 S02.07    75.99     0.00   r           r           r
   12 S02.08    86.56     0.00   r           r           r
   13 S02.09    97.13     0.00   r           r           r
   14 S02.10   107.70     0.00   r           r           r
   15 S02.11   115.70     0.00   r           r           r
   16 S02.12   123.70     0.00   r           r           r
   17 S02.13   127.96     0.00   r           r           r
   18 C02.01   127.96    -3.38   r           r           r
   19 C02.02   127.96   -15.31   r           r           r
   20 C02.03   127.96   -27.24   r           r           r
   21 C02.04   127.96   -39.17   r           r           r
   22 P02.01   127.96   -41.22   s 2.7e+002  r            r
   23 P02.02   127.96   -43.27   s 8.3e+002  r            r
   24 P02.03   127.96   -45.32   s 1.3e+003  r            r
   25 P02.04   127.96   -47.37   f 0.0000    f 0.0000    r
   26 S03.01   132.22     0.00   r           r           r
   27 S03.02   140.22     0.00   r           r           r
   28 S03.03   148.22     0.00   r           r           r
   29 S03.04   160.97     0.00   r           r           r
   30 S03.05   173.72     0.00   r           r           r
   31 S03.06   186.47     0.00   r           r           r
   32 S03.07   199.22     0.00   r           r           r
   33 S03.08   211.97     0.00   r           r           r
   34 S03.09   224.72     0.00   r           r           r
```

```
35 S03.10   237.47     0.00   r          r          r
   36 S03.11   250.22     0.00   r          r          r
   37 S03.12   262.97     0.00   r          r          r
   38 S03.13   275.72     0.00   r          r          r
   39 S03.14   283.72     0.00   r          r          r
   40 S03.15   291.72     0.00   r          r          r
   41 S03.16   295.98     0.00   r          r          r
   42 C03.01   295.98    -3.38   r          r          r
   43 C03.02   295.98   -15.33   r          r          r
   44 C03.03   295.98   -27.28   r          r          r
   45 C03.04   295.98   -39.23   r          r          r
   46 P03.01   295.98   -41.46   s 3.2e+002  r          r
   47 P03.02   295.98   -43.69   s 9.2e+002  r          r
   48 P03.03   295.98   -45.92   s 1.5e+003  r          r
   49 P03.04   295.98   -48.15   s 2e+003   r          r
   50 P03.05   295.98   -50.38   f 0.0000   f 0.0000   r
   51 S04.01   300.24     0.00   r          r          r
   52 S04.02   308.24     0.00   r          r          r
   53 S04.03   316.24     0.00   r          r          r
   54 S04.04   326.01     0.00   r          r          r
   55 S04.05   335.78     0.00   r          r          r
   56 S04.06   345.55     0.00   r          r          r
   57 S04.07   355.32     0.00   r          r          r
   58 S04.08   365.09     0.00   r          r          r
   59 S04.09   374.86     0.00   r          r          r
   60 S04.10   384.63     0.00   r          r          r
   61 S04.11   394.40     0.00   r          r          r
   62 S04.12   404.17     0.00   r          r          r
   63 S04.13   413.94     0.00   r          r          r
   64 C04.01   413.94    -1.00   r          r          r
   65 P04.01   413.94    -2.00   f 0.0000   f 0.0000   r
   66 S05.01   415.94     0.00   s 7.1e+003  r          r

Spring Information at node points:

k's = k/ft or ft-k/rad.;  d's = ft or rad.
node   spring      k1      d1      k2      d2
  #    name
   22  P02X01       272.74  0.149         0.00  1.000         0.00 1000.000
   23  P02X02       828.36  0.105         0.00  1.000         0.00 1000.000
   24  P02X03      1326.91  0.106         0.00  1.000         0.00 1000.000
   46  P03X01       317.46  0.149         0.00  1.000         0.00 1000.000
   47  P03X02       919.77  0.110         0.00  1.000         0.00 1000.000
   48  P03X03      1476.21  0.109         0.00  1.000         0.00 1000.000
   49  P03X04      2038.47  0.109         0.00  1.000         0.00 1000.000
   66  S05X01      7061.00  0.272         0.00  1.000         0.00 1000.000

Structural Setup:
Spans= 5, Columns= 4, Piles= 4, Link Beams= 0

Element Information:
   element     nodes      depth
 # name fix    i  j     L   d   area    Ei     Ef     Icr     q     Mpp    Mpn  tol status
  1 S01-01 rn   1   2   2.00 6.8 103.5 629528  60480 826.75  -0.01  99999  99999 0.02 e
  2 C01-01 rs   2   3   1.00 6.0 56.5 629528  62107  94.88   0.00  99999  99999 0.02 e
  3 P01-01 rn   3   4   1.00 6.0 56.5 629528  62107  47.44   0.00  99999  99999 0.02 e
  4 S02-01 rn   2   5  10.57 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
  5 S02-02 rn   5   6  10.57 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
  6 S02-03 rn   6   7  10.57 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
  7 S02-04 rn   7   8  10.57 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
  8 S02-05 rn   8   9  10.57 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
  9 S02-06 rn   9  10  10.57 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 10 S02-07 rn  10  11  10.57 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 11 S02-08 rn  11  12  10.57 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 12 S02-09 rn  12  13  10.57 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 13 S02-10 rn  13  14  10.57 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 14 S02-11 rn  14  15   8.00 6.8 109.6 629528  60480 778.93  -0.01  99999  99999 0.02 e
 15 S02-12 rn  15  16   8.00 6.8 109.6 629528  60480 778.93  -0.01  99999  99999 0.02 e
```

