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 \leq \frac{D_c}{D_s} \leq 1.00 \quad (SDC \ 7.6.1-1)

0.7 \leq \frac{D_{ftg}}{D_c} \quad (SDC \ 7.6.1-2)

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:

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

21.2.1.2 Column Reinforcement Requirements

(1) Longitudinal Reinforcement

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

Minimum Longitudinal Reinforcement Area:

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

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

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, $\rho_s$, shall be sufficient to ensure that the column meets the performance requirements as specified in SDC Section 4.1.

$$\rho_s = \frac{4(A_h)}{D(s)}$$

for columns with circular or interlocking cores (SDC 3.8.1-1)

For rectangular columns with ties and cross ties, the corresponding equation for $\rho_s$, is:

$$\rho_s = \frac{A_v}{D_c s}$$  \hspace{1cm} (SDC 3.8.1-2)

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:
---

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:

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

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

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:

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

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

where:

- $k_i^e$ = smaller effective bent or column stiffness (kip/in.)
- $m_i$ = tributary mass of column or bent $i$ (kip·sec²/ft)
- $k_j^e$ = larger effective bent or column stiffness (kip/in.)
- $m_j$ = tributary mass of column or bent $j$ (kip·sec²/ft)

---

- $\frac{1}{2}$ of the least column cross-section dimension for columns and $\frac{1}{2}$ 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.
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 \(P-\Delta\) 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:

\[
f'_{ce} = \text{Greater of } \begin{cases} 1.3(f'_c) \\ 5,000\text{ psi} \end{cases} \quad (SDC~3.2.6-3)
\]

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:

\[ \frac{T_i}{T_j} \geq 0.7 \]  

(SDC 7.1.2-1)

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 \( \mu \) of 3 to ensure dependable rotational capacity in the plastic hinge regions regardless of the displacement demand imparted to the member.

\[
\Delta_c = \Delta_{y1} + \Delta_p \quad (SDC \ 3.1.3-1)
\]

\[
\Delta_{y1} = \frac{L^2}{3} (\phi_y) \quad (SDC \ 3.1.3-2)
\]

\[
\Delta_p = \theta_p \left( L - \frac{L_p}{2} \right) \quad (SDC \ 3.1.3-3)
\]

\[
\theta_p = L_p \phi_p \quad (SDC \ 3.1.3-4)
\]

\[
\phi_p = \phi_u - \phi_y \quad (SDC \ 3.1.3-5)
\]

\[
\Delta_c = \Delta_{y1} + \Delta_{p1}; \quad \Delta_c = \Delta_{y2} + \Delta_{p2} \quad (SDC \ 3.1.3-6)
\]

\[
\Delta_{y1} = \frac{L^2}{3} (\phi_y); \quad \Delta_{y2} = \frac{L^2}{3} (\phi_y) \quad (SDC \ 3.1.3-7)
\]

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

\[
\theta_{p1} = L_{p1} \phi_{p1}; \quad \theta_{p2} = L_{p2} \phi_{p2} \quad (SDC \ 3.1.3-9)
\]

\[
\phi_{p1} = \phi_{u1} - \phi_y; \quad \phi_{p2} = \phi_{u2} - \phi_y \quad (SDC \ 3.1.3-10)
\]

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.)

$\Delta_p$ = idealized plastic displacement capacity due to rotation of the plastic hinge (in.)

$\Delta_{Y}^{col}$ = idealized yield displacement of the column at the formation of the plastic hinge (in.)

$\phi_Y$ = idealized yield curvature defined by an elastic-perfectly-plastic representation of the cross section’s $M$-$\phi$ curve, see SDC Figures 3.3.1-1 and 3.3.1-2 (rad/in.)

$\phi_p$ = idealized plastic curvature capacity (assumed constant over $L_p$) (rad/in.)

$\theta_p$ = plastic rotation capacity (radian)

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

It is Caltrans’ practice to use an idealized bilinear $M$-$\phi$ 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
21.2.3 Displacement Ductility Demand Requirements

The displacement ductility demand is mathematically defined as

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

(SDC 2.2.3-1)

where:

\( \Delta_D \) = 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, \( \mu_D \), 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:

\[
\Delta_D < \Delta_C \quad \text{(SDC 4.1.1-1)}
\]

where:
\( \Delta_C \) = the bridge or frame displacement capacity when the first ultimate capacity is reached by any plastic hinge (in.)
\( \Delta_D \) = 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, \( \Delta_D \), can be calculated from Equation 21.2-1.

\[
\Delta_D = \frac{ma}{k_e} \quad \text{(21.2-1)}
\]

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:

\[
B_{\text{eff}} = B_{\text{cap}} + (12t)
\]

\((SDC\ 7.3.1.1-1)\)

where:

- \( t \) = thickness of the top or bottom slab (in.)
- \( B_{\text{cap}} \) = 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:

\[
K_{\text{abut}} = K_i \left( \frac{h}{5.5} \right) \quad (SDC \ 7.8.1-2)
\]

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:

$$P_{w} = A_{e}(5) \left( \frac{h_{bw} \text{ or } h_{dia}}{5.5} \right) \text{ kip – ft} \quad (SDC 7.8.1-3)$$

where:

$$A_{e} = \begin{cases} h_{bw}w_{bw} & \text{For seat abutments} \\ h_{dia}w_{dia} & \text{For diaphragmabutments} \end{cases} \quad (SDC 7.8.1-4)$$

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:

$$R_{A} = \frac{\Delta_{D}}{\Delta_{eff}} \quad (SDC 7.8.1-5)$$

where:

- $R_{A}$ = abutment displacement coefficient
- $\Delta_{D}$ = the longitudinal displacement demand at the abutment from elastic analysis
- $\Delta_{eff}$ = 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 $P$-$\Delta$ Effects

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

$$P_{dl} \Delta_{r} \leq 0.20M_{p}^{col} \quad (SDC 4.2-1)$$

where:

- $M_{p}^{col}$ = idealized plastic moment capacity of a column calculated from $M$-$\phi$ analysis
- $P_{dl}$ = dead load axial force
- $\Delta_{r}$ = 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:

$$\phi V_n \geq V_0$$  \hspace{1cm} (SDC 3.6.1-1)

where:

$$V_n = V_c + V_s$$  \hspace{1cm} (SDC 3.6.1-2)

$$\phi = 0.90$$

21.2.7.1 Shear Demand $V_o$

Shear demand associated with overstrength moment may be calculated from:

$$V_0 = \frac{M_0^{col}}{L}$$  \hspace{1cm} (21.2-2)

where:

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

$L$ = 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

$$V_c = \nu_c A_c$$  \hspace{1cm} (SDC 3.6.2-1)

where:

$$A_c = (0.8)A_g$$  \hspace{1cm} (SDC 3.6.2-2)

$$\nu_c = f_i f_2 \sqrt{f_c} \leq 4\sqrt{f_c} \hspace{1cm} \text{(Inside the plastic hinge region)}$$  \hspace{1cm} (SDC 3.6.2-3)

$$= 3f_2 \sqrt{f_c} \leq 4\sqrt{f_c} \hspace{1cm} \text{(Outside the plastic hinge region)}$$  \hspace{1cm} (SDC 3.6.2-4)
$0.3 \leq f_1 = \frac{\rho_s f_{yh}}{0.150} + 3.67 - \mu_d \leq 3 \quad (f_{yh} \text{ in ksi}) \quad (SDC \ 3.6.2-5)$

$\rho_s f_{yh} \leq 0.35 \text{ksi} \quad (21.2-3)$

$f_2 = 1 + \frac{P_c}{2000 A_g} < 1.5 \quad (P_c \text{ is in lb, } A_g \text{ is in in.}^2) \quad (SDC \ 3.6.2-6)$

### 21.2.7.3 Transverse Reinforcement Shear Capacity $V_s$

$$V_s = \left( \frac{A_v f_{yw} D_f}{s} \right) \quad (SDC \ 3.6.3-1)$$

where:

$$A_v = n \left( \frac{\pi}{2} \right) A_c \quad (SDC \ 3.6.3-2)$$

$n = \text{ number of individual interlocking spiral or hoop core sections}$

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

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

### 21.2.7.5 Minimum Shear Reinforcement

$$A_{v,min} \geq 0.025 \frac{D_s}{f_{yh}} \quad (\text{in.}^2) \quad (SDC \ 3.6.5.2-1)$$

### 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:

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

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

where:

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

$F_{sk} = \text{ shear force associated with the column overstrength moment, including overturning effects (kip)}$
\[ P = \text{absolute value of the net axial force normal to the shear plane (kip)} \]
\[ = \text{lowest column axial load if net } P \text{ is compressive considering overturning effects} \]
\[ = \text{largest column axial load if net } P \text{ is tensile, considering overturning effects} \]

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

\[
A_{cv} \geq \frac{4.0F_{sk}}{f_c} \quad (SDC \ 7.6.7-3)
\]
\[
A_{cv} \geq 0.67F_{sk} \quad (SDC \ 7.6.7-4)
\]

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 \(M-\phi\) 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 \(\varepsilon_{SU}^R\) 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:

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

\[ t = \text{thickness of the top or bottom slab} \]

### 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.

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

where:

$D_c = \text{cross sectional dimension of the column (in.)}$

$D_s = \text{depth of the superstructure (in.)}$

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.