```
16 S02-13 rn  16  17   4.26 6.8 115.6 629528  60480 826.75  -0.01  99999  99999 0.02 e
 17 C02-01 rn  17  18   3.38 6.0 56.5 629528  62107  94.88   0.00  99999  99999 0.02 e
 18 C02-02 rn  18  19  11.93 6.0 56.5 629528  62107  47.44   0.00  32060  34566 0.02 e
 19 C02-03 rn  19  20  11.93 6.0 56.5 629528  62107  47.44   0.00  32060  34566 0.02 e
 20 C02-04 rn  20  21  11.93 6.0 56.5 629528  62107  47.44   0.00  32060  34566 0.02 e
 21 P02-01 rn  21  22   2.05 6.0 56.5 629528  62107  47.44   0.00  32060  34566 0.02 e
 22 P02-02 rn  22  23   2.05 6.0 56.5 629528  62107  47.44   0.00  32060  34566 0.02 e
 23 P02-03 rn  23  24   2.05 6.0 56.5 629528  62107  47.44   0.00  32060  34566 0.02 e
 24 P02-04 re  24  25   2.05 6.0 56.5 629528  62107  47.44   0.00  32060  34566 0.02 e
 25 S03-01 rn  17  26   4.26 6.8 115.6 629528  60480 826.75  -0.01  99999  99999 0.02 e
 26 S03-02 rn  26  27   8.00 6.8 109.6 629528  60480 778.93  -0.01  99999  99999 0.02 e
 27 S03-03 rn  27  28   8.00 6.8 109.6 629528  60480 778.93  -0.01  99999  99999 0.02 e
 28 S03-04 rn  28  29  12.75 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 29 S03-05 rn  29  30  12.75 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 30 S03-06 rn  30  31  12.75 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 31 S03-07 rn  31  32  12.75 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 32 S03-08 rn  32  33  12.75 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 33 S03-09 rn  33  34  12.75 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 34 S03-10 rn  34  35  12.75 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 35 S03-11 rn  35  36  12.75 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 36 S03-12 rn  36  37  12.75 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 37 S03-13 rn  37  38  12.75 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 38 S03-14 rn  38  39   8.00 6.8 109.6 629528  60480 778.93  -0.01  99999  99999 0.02 e
 39 S03-15 rn  39  40   8.00 6.8 109.6 629528  60480 778.93  -0.01  99999  99999 0.02 e
 40 S03-16 rn  40  41   4.26 6.8 115.6 629528  60480 826.75  -0.01  99999  99999 0.02 e
 41 C03-01 rn  41  42   3.38 6.0 56.5 629528  62107  94.44   0.00  99999  99999 0.02 e
 42 C03-02 rn  42  43  11.95 6.0 56.5 629528  62107  47.22   0.00  34512  31835 0.02 e
 43 C03-03 rn  43  44  11.95 6.0 56.5 629528  62107  47.22   0.00  34512  31835 0.02 e
 44 C03-04 rn  44  45  11.95 6.0 56.5 629528  62107  47.22   0.00  34512  31835 0.02 e
 45 P03-01 rn  45  46   2.23 6.0 56.5 629528  62107  47.22   0.00  34512  31835 0.02 e
 46 P03-02 rn  46  47   2.23 6.0 56.5 629528  62107  47.22   0.00  34512  31835 0.02 e
 47 P03-03 rn  47  48   2.23 6.0 56.5 629528  62107  47.22   0.00  34512  31835 0.02 e
 48 P03-04 rn  48  49   2.23 6.0 56.5 629528  62107  47.22   0.00  34512  31835 0.02 e
 49 P03-05 re  49  50   2.23 6.0 56.5 629528  62107  47.22   0.00  34512  31835 0.02 e
 50 S04-01 rn  41  51   4.26 6.8 115.6 629528  60480 826.75  -0.01  99999  99999 0.02 e
 51 S04-02 rn  51  52   8.00 6.8 109.6 629528  60480 778.93  -0.01  99999  99999 0.02 e
 52 S04-03 rn  52  53   8.00 6.8 109.6 629528  60480 778.93  -0.01  99999  99999 0.02 e
 53 S04-04 rn  53  54   9.77 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 54 S04-05 rn  54  55   9.77 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 55 S04-06 rn  55  56   9.77 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 56 S04-07 rn  56  57   9.77 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 57 S04-08 rn  57  58   9.77 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 58 S04-09 rn  58  59   9.77 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 59 S04-10 rn  59  60   9.77 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 60 S04-11 rn  60  61   9.77 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 61 S04-12 rn  61  62   9.77 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 62 S04-13 rn  62  63   9.77 6.8 103.5 629528  60480 731.10  -0.01  99999  99999 0.02 e
 63 C04-01 rs  63  64   1.00 6.0 56.5 629528  62107  94.44   0.00  99999  99999 0.02 e
 64 P04-01 rn  64  65   1.00 6.0 56.5 629528  62107  47.22   0.00  99999  99999 0.02 e
 65 S05-01 rn  63  66   2.00 6.8 103.5 629528  60480 826.75  -0.01  99999  99999 0.02 e
bandwidth of the problem = 11
Number of rows and columns in strage = 198 x 33
-----------------------------------------------------------
-----------------------------------------------------------
-----------------------------------------------------------
Cumulative Results of analysis at end of stage   8

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL=    4.2)  /    (in)
S05X01     1      2      574.4611          3.3364
P03X02     2      2      693.4553          6.8393
P03X01     3      2      697.6155          6.9630
P02X01     4      2      773.7037          9.2504
P02X02     5      2      790.2726          9.7509
P03X03     6      2      800.7841         10.0718
C02-02     7     rs      823.1016         10.7641
C03-02     8     rs      826.2495         10.9793
```

node#	name	---------- GLOBAL	---------	
		Displ.x	Displ.y	Rotation
1	S01.00	0.91494	-0.00139	0.00070
2	S01.01	0.91494	0.00001	0.00070
3	C01.01	0.45747	0.00000	-0.45747
4	P01.01	0.00000	0.00000	-0.45747
5	S02.01	0.91493	0.00733	0.00068
6	S02.02	0.91490	0.01435	0.00064
7	S02.03	0.91487	0.02074	0.00057
8	S02.04	0.91481	0.02619	0.00046
9	S02.05	0.91474	0.03040	0.00033
10	S02.06	0.91466	0.03304	0.00017
11	S02.07	0.91457	0.03381	-0.00003
12	S02.08	0.91446	0.03238	-0.00025
13	S02.09	0.91433	0.02846	-0.00050
14	S02.10	0.91419	0.02172	-0.00078
15	S02.11	0.91409	0.01461	-0.00100
16	S02.12	0.91397	0.00569	-0.00123
17	S02.13	0.91391	0.00017	-0.00136
18	C02.01	0.90584	0.00016	-0.00339
19	C02.02	0.78568	0.00012	-0.01570
20	C02.03	0.54645	0.00007	-0.02377
21	C02.04	0.23382	0.00003	-0.02801
22	P02.01	0.17604	0.00002	-0.02835
23	P02.02	0.11766	0.00001	-0.02858
24	P02.03	0.05892	0.00001	-0.02871
25	P02.04	0.00000	0.00000	0.00000
26	S03.01	0.91389	-0.00523	-0.00118
27	S03.02	0.91386	-0.01338	-0.00086
28	S03.03	0.91381	-0.01907	-0.00057
29	S03.04	0.91372	-0.02355	-0.00015
30	S03.05	0.91360	-0.02325	0.00018
31	S03.06	0.91347	-0.01929	0.00042
32	S03.07	0.91332	-0.01282	0.00058
33	S03.08	0.91314	-0.00498	0.00064
34	S03.09	0.91294	0.00309	0.00061
35	S03.10	0.91273	0.01025	0.00050
36	S03.11	0.91249	0.01537	0.00029
37	S03.12	0.91223	0.01729	0.00000
38	S03.13	0.91195	0.01487	-0.00039
39	S03.14	0.91178	0.01070	-0.00066
40	S03.15	0.91160	0.00425	-0.00096
41	S03.16	0.91150	-0.00020	-0.00113
42	C03.01	0.90448	-0.00018	-0.00301
43	C03.02	0.79901	-0.00014	-0.01407
44	C03.03	0.58202	-0.00009	-0.02167
45	C03.04	0.29496	-0.00004	-0.02580
46	P03.01	0.23698	-0.00003	-0.02619
47	P03.02	0.17826	-0.00003	-0.02646
48	P03.03	0.11906	-0.00002	-0.02662
49	P03.04	0.05959	-0.00001	-0.02671
50	P03.05	0.00000	0.00000	0.00000
51	S04.01	0.91145	-0.00476	-0.00102
52	S04.02	0.91133	-0.01206	-0.00081
53	S04.03	0.91121	-0.01776	-0.00062
54	S04.04	0.91104	-0.02267	-0.00039
55	S04.05	0.91086	-0.02546	-0.00019
56	S04.06	0.91067	-0.02639	-0.00001
57	S04.07	0.91047	-0.02567	0.00015
58	S04.08	0.91025	-0.02355	0.00028
59	S04.09	0.91002	-0.02025	0.00039
60	S04.10	0.90978	-0.01601	0.00047
61	S04.11	0.90953	-0.01107	0.00053
62	S04.12	0.90926	-0.00565	0.00057
63	S04.13	0.90898	-0.00001	0.00058
64	C04.01	0.45449	0.00000	-0.45449
65	P04.01	0.00000	0.00000	-0.45449
66	S05.01	0.90892	0.00116	0.00058

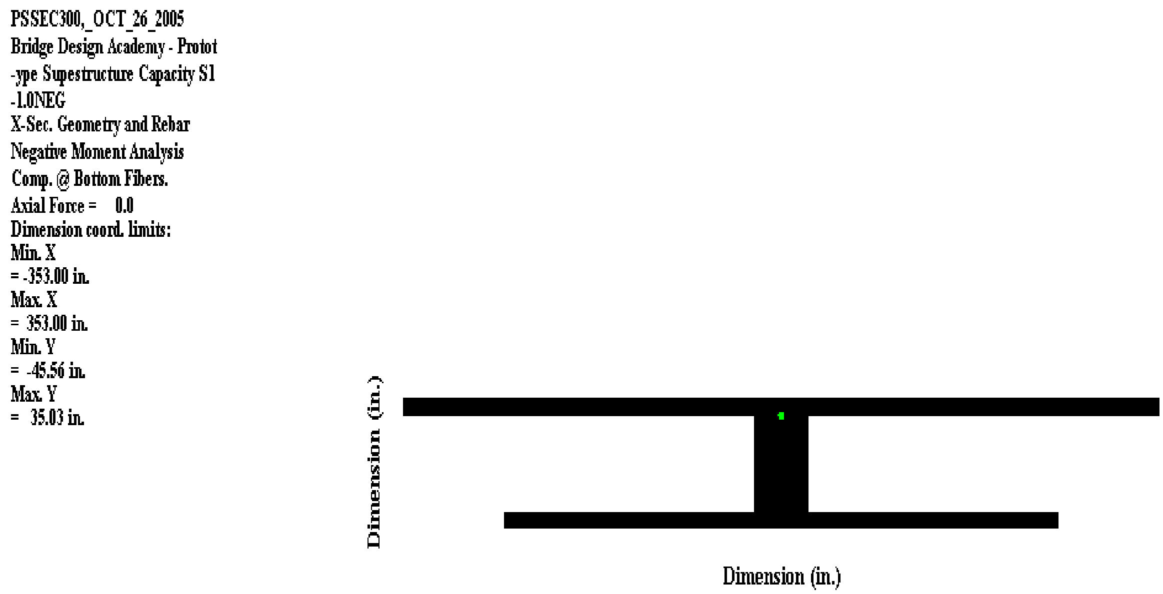
```
element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial     shear     moment
  1 S01-01 rn   1  0.91494 -0.00139  0.00070      7.84     -0.01      -0.02
                2  0.91494  0.00001  0.00070     -7.84      0.03      -0.12
  2 C01-01 rs   2 -0.00001  0.91494 -0.45747   -121.46     -6.52      -0.08
                3  0.00000  0.45747 -0.45747    121.46      6.52      -7.69
  3 P01-01 rn   3  0.00000  0.45747 -0.45747   -121.46      2.15       2.75
                4  0.00000  0.00000 -0.45747    121.46     -2.15      -0.30
  4 S02-01 rn   2  0.91494  0.00001  0.00070     66.87   -121.49       0.11
                5  0.91493  0.00733  0.00068    -66.87    121.59   -1284.78
  5 S02-02 rn   5  0.91493  0.00733  0.00068    154.24   -121.59    1284.77
                6  0.91490  0.01435  0.00064   -154.24    121.70   -2570.58
  6 S02-03 rn   6  0.91490  0.01435  0.00064    241.60   -121.69    2570.61
                7  0.91487  0.02074  0.00057   -241.60    121.80   -3857.45
  7 S02-04 rn   7  0.91487  0.02074  0.00057    328.86   -121.79    3857.49
                8  0.91481  0.02619  0.00046   -328.86    121.90   -5145.39
  8 S02-05 rn   8  0.91481  0.02619  0.00046    416.22   -121.90    5145.38
                9  0.91474  0.03040  0.00033   -416.22    122.01   -6434.50
  9 S02-06 rn   9  0.91474  0.03040  0.00033    503.69   -122.01    6434.51
               10  0.91466  0.03304  0.00017   -503.69    122.12   -7724.70
 10 S02-07 rn  10  0.91466  0.03304  0.00017    590.80   -122.12    7724.80
               11  0.91457  0.03381 -0.00003   -590.80    122.23   -9016.16
 11 S02-08 rn  11  0.91457  0.03381 -0.00003    678.33   -122.23    9016.11
               12  0.91446  0.03238 -0.00025   -678.33    122.33 -10308.64
 12 S02-09 rn  12  0.91446  0.03238 -0.00025    765.50   -122.32   10308.56
               13  0.91433  0.02846 -0.00050   -765.50    122.43 -11602.05
 13 S02-10 rn  13  0.91433  0.02846 -0.00050    852.83   -122.42   11602.07
               14  0.91419  0.02172 -0.00078   -852.83    122.53 -12896.67
 14 S02-11 rn  14  0.91419  0.02172 -0.00078    929.37   -122.52   12896.78
               15  0.91409  0.01461 -0.00100   -929.37    122.60 -13877.32
 15 S02-12 rn  15  0.91409  0.01461 -0.00100    995.44   -122.61   13877.38
               16  0.91397  0.00569 -0.00123   -995.44    122.69 -14858.62
 16 S02-13 rn  16  0.91397  0.00569 -0.00123   1045.39   -122.66   14858.62
               17  0.91391  0.00017 -0.00136  -1045.39    122.71 -15381.25
 17 C02-01 rn  17 -0.00017  0.91391 -0.00136   -129.65    802.62   37277.89
               18 -0.00016  0.90584 -0.00339    129.65   -802.62 -34564.80
 18 C02-02 rs  18 -0.00016  0.90584 -0.00381   -129.70    803.15   34566.00
               19 -0.00012  0.78568 -0.01570    129.70   -803.15 -24984.42
 19 C02-03 rn  19 -0.00012  0.78568 -0.01570   -129.70    803.20   24984.50
               20 -0.00007  0.54645 -0.02377    129.70   -803.20 -15402.33
 20 C02-04 rn  20 -0.00007  0.54645 -0.02377   -129.70    803.19   15402.42