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.

Since 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.

\[
M_0 = \left( +M_{dl} \right) + \left( +M_{ps} \right) + \left( +M_{eq} \right)
\]

When earthquake forces add to dead load and secondary prestress forces.

\[
-M_0 = \left( +M_{dl} \right) + \left( +M_{ps} \right) + \left( -M_{eq} \right)
\]

When earthquake forces counteract dead load and secondary prestress forces.

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, $M_{o,\text{col}}$.

$$M_{0,\text{soffit}} = M_{dl,\text{soffit}} + M_{ps} + M_{eq}$$

where $M_{0,\text{soffit}}$ is the overstrength column moment.

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

$$M_{eq} = M_{0,\text{soffit}} - (M_{dl,\text{soffit}} + M_{ps})$$

In the above equation, the overstrength column moment $M_{o,\text{col}}$ is given as:

$$M_{o,\text{col}} = 1.2M_{p}$$

(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:

\[ M_{Ds}^L = M_{dl}^L + M_{ps}^L + M_{eq}^L \]  \hspace{1cm} (21.2-7)

\[ M_{Ds}^R = M_{dl}^R + M_{ps}^R + M_{eq}^R \]  \hspace{1cm} (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:

\[ V_{D}^L = V_{dl}^L + V_{ps}^L + V_{eq}^L \]  \hspace{1cm} (21.2-9)

\[ V_{D}^R = V_{dl}^R + V_{ps}^R + V_{eq}^R \]  \hspace{1cm} (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.,

\[ M_{ne}^{sup(R)} \geq \sum M_{dl}^R + M_{ps}^R + M_{eq}^R \]  

(SDC 4.3.2-1)

\[ M_{ne}^{sup(L)} \geq \sum M_{dl}^L + M_{ps}^L + M_{eq}^L \]  

(SDC 4.3.2-2)

where:

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

(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](image-url)

Note: Axial forces not shown
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 \( M-\phi \) 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, \( \varepsilon_{su}^R = 0.03 \) 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
  \[
  A_{ps} = \frac{P_{jack}}{(0.75)(270)} \quad (21.2-11)
  \]
- 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, $p_c$:  
$$p_c \leq 0.25 f'_c$$  (psi)  \hspace{1cm} (SDC 7.4.2-1)

For principal tension, $p_t$:  
$$p_t \leq 12 \sqrt{f'_c}$$  (psi)  \hspace{1cm} (SDC 7.4.2-2)

$$p_i = \frac{f_h + f_v}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2}$$  \hspace{1cm} (SDC 7.4.4.1-1)

$$p_c = \frac{f_h + f_v}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2}$$  \hspace{1cm} (SDC 7.4.4.1-2)

$$v_{jv} = \frac{T_c}{A_{jv}}$$  \hspace{1cm} (SDC 7.4.4.1-3)

$$A_{jv} = l_{ac} B_{cap}$$  \hspace{1cm} (SDC 7.4.4.1-4)

$$f_v = \frac{P_c}{A_{jh}}$$  \hspace{1cm} (SDC 7.4.4.1-5)

$$A_{jh} = (D_c + D_s) B_{cap}$$  \hspace{1cm} (SDC 7.4.4.1-6)

$$f_h = \frac{P_b}{B_{cap} D_s}$$  \hspace{1cm} (SDC 7.4.4.1-7)

where:

$$f_h = \text{average normal stress in the horizontal direction (ksi)}$$
\( f_v \) = 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 \( M_{col0} / h \)) associated with the column overstrength plastic hinging moment, \( M_{col0} \). 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 \( 3.5 \sqrt{f_v} \) (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:

\[ \rho_{s,min} = \frac{3.5 \sqrt{f_v}}{f_{yh}} \text{ (psi) } \quad (SDC \ 7.4.4.2-1) \]

If \( p_t \) is greater than \( 3.5 \sqrt{f_v} \), 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).

\[ S < D_c \]  

(SDC 7.4.3-1)

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

![Bent Cap Top and Bottom Reinforcement](image)

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 \( A_{sv}^{\top} \) is given as

\[ A_{sv}^{\top} = 0.2 \times A_{st} \]  

(SDC 7.4.4.3-1)

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, \( A_{jh} \), shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18 inches.

\[
A_{jh} = 0.1 \times A_{st}
\]  

(SDC 7.4.4.3-2)

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.

\[
A_{sf} \geq \max \left\{ 0.1 \times A_{cap}^{top}, 0.1 \times A_{cap}^{bot} \right\}
\]  

(SDC 7.4.4.3-3)

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°, “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:

\[
A_{j-bar} = 0.08A_{st}
\]  

(SDC 7.4.4.3-4)

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:

\[
\rho_t = 0.4 \left[ \frac{A_{st}}{I_{tac,provided}^2} \right]
\]  

(SDC 7.4.4.3-5)

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where:

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

\[ I_{ac} = \text{actual length of column longitudinal reinforcement embedded into the bent cap (in.)} \]

For interlocking cores, \( \rho_s \) 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:

\[ I_{ac, \text{required}} = 24d \rho \]  

\( (SDC \ 8.2.1-1) \)

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.

Figure \( SDC \ 7.4.5-1 \) Knee Joint Failure Modes
Two cases of Knee joints are identified as follows:

Case 1: \[ S < \frac{D_c}{2} \]  

Case 2: \[ \frac{D_c}{2} \leq S < D_c \]  

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:

\[ A_{u-bar}^{\min} = 0.33 A_{st} \]  

where:

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

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, \( A_{v}^{jv} \) 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.

\[ A_{v}^{jv} = 0.2 \times A_{st} \]  

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, \( A_{v}^{jh} \), 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).

\[ A_{v}^{jh} = 0.1 \times A_{st} \]  

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:

\[
A_y^{sf} \geq \begin{cases} 
0.1 \times A_{cap}^{top} \\
0.1 \times A_{cap}^{bot} 
\end{cases} 
\]  

(SDC 7.4.5.1-6)

where:

\[A_{cap}^{top} = \text{Area of bent cap top flexural steel (in.}^2\text{)}\]

\[A_{cap}^{bot} = \text{Area of bent cap bottom flexural steel (in.}^2\text{)}\]

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:

\[ A_{hc_{min}}^{j} = 0.33A_{u-bar} \]  \hspace{1cm} (SDC 7.4.5.1-7)

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.

\[ \rho_s = \frac{0.76A_{st}}{D_c l_{ac,provided}} \]  \hspace{1cm} (For Case 1 Knee joint)  \hspace{1cm} (SDC 7.4.5.1-9)

\[ \rho_s = 0.4 \left[ \frac{A_{st}}{l_{ac,provided}^2} \right] \]  \hspace{1cm} (For Case 2 Knee joint, Integral bent cap)  \hspace{1cm} (SDC 7.4.5.1-10)

\[ \rho_s = 0.6 \left[ \frac{A_{st}}{l_{ac,provided}^2} \right] \]  \hspace{1cm} (For Case 2 Knee joint, Non-integral bent cap)  \hspace{1cm} (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, \( \rho_s \) shall be calculated on the basis of
$A_{st}$ and $D_e$ 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:

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

*(SDC 7.8.3-1)*

where:

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

---

*Figure SDC 7.8.3-1 Abutment Seat Width Requirements*
\[ \Delta_{p/s} = \text{displacement attributed to pre-stress shortening (in.)} \]
\[ \Delta_{cr+sh} = \text{displacement attributed to creep and shrinkage (in.)} \]
\[ \Delta_{temp} = \text{displacement attributed to thermal expansion and contraction (in.)} \]
\[ \Delta_{eq} = \text{displacement demand, } \Delta_D \text{ for the adjacent frame. Displacement of the abutment is assumed to be zero (in.)} \]

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 \( \Delta_{temp} \), prestress shortening \( \Delta_{p/s} \), creep and shrinkage \( \Delta_{cr+sh} \), and the relative longitudinal earthquake displacement demand between the two frames \( \Delta_{eq} \) - 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:

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

where:

\[
\Delta_{eq} = \sqrt{\left( \Delta_D \right)^2 + \left( \frac{\Delta}{2} \right)^2} \quad (SDC \ 7.2.5.4-2)
\]

- \( \Delta_{eq} \) = 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](image)
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:

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

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

\[
0.5 \leq \alpha \leq 1 \quad (SDC \ 7.8.4-3)
\]

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)

\( \alpha = \) 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:

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

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, \( A_{sk} \)

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

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

\[
0.4A_{cv} < F_{sk} \leq \min\left(\frac{0.25f_{ye}A_{cv}}{1.5A_{cv}}\right) \quad (SDC \ 7.8.4.1A-3)
\]
\[ A_{sk} \geq \frac{0.05A_{cv}}{f_{ye}} \]  \hspace{1cm} (SDC 7.8.4.1A-4)

where:
\[ A_{cv} = \text{area of concrete engaged in interface shear transfer (in.}^2\)