               21 -0.00003  0.23382 -0.02801    129.70   -803.19   -5820.25
 21 P02-01 rn  21 -0.00003  0.23382 -0.02801   -129.70    802.91    5819.92
               22 -0.00002  0.17604 -0.02835    129.70   -802.91   -4173.36
 22 P02-02 rn  22 -0.00002  0.17604 -0.02835   -129.70    763.19    4173.73
               23 -0.00001  0.11766 -0.02858    129.70   -763.19   -2608.48
 23 P02-03 rn  23 -0.00001  0.11766 -0.02858   -129.70    675.43    2608.88
               24 -0.00001  0.05892 -0.02871    129.70   -675.43   -1224.02
 24 P02-04 re  24 -0.00001  0.05892 -0.02871   -129.70    596.96    1223.94
               25  0.00000  0.00000 -0.02876    129.70   -596.96       0.19
 25 S03-01 rn  17  0.91391  0.00017 -0.00136    274.71   -252.37 -21895.31
               26  0.91389 -0.00523 -0.00118   -274.71    252.41   20820.07
 26 S03-02 rn  26  0.91389 -0.00523 -0.00118    323.11   -252.43 -20820.31
               27  0.91386 -0.01338 -0.00086   -323.11    252.51   18800.50
 27 S03-03 rn  27  0.91386 -0.01338 -0.00086    389.44   -252.52 -18800.52
               28  0.91381 -0.01907 -0.00057   -389.44    252.60   16780.05
 28 S03-04 rn  28  0.91381 -0.01907 -0.00057    475.44   -252.60 -16780.07
               29  0.91372 -0.02355 -0.00015   -475.44    252.73   13558.60
 29 S03-05 rn  29  0.91372 -0.02355 -0.00015    580.52   -252.73 -13558.60
               30  0.91360 -0.02325  0.00018   -580.52    252.86   10335.48
 30 S03-06 rn  30  0.91360 -0.02325  0.00018    685.74   -252.86 -10335.48
               31  0.91347 -0.01929  0.00042   -685.74    252.98    7110.72
 31 S03-07 rn  31  0.91347 -0.01929  0.00042    791.29   -252.99   -7110.76
               32  0.91332 -0.01282  0.00058   -791.29    253.11    3884.34
 32 S03-08 rn  32  0.91332 -0.01282  0.00058    896.76   -253.11   -3884.34
               33  0.91314 -0.00498  0.00064   -896.76    253.24     656.30
 33 S03-09 rn  33  0.91314 -0.00498  0.00064   1001.66   -253.24    -656.33
               34  0.91294  0.00309  0.00061  -1001.66    253.37   -2573.29
```

34 S03-10 rn	34	0.91294	0.00309	0.00061	1106.93	-253.37	2573.29
	35	0.91273	0.01025	0.00050	-1106.93	253.49	-5804.54
35 S03-11 rn	35	0.91273	0.01025	0.00050	1211.99	-253.49	5804.56
	36	0.91249	0.01537	0.00029	-1211.99	253.62	-9037.39
36 S03-12 rn	36	0.91249	0.01537	0.00029	1317.09	-253.62	9037.42
	37	0.91223	0.01729	0.00000	-1317.09	253.75	-12271.85
37 S03-13 rn	37	0.91223	0.01729	0.00000	1422.49	-253.74	12271.87
	38	0.91195	0.01487	-0.00039	-1422.49	253.87	-15507.92
38 S03-14 rn	38	0.91195	0.01487	-0.00039	1508.34	-253.87	15507.96
	39	0.91178	0.01070	-0.00066	-1508.34	253.95	-17539.28
39 S03-15 rn	39	0.91178	0.01070	-0.00066	1574.11	-253.95	17539.29
	40	0.91160	0.00425	-0.00096	-1574.11	254.03	-19571.26
40 S03-16 rn	40	0.91160	0.00425	-0.00096	1624.06	-254.03	19571.27
	41	0.91150	-0.00020	-0.00113	-1624.06	254.07	$\boxed{-20653.43}$
41 C03-01 rn	41	0.00020	0.91150	-0.00113	139.21	721.18	34272.36
	42	0.00018	0.90448	-0.00301	-139.21	-721.18	-31834.75
42 C03-02 rs	42	0.00018	0.90448	-0.00301	139.37	721.63	31835.00
	43	0.00014	0.79901	-0.01407	-139.37	-721.63	-23211.55
43 C03-03 rn	43	0.00014	0.79901	-0.01407	139.37	721.60	23211.56
	44	0.00009	0.58202	-0.02167	-139.37	-721.60	-14588.34
44 C03-04 rn	44	0.00009	0.58202	-0.02167	139.37	721.60	14588.36
	45	0.00004	0.29496	-0.02580	-139.37	-721.60	-5965.18
45 P03-01 rn	45	0.00004	0.29496	-0.02580	139.37	721.93	5964.92
	46	0.00003	0.23698	-0.02619	-139.37	-721.93	-4354.71
46 P03-02 rn	46	0.00003	0.23698	-0.02619	139.37	675.41	4356.28
	47	0.00003	0.17826	-0.02646	-139.37	-675.41	-2849.53
47 P03-03 rn	47	0.00003	0.17826	-0.02646	139.37	573.88	2849.93
	48	0.00002	0.11906	-0.02662	-139.37	-573.88	-1570.10
48 P03-04 rn	48	0.00002	0.11906	-0.02662	139.37	412.94	1570.30
	49	0.00001	0.05959	-0.02671	-139.37	-412.94	-649.82
49 P03-05 re	49	0.00001	0.05959	-0.02671	139.37	291.44	649.96
	50	0.00000	0.00000	-0.02673	-139.37	-291.44	0.02
50 S04-01 rn	41	0.91150	-0.00020	-0.00113	935.80	-114.87	$\boxed{-13619.84}$
	51	0.91145	-0.00476	-0.00102	-935.80	114.91	13130.47
51 S04-02 rn	51	0.91145	-0.00476	-0.00102	983.99	-114.90	-13130.79
	52	0.91133	-0.01206	-0.00081	-983.99	114.98	12211.27
52 S04-03 rn	52	0.91133	-0.01206	-0.00081	1049.77	-114.98	-12211.27
	53	0.91121	-0.01776	-0.00062	-1049.77	115.06	11291.09
53 S04-04 rn	53	0.91121	-0.01776	-0.00062	1122.99	-115.04	-11291.09
	54	0.91104	-0.02267	-0.00039	-1122.99	115.14	10166.65
54 S04-05 rn	54	0.91104	-0.02267	-0.00039	1203.85	-115.16	-10166.64
	55	0.91086	-0.02546	-0.00019	-1203.85	115.26	9041.13
55 S04-06 rn	55	0.91086	-0.02546	-0.00019	1284.17	-115.24	-9041.08
	56	0.91067	-0.02639	-0.00001	-1284.17	115.34	7914.66
56 S04-07 rn	56	0.91067	-0.02639	-0.00001	1364.86	-115.36	-7914.64
	57	0.91047	-0.02567	0.00015	-1364.86	115.46	6787.06
57 S04-08 rn	57	0.91047	-0.02567	0.00015	1445.46	-115.47	-6787.07
	58	0.91025	-0.02355	0.00028	-1445.46	115.56	5658.43
58 S04-09 rn	58	0.91025	-0.02355	0.00028	1526.44	-115.57	-5658.45
	59	0.91002	-0.02025	0.00039	-1526.44	115.67	4528.83
59 S04-10 rn	59	0.91002	-0.02025	0.00039	1607.02	-115.68	-4528.86
	60	0.90978	-0.01601	0.00047	-1607.02	115.78	3398.19
60 S04-11 rn	60	0.90978	-0.01601	0.00047	1687.98	-115.78	-3398.23
	61	0.90953	-0.01107	0.00053	-1687.98	115.88	2266.51
61 S04-12 rn	61	0.90953	-0.01107	0.00053	1768.74	-115.89	-2266.52
	62	0.90926	-0.00565	0.00057	-1768.74	115.99	1133.80
62 S04-13 rn	62	0.90926	-0.00565	0.00057	1849.31	-115.99	-1133.79
	63	0.90898	-0.00001	0.00058	-1849.31	116.09	0.10
63 C04-01 rs	63	0.00001	0.90898	-0.45449	116.10	3.57	-4.40
	64	0.00000	0.45449	-0.45449	-116.10	-3.57	-2.28
64 P04-01 rn	64	0.00000	0.45449	-0.45449	116.10	-8.59	-4.71
	65	0.00000	0.00000	-0.45449	-116.10	8.59	-3.97
65 S05-01 rn	63	0.90898	-0.00001	0.00058	1913.55	0.03	-0.06
	66	0.90892	0.00116	0.00058	-1913.55	-0.01	0.02

APPENDIX 21.3-16 *PSSECx* Input File