In the above equations, \( f_{se} \) 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), \( A_{sh} \)**

\[ A_{sh} = (2.0)A_{sk(provided)}^{Iso} \] \hspace{1cm} Isolated shear key \hspace{1cm} (SDC 7.8.4.1B-1)

\[ A_{sh} = \max \left[ \frac{F_{sk}}{f_{ye}} \right] A_{sk(provided)}^{Non-iso} \] \hspace{1cm} Non-isolated shear key \hspace{1cm} (SDC 7.8.4.1B-2)

where:
\[ A_{sk(provided)}^{Iso} = \text{area of interface shear reinforcement provided in SDC Equation 7.8.4.1A-1(in.}^2\)
\[ A_{sk(provided)}^{Non-iso} = \text{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):

\[ L_{min,hooked} = 0.6(a+b) + l_{dh} \] \hspace{1cm} (SDC 7.8.4.1B-3)
\[ L_{min,headed} = 0.6(a+b) + 3 \text{ in.} \] \hspace{1cm} (SDC 7.8.4.1B-4)

where:
\[ a = \text{vertical distance from the location of the applied force on the shear key to the top surface of the stem wall, taken as one-half the vertical length of the expansion joint filler plus the pad thickness (see Figure SDC 7.8.4.1-1(A))} \]
\[ b = \text{vertical distance from the top surface of the stem wall to the centroid of the lowest layer of shear key horizontal reinforcement} \]
\[ l_{dh} = \text{development length in tension of standard hooked bars as specified in AASHTO (2012)} \]
* 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

1. Select column size, column reinforcement, and bent cap width  
   \((SDC\; Sections\; 3.7.1,\; 3.8.1,\; 7.6.1,\; 8.2.5)\)

2. Perform cross-section analysis to determine  
   column effective moment of inertia  
   (Material properties - \(SDC\; Sections\; 3.2.6,\; 3.2.3\))  
   Ensure that dead load on column \(\sim\) 10% column ultimate compressive capacity \(A_g f_c\)

3. Check span configuration/balanced stiffness  
   \((SDC\; Section\; 7.1.1)\)

4. Check frame geometry  
   \((SDC\; Eq.\; 7.1.2-1)\)

5. Calculate minimum local displacement ductility capacity and demand  
   \((SDC\; Sections\; 3.1.3,\; 3.1.4,\; 3.1.4.1)\)  
   Check that local displacement ductility capacity, \(\mu_c \geq 3\)
6. Perform transverse pushover analysis
   \((SDC\ \text{Section}\ 5.2.3,\ SDC\ \text{Eq.}\ 7.3.1.1-1)\)
   Check that:
   \(\text{(a)}\ \Delta_D < \Delta_C\)
   \((SDC\ \text{Eq.}\ 4.1.1-1)\)
   \(\text{(b)}\ \mu_D \leq \text{target value from } SDC\ \text{Section}\ 2.2.4\)

7. Perform longitudinal pushover analysis
   \((SDC\ \text{Sections}\ 5.2.3,\ 7.8.1)\)
   Check that: \(\text{(a)}\ \Delta_D < \Delta_C\)
   \(\text{(b)}\ \mu_D \leq \text{target value from } SDC\ \text{Section}\ 2.2.4\)

8. Check \(P-\Delta\) effects in transverse and longitudinal directions \((SDC\ \text{Eq.}\ 4.2-1)\)

9. Check bent minimum lateral strength in transverse and longitudinal directions
   \((SDC\ \text{Section}\ 3.5)\)

10. Perform column shear design in transverse and longitudinal directions
    \((SDC\ \text{Sections}\ 3.6.1,\ 3.6.2,\ 3.6.3,\ 3.6.5,\ 4.3.1)\)

11. Design column shear key
    \((SDC\ \text{Section}\ 7.6.7)\)

Are column bases pinned?

\(\text{No}\)

\(\text{Yes}\)
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)
   \[ M_{eq@soffit} = M_{0@soffit} - \left( M_{dl@soffit} + M_{ps@soffit} \right) \]

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 \( \geq 0.6 \text{g} \) ?

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?  

Yes  

20. Determine minimum hinge seat width 
(SDC Section 7.2.5.4)

No

Seat type abutments?  