```
PSSEC300,_OCT_26_2005
Bridge Design Academy - Prototype Supestructure Capacity S1 1.0NEG
Number of different types of concrete
1
For each concrete type input:
Type number; Model code= 0 simple(unconfined/confined), 1 Mander's (unconfined)
strength f'c0 (ksi), strain ec0, strength fcu (ksi), ult. strain ecu, conc. density
1    1
5.200   .002       0.5    0.0025      150
Number of different types of P/S steel
1
For each type, 1st line for tensile parameters,2nd line for cmpressive parameters
type#;E;fy;strain hard. factor;fu;ult. strain;PS-code: 0 tendons, 1 otherwise
      E;fy;strain hard. factor;fu;ult. strain
1   28500    245    2    270   0.030   0
     0        0     0     0      0
Number of different types of mild steel
1
For each steel type input:
Type number;Model code= 0 simple, 1 complex
E(ksi);fy(ksi);strain hard. factor;fu(ksi);ultimate strain
1    1
29000    68   6.41   95   0.09
Number of Conc. Subsections
1
For each Subsec.:Subsection #,Section shape type, Concrete type, No. of fibers
Subsec. Dim.(in):(See Manual for input parameters.)
Subsec. Dim.(in):(See Manual for input parameters.)
Global coord. of the center of Subsec.: Xg, Yg
1  I-shaped,    1   200
706.0  48.0  517.0
81.0  9.125  8.25
0   -5.26
Number of P/S steel groups
1
For each group:group#;P/S type;x-coord.(in);y-coord.(in);area(in^2);P/S force
1    1     0    25.4412     38.28   6157
Number of mild steel rebar cages (rebar distributed around the perimeter)
0
cage#;steel type;cage shape;#of bars;x(in) of 1st bar(y=0);area(in^2)of bar
Number of mild steel groups (no logical pattern for distribution)
2   n
group#;steel type;x-coord.(in); y-coord.(in); area(in^2)
1   1    0   31.80  47.40
2   1    0  -42.13  34.76
Non P/S Axial load on mid-depth of section (Kips)(+ sign=compression)
0
Numerical Computation Factor (1 to 10)
5
Computer Graphics Card identifier: 0 none; 2 CGA; 3 Hercules; 9 EGA; 12 VGA
12
Output control: 0 short; 1 long output
1
X-Sec. plot control (0=no plot, 1=each stage, 2=every iteration of each step)
0
Analysis Control:  p - Positive moment, n - Negative moment
n
```

APPENDIX 21.3-17 *PSSECx* Model for Superstructure



APPENDIX 21.3-18 Partial Output from *PSSECx* Run

```
                                    05-15-2006
********   SECx   ********

        DUCTILITY and STRENGTH of
Rectangular, T-, I-, Hammer, Octagonal, Circular, Ring,
     and Hollowed shaped Prestressed and Reinforced
      Concrete Sections using fiber models
            Ver. 3.00, OCT-26-2005
Copyright (C) 2005 By Mark Seyed and Don Lee.
This program should not be distributed under any condition.
This release is for demo ONLY (beta testing is not complete).

Caltrans or the author make no expressed or implied warranty of any
kind with regard to this program. In no event shall the author or
Caltrans be held liable for incidental or consequential damages
arising out of the use of this program.

JOB TITLE:    Bridge Design Academy - Prototype Supestructure Capacity S1 1.0NEG

       Concrete Data, Complex Model, Mander's unconfined

Concrete Type                        =      1
Compressive Strength (max.)    (ksi)  =  5.200
Strain at max. Strength              = .00200
Strength at Ultimate Strain    (ksi)  =  0.000
Ultimate strain                      = .00500
Unit Weight (pcf)                    = 150.00

       Prestressing Steel Data

 Material  Yield    Hardening  Ultimate  Yield   Ultimate  Modulus
   No.     Strain    Strain     Strain   Stress   Stress   of Elasticity
                                          ksi      ksi      ksi
   1    0.00860   0.00860   0.03000    245.10    270.00  28500.00 Tensile prop.
        0.00000   0.00000   0.00000      0.00      0.00      0.00 Compressive prop.
Prestress element type #      1 is 7-wire and Low-Relaxation Tendon
 with 270 ksi strands.
 ( Refer to PCI Design Handbook 4th Edition.)