Yes  

21. Determine minimum abutment seat width 
(SDC Section 7.8.3)

No

22. Design abutment shear key reinforcement 
(SDC Section 7.8.4)

23. Check requirements for No-slip 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: \( f'_c = 4 \text{ ksi} \)
- Reinforcing steel: A706, \( f_y = 60 \text{ ksi} ; E_s = 29,000 \text{ksi} ; f_{ye} = 68 \text{ ksi} ; \)
- \( f_{ue} = 95 \text{ ksi} \)

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: \( 8.0 \pm 0.25 \)
- Peak Ground Acceleration: 0.5g

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 ($M = 8.0 \pm 0.25$)
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 \( 0.70 \leq \left[ D_c / D_s \right] = 0.89 \leq 1.00. \) OK. \( \text{(SDC 7.6.1-1)} \)

21.3.3.2 Bent Cap Width

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

21.3.3.3 Column Longitudinal and Transverse Reinforcement

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

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. < 6x1.693 = 10.2 in. < 72/5 = 14.4 in. OK. \( \text{(SDC Section 8.2.5)} \)

Number of #14 bars = \( \frac{61.07}{2.25} = 27.1 \)

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

Assuming a concrete cover of 2 in. as specified in \textit{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):

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

\[ \therefore \text{Spacing of longitudinal bars} = \frac{\pi d_M}{26} = 7.7 \text{ in.} > 1.5(1.693) \text{ in.} > 1.5 \text{ in.} \] OK. \( \text{(AASHTO 5.10.3.1)} \)

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:

<table>
<thead>
<tr>
<th>Bent 2 ($P_c$) (kip)</th>
<th>Column 1</th>
<th>Column 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,489</td>
<td></td>
<td>1,494</td>
</tr>
<tr>
<td>Bent 3 ($P_c$) (kip)</td>
<td>1,425</td>
<td>1,453</td>
</tr>
</tbody>
</table>

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

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

Bent Cap Weight = 8(6.75)(49.44)(0.150) = 400 kips

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

<table>
<thead>
<tr>
<th>Bent 2 ($P_c$) (kip)</th>
<th>Column 1</th>
<th>Column 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,689</td>
<td></td>
<td>1,694</td>
</tr>
<tr>
<td>Bent 3 ($P_c$) (kip)</td>
<td>1,625</td>
<td>1,653</td>
</tr>
</tbody>
</table>

21.3.4.2 Check Column Dead Load Axial Force Ratio

Using Column 2 of Bent 2 (worst case): $\frac{1694 \times 100\%}{4071.5(4)} = 10.4\% \sim 10\% \quad \text{OK.}$
21.3.4.3 Material and Section Properties for Section Analysis Using xSECTION Program

Expected compressive strength of concrete

\[ f_{ce}' = 1.3(4,000) = 5,200 \text{ psi} * > 5,000 \text{ psi} \quad \text{OK.} \quad (SDC 3.2.6-3) \]

* 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, \( f_{cc}' = 5,200 \text{ psi} \)*
- 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, \( P_c = 1,694 \text{ kips} \).
- Bent 3 Column Axial Force, \( P_c = 1,653 \text{ kips} \).

From \(M-\phi\) analysis results, cracked moment of inertia, \( I_e = 23,717 \text{ ft}^4 \) for Bent 2 columns (See Appendices 21.3-1 and 21.3-2). For Bent 3, \( I_e = 23,612 \text{ ft}^4 \).

21.3.5 Step 3 - Check Span Configuration/Balanced Stiffness

21.3.5.1 Bent 2 Stiffness

\[
E_e = 33(w_c)^{1.5} \sqrt{f_{cc}} \text{ psi} = \frac{33(150)^{1.5} \sqrt{5,200}}{1,000} = 4,372 \text{ ksi} \quad (SDC 3.2.6-1)
\]

\[
k_2^e = \frac{3E_eL_e^2}{L^2} = (2) \left[ \frac{(3)(4,372)(23,717)(12^4)}{44 \times (12)^3} \right] = 87.64 \text{ kip/in.}
\]
21.3.5.2 Bent 3 Stiffness

\[
k_3^e = \left(2 \frac{(3)(4.372)(23.612)(12^4)}{(47)(12)^3} \right) = 71.59 \text{ kip/in.}
\]

\[
m_2 = \text{Total tributary mass at Bent 2} = \frac{(2)(1.694)}{(32.2)(12)} = 8.77 \text{ kip/s}^2/\text{in.}
\]

\[
m_3 = \text{Total tributary mass at Bent 3} = \frac{(2)(1.653)}{(32.2)(12)} = 8.56 \text{ kip/s}^2/\text{in.}
\]

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

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

\[
L = 44 \text{ ft}
\]

\[
\phi_y = 0.000078 \text{ rad/in. \ as read from the } M - \phi \text{ data listed in Appendix 21.3-1.}
\]

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

\[
= 0.08(528) + 0.15(68)(1.693) = 59.51 \text{ in.} > (0.3(68)(1.693)) = 34.54 \text{ in.} \quad \text{OK.} \quad (SDC 7.6.2.1-1)
\]

\[
\Delta_Y = \left(\frac{L^2}{3}\right) \phi_y = \frac{1}{3} (528)^2 (0.000078) = 7.25 \text{ in.} \quad (SDC 3.1.3-2)
\]

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

Plastic rotation, \( \theta_p = L_p \phi_p = 59.51 \times 0.000747 = 0.044454 \text{ rad.} \quad (SDC 3.1.3-4) \)
Plastic displacement, $\Delta_p = \theta_p \left( L - \frac{L_p}{2} \right) = 0.04445 \left( 528 - \frac{59.51}{2} \right) = 22.15$ in. \\

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

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

OK.  

(2) Bent 3 Columns \\

Similarly, $\Delta_p = 24.93$ in., $\Delta_Y = 8.27$ in. \\

$\mu_c = \frac{\Delta_c}{\Delta_y} = \frac{33.20}{8.27} = 4.0 > 3$  

OK.

21.3.7.2 Displacement Ductility Demand 

(1) Bent 2 

The period of fundamental mode of vibration is as:

$$T_2 = 2\pi \sqrt{\frac{m_c}{k_e}} = 2\pi \sqrt{\frac{8.77}{87.64}} = 1.99 \text{ sec}.$$ 

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, $\mu_D = \frac{13.92}{7.25} = 1.9 \leq 5$ 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, $a_3 = 0.33g$  

(21.3.2) 

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

Displacement Demand ductility, $\mu_D = \frac{15.25}{8.27} = 1.8 \leq 5$  

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.

<table>
<thead>
<tr>
<th>$P_c$ (kip)</th>
<th>$M_p$ (kip-ft)</th>
<th>$I_e$ (ft$^4$)</th>
<th>$\phi_y$ (rad/in.)</th>
<th>$\phi_p$ (rad/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,694</td>
<td>13,838</td>
<td>23.717</td>
<td>0.000078</td>
<td>0.000747</td>
</tr>
</tbody>
</table>

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 $A = 62.62 \text{ft}^4$ instead of $A = 66.62 \text{ft}^4$ 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 $= 0.171(3,382) = 578 \text{kips}$, 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.
These values can be quickly checked using simple hand calculations as described below:

\[ M_{\text{overturning}} = 578 \times 44 = 25,432 \text{ kip-ft} \]

Axial compression corresponding to \( M_{\text{overturning}} \): \[ \Delta P = \pm \frac{25,432}{34} = 748 \text{ kips} \]

* Dimensions along the skewed bent line

Figure 21.3-3 Transverse Pushover Analysis Model
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).

<table>
<thead>
<tr>
<th>Column Type</th>
<th>(P_c) (kip)</th>
<th>(M_p) (kip-ft)</th>
<th>(I_e) (ft(^4))</th>
<th>(\phi_y) (rad/in.)</th>
<th>(\phi_p) (rad/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>907</td>
<td>12,636</td>
<td>21.496</td>
<td>0.000079</td>
<td>0.000836</td>
</tr>
<tr>
<td>Compression</td>
<td>2,474</td>
<td>14,964</td>
<td>25.572</td>
<td>0.000079</td>
<td>0.000682</td>
</tr>
</tbody>
</table>

Note that higher compression produces a higher value of \(M_p\) but a reduction in \(\phi_p\). 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 \(\Delta_y\), which was calculated previously based upon the assumption that cap beam is infinitely rigid, is updated to 8.79 inches. The corresponding lateral force = \(0.176 \times (3,382) = 595\) kips.
21.3.8.2 Displacement Ductility Capacity

The main purpose of the preliminary calculation for $A_C$ 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 $\phi_p$, and summarized below:

<table>
<thead>
<tr>
<th>Tension Column</th>
<th>Compression Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L = 44$ ft, $L_p = 59.51$ in.</td>
<td>$L = 44$ ft, $L_p = 59.51$ in.</td>
</tr>
<tr>
<td>$\phi_p = 0.000836$ rad/in.</td>
<td>$\phi_p = 0.000682$ rad/in.</td>
</tr>
<tr>
<td>$\Delta_p = 24.79$ in.</td>
<td>$\Delta_p = 20.22$ in.</td>
</tr>
<tr>
<td>$\Delta_c = 10.52 + 24.79 = 35.31$ in.</td>
<td>$\Delta_c = 8.79 + 20.22 = 29.01$ in.</td>
</tr>
</tbody>
</table>

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.

<table>
<thead>
<tr>
<th>Hinge Location</th>
<th>Yield Displacement (in.)</th>
<th>Plastic Deformation (in.)</th>
<th>Total Displacement Capacity (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression Column Top</td>
<td>8.79</td>
<td>20.22</td>
<td>29.01*</td>
</tr>
<tr>
<td>Tension Column Top</td>
<td>10.52</td>
<td>24.79</td>
<td>35.31</td>
</tr>
</tbody>
</table>

* Critical bent displacement capacity, $A_C$.