       Mild Steel Reinforcing Data

 Material   Yield    Hardening   Ultimate   Yield   Ultimate
   No.      Strain    Strain      Strain    Stress   Stress
                                             ksi      ksi
   1    0.00234    0.01503    0.09000     68.00     95.00

     Rectangular, T-, or I-shaped section information

Depth of Section             (in.)    =   81.00
Top Flange width             (in.)    =  706.00
Top Flange thickness         (in.)    =    9.13
Bot Flange width             (in.)    =  517.00
Bot Flange thickness         (in.)    =    8.25
Web thickness                (in.)    =   48.00

        Concrete fiber information

  Fiber   Material     x       y     area
    #        #       (in)    (in)  (in^2)
    1    1.0      0.00    -45.56    203.11
    2    1.0      0.00    -45.17    203.11
   .................................
   .................................
   .................................
  197    1.0      0.00     33.79    292.83
  199    1.0      0.00     34.62    292.83
  200    1.0      0.00     35.03    292.83
```

```
       Prestressing Steel Fiber Data

 Fiber  Material      x          y         area    P/S force
   No.    No.        (in)       (in)       (in^2)    Kips

     1     1       0.00      25.44      38.28    6157.00

    Total P/S force on the section =    6157.0 kips
    Total moment due to P/S about point (0, 0)  =   13053.5 ft-kip

       Mild Steel Fiber Data

 Fiber  Material      x          y         area
   No.    No.        (in)       (in)       (in^2)

     1     1       0.00      31.80      47.40
     2     1       0.00     -42.13      34.76

Axial load at mid-depth of section (kip)(positive means compression) =     0.0

***************************************************
* Analysis Results ---   Negative Moment Capacity *
***************************************************

 Initial state due to P/S without non-P/S axial force:
 N.A. Loc.     Curvature     Conc. Strain @ max. compressed fiber
    -41.50    0.0000023         0.00017950

 Undeformed P/S element position w.r.t. reference plane
 P/S Fiber    Loc.(y)      Undef. pos.      Conc. Strain @ same loc.
      1      25.44     -0.0058006         -0.0001570

Force Equilibrium Condition of the x-section:

     Max.             Max.
     Conc.    Neutral Steel           Steel
     Strain    Axis   Strain Conc.    force       P/S     Net  Curvature Moment
step epscmax   in.    Tens.  Comp.  Comp. Tens.   force   force  in/in   (K-ft)
 0 -.00001  -41.50 -.00000   5923.   236.    -1. -6157.  -0.8 0.000002      -4.
 1 -.00001  -42.26 0.00000   5923.   235.     0. -6158.  -0.4 0.000002    -147.
 2 -.00001  -43.05 0.00000   5923.   236.     0. -6158.  -0.5 0.000002    -307.
 3 -.00000  -43.86 0.00000   5923.   236.     0. -6159.   0.3 0.000002    -486.
 4 -.00000  -44.70 0.00000   5924.   237.     0. -6160.   0.2 0.000002    -683.
 5 0.00000  -45.56 0.00000   5925.   237.     0. -6161.  -0.7 0.000002    -899.
 6 0.00010 9055.25 0.00000   5983.   237.     0. -6220.  -0.4 -.000000  -13142.
 7 0.00011  362.50 0.00000   5990.   237.     0. -6227.  -0.3 -.000000  -14634.
 8 0.00013  174.76 0.00000   5997.   237.     0. -6235.   0.8 -.000001  -16309.
 9 0.00014  110.77 0.00000   6006.   237.     0. -6244.   0.9 -.000001  -18186.
10 0.00016   78.67 0.00000   6017.   237.     0. -6254.  -0.1 -.000001  -20287.
11 0.00018   59.45 0.00000   6028.   238.     0. -6265.  -0.7 -.000002  -22643.
12 0.00020   46.72 0.00000   6041.   238.     0. -6278.  -0.3 -.000002  -25286.
13 0.00022   37.74 0.00000   6055.   238.     0. -6292.  -1.0 -.000003  -28243.
14 0.00025   29.97 -.00001   6079.   242.    -8. -6312.  -0.1 -.000003  -31443.
15 0.00028   14.77 -.00008   6224.   268.  -109. -6383.   0.4 -.000005  -33995.
16 0.00032    2.23 -.00020   6470.   296.  -269. -6496.  -0.7 -.000007  -36442.
17 0.00035   -6.69 -.00035   6806.   326.  -483. -6648.  -0.9 -.000009  -39119.
18 0.00040  -12.96 -.00055   7231.   359.  -751. -6840.   0.5 -.000012  -42153.
19 0.00045  -17.40 -.00078   7745.   395. -1072. -7069.   0.4 -.000016  -45615.
20 0.00050  -20.61 -.00105   8346.   435. -1445. -7336.   0.3 -.000020  -49549.
21 0.00056  -22.97 -.00136   9034.   480. -1872. -7642.  -0.4 -.000025  -53987.
22 0.00063  -24.75 -.00171   9811.   530. -2354. -7987.  -0.2 -.000030  -58960.
23 0.00071  -26.11 -.00210  10680.   587. -2893. -8372.  -1.0 -.000036  -64494.
24 0.00079  -27.79 -.00266  11498.   645. -3223. -8920.   0.4 -.000045  -69683.
25 0.00089  -30.09 -.00356  12094.   698. -3223. -9568.  -0.5 -.000058  -73488.
26 0.00100  -32.67 -.00499  12356.   739. -3223. -9872.   0.3 -.000077  -75410.
27 0.00112  -34.65 -.00682  12476.   774. -3223.-10027.   0.9 -.000103  -76502.
28 0.00126  -36.06 -.00897  12528.   809. -3223.-10115.   0.1 -.000132  -77210.
```

```
29 0.00141  -37.05 -.01141  12543.   848. -3223.-10168.   0.6 -.000166  -77720.
30 0.00158  -37.76 -.01411  12533.   893. -3223.-10204.   0.9 -.000203  -78121.
31 0.00178  -38.26 -.01704  12639.   948. -3360.-10228.   0.9 -.000243  -79244.
32 0.00199  -38.64 -.02027  12782.  1012. -3546.-10247.  -0.3 -.000288  -80600.
33 0.00223  -38.94 -.02388  12893.  1084. -3716.-10261.   0.6 -.000338  -81787.
34 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.
35 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.
36 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.
37 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.
38 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.
39 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.
40 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.

Prestress Tendon Strain on the x-section:

     Max.
     Conc.    Neutral P/S Steel  Strain
     Strain    Axis
step epscmax   in.   No. Strain  No. Strain  No. Strain  No. Strain  No. Strain
 0 -.00001  -41.50  1 -.005644
 1 -.00001  -42.26  1 -.005644
 2 -.00001  -43.05  1 -.005645
 3 -.00000  -43.86  1 -.005646
 4 -.00000  -44.70  1 -.005647
 5 0.00000  -45.56  1 -.005648
 6 0.00010 9055.25  1 -.005701
 7 0.00011  362.50  1 -.005708
   ..................
   ..................
   ..................

22 0.00063  -24.75  1 -.007321
23 0.00071  -26.11  1 -.007674
24 0.00079  -27.79  1 -.008176
25 0.00089  -30.09  1 -.008996
26 0.00100  -32.67  1 -.010301
27 0.00112  -34.65  1 -.011970
28 0.00126  -36.06  1 -.013932
29 0.00141  -37.05  1 -.016152
30 0.00158  -37.76  1 -.018616
31 0.00178  -38.26  1 -.021292
32 0.00199  -38.64  1 -.024243
33 0.00223  -38.94  1 -.027534
34 0.00000    0.00  1 -.005801
35 0.00000    0.00  1 -.005801
36 0.00000    0.00  1 -.005801
37 0.00000    0.00  1 -.005801
38 0.00000    0.00  1 -.005801
39 0.00000    0.00  1 -.005801
40 0.00000    0.00  1 -.005801

Recommended value of 'effective moment of inertia' based on
     initial slope of moment-curvature diagram (ft^4) = 211.8303

Yield pt. is defined as the First mild steel yields.
The first mild steel yields between the following Steps:    23  and     24
The computation of mild steel yield point IS within 2% tolerance.
The first P/S steel yields between the following Steps:    24  and     25
The computation of P/S steel yield point IS NOT within 2% tolerance.

              Curvature(rad/in)      Moments (ft-K)
Yield              0.000040          67871
Nominal          See force equilibrium table at concrete strain of .003
Ultimate           0.000000              0


End
```

NOTATION

AASHTO	=	*AASHTO LRFD Bridge Design Specifications* with Interims and *California Amendments*
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})
B_{cap}	=	bent cap width (in.)
BDD	=	*Caltrans Bridge Design Details*
B_{eff}	=	effective width of the superstructure for resisting longitudinal seismic moments (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.)
E_c	=	modulus of elasticity of concrete (ksi)
EDA	=	*Elastic Dynamic Analysis*
ESA	=	*Elastic Static Analysis*
F_{sk}	=	abutment shear key force capacity; Shear force associated with column overstrength moment, including overturning effects (ksi)
I_{eff} , I_e	=	effective moment of inertia for computing member stiffness (in.^{4})
ISA	=	*Inelastic Static Analysis*
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)

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.)
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
MTD	=	*Caltrans Memo To Designers*
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)
R_A	=	abutment displacement coefficient
S	=	cap beam short stub length (ft)
SDC	=	Seismic Design Criteria
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.)
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)
a	=	demand spectral acceleration
b_v	=	effective web width taken as the minimum web width within the shear depth d_v (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.)
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)
f_1, f_2	=	concrete shear factors for ductile members
g	=	acceleration due to gravity, 32.2 ft/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_{dia}	=	backwall height for diaphragm abutment (in.)
h_{bw}	=	backwall height for seat abutment (in.)
k_i^e, k_j^e	=	smaller and larger effective bent or column stiffness, respectively (kip/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_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)
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)

s = spacing of shear/transverse reinforcement (in.)

t = top or bottom slab thickness (in.)

v_{jv} = nominal vertical shear stress in a moment resisting joint (psi)

v_c = permissible shear stress carried by concrete (psi)

w = 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_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_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)
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

REFERENCES

1. AASHTO, (2012). "*AASHTO LRFD Bridge Design Specifications*," U.S. Customary Units 2012 (6th Edition), American Association of State Highway and Transportation Officials, Washington, D.C.

2. Caltrans, (2014). *California Amendments to the AASHTO LRFD Bridge Design Specifications – 6th Edition,* California Department of Transportation, Sacramento, CA.

3. Caltrans, (2013). "*Seismic Design Criteria," Version 1.7*, California Department of Transportation, Sacramento, CA.

4. Caltrans (2009). *Bridge Memo To Designers 6-1 Column Analysis Consideration*, California Department of Transportation, Sacramento, CA, February 2009.

5. Caltrans, (2007). *CTBridge Help System, Version 1.3 (Online), Caltrans Bridge Analysis and Design*, California Department of Transportation, Sacramento, CA.

6. Caltrans, (2001a). Bridge *Memo To Designers 20-6 Seismic Strength of Concrete Bridge Superstructures*, California Department of Transportation, Sacramento, CA.

7. Caltrans, (2001b). *Memo To Designers 20-9 Splices in Bar Reinforcing Steel*, California Department of Transportation, Sacramento, CA, August 2001.

8. CSI, (1976-2007). *SAP2000 Advanced 11.0.8, Static and Dynamic Finite Element Analysis of Structures*, Computers and Structures, Inc., Berkeley, CA.

9. Mahan, M., (2006). *User's Manual for "xSECTION," Cross Section Analysis Program*, Version 4.00, California Department of Transportation, Sacramento, CA.

10. Mahan, M., (1995). *User's Manual for "wFRAME," 2-D Push Analysis Program*, Version 1.13, California Department of Transportation, Sacramento, CA.