21.3.8.3 Displacement Ductility Demand

(1) Bent 2

$$ k_2^c = \frac{F_y}{A_y} = \frac{595}{8.79} = 67.69 \frac{k}{in} $$

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

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

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

$$ \mu_D = \frac{16.02}{8.79} = 1.82 < 5 \quad \text{OK. (SDC Section 2.2.4)} $$
Also, $\Delta_D = 16.02$ in. $< \Delta_C = 29.01$ 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 $P-\Delta$ 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):

\[
k_2^f = \frac{F_y}{\Delta_y} = \frac{640}{9.6} = 66.67 \text{ kip/in.}
\]

\[
T = 2\pi \sqrt{\frac{8.77}{66.67}} = 2.28 \text{ sec.}
\]

\[a_2 = 0.3g; \quad \Delta_D = \frac{8.77(0.32)(32.2)(12)}{66.67} = 16.27 \text{ in.}
\]

\[
\mu_D = \frac{16.27}{9.6} = 1.69 < 5 \quad \text{OK.}
\]

\[\Delta_D = 16.27 \text{ in.} < \Delta_C = 29.01 \text{ in.} \quad \text{OK.}
\]

![Figure 21.3-4 Force – Displacement Relations](image-url)
(2) Bent 3

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

<table>
<thead>
<tr>
<th>Tension Column</th>
<th>Compression Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L = 47\text{ft}, L_p = 62.39\text{in.} )</td>
<td>( L = 47\text{ft}, L_p = 62.39\text{in.} )</td>
</tr>
<tr>
<td>( \phi_p = 0.000842\text{rad/in.} )</td>
<td>( \phi_p = 0.000685\text{rad/in.} )</td>
</tr>
<tr>
<td>( \Delta_p = 27.99\text{in.} )</td>
<td>( \Delta_p = 22.77\text{in.} )</td>
</tr>
<tr>
<td>( \Delta_c = 11.48 + 27.99 = 39.47\text{in.} )</td>
<td>( \Delta_c = 9.71 + 22.77 = 32.48\text{in.} )*</td>
</tr>
</tbody>
</table>

* Critical bent displacement capacity, \( \Delta_c \).

**Seismic Demand**

\[
k^3_e = \frac{F_y}{\Delta_y} = \frac{0.180(3.278)}{9.71} = 60.77\text{kip/in.}
\]

The period of vibration, \( T = 2\pi \sqrt{\frac{8.56}{60.77}} = 2.36\text{sec} \)

From Design Spectrum, the spectral acceleration \( a_3 \) is read as 0.31g.

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

\[
\mu_D = \frac{16.87}{9.71} = 1.74 < 5 \quad \text{OK.}
\]

Also, \( \Delta_D = 16.87\text{in.} < \Delta_C = 32.48\text{in.} \) \quad \text{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:

\[
A_e = h_{bw}w_{bw} = 6.75(46.46) = 313.6\text{ft}^2 \quad (SDC 7.8.1-4)
\]

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

\[
P_w = A_e(5) \left( \frac{h_{bw}}{5.5} \right) = (313.6)(5) \left( \frac{6.75}{5.5} \right) = 1,924\text{kips} \quad (SDC 7.8.1-3)
\]
Using initial embankment fill stiffness,

\[ K_i \approx 50 \left( \frac{\text{kips}}{\text{in.}} \right) \frac{\text{ft}}{50} \]  

(SDC 7.8.1-1)

Initial abutment stiffness

\[ K_{\text{abut}} = K_i w \left( \frac{h}{5.5} \right) = 50(46.46) \left( \frac{6.75}{5.5} \right) = 2,851 \text{ kip/in.} \]  

(SDC 7.8.1-2)

\[ \Delta = \frac{F}{K} = \frac{1,924}{2,851} = 0.67 \text{ in.} \]  

(See Figure 21.3-6)

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

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

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

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, \( m = \frac{W}{g} = \frac{8,430}{32.2 \times 12} = 21.82 \text{kip} - \text{s}^2/\text{in.} \)

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

\[ S_a = 0.48g \]

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

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

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

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

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

\( K_1 = 2,196 \text{ kip/ft} \) and \( \Delta_1 = 0.272 \text{ ft} \)

\( K_2 = 0 \text{ kip/ft} \) and \( \Delta_2 = 1.0 \text{ ft} \)

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:

<table>
<thead>
<tr>
<th>Location</th>
<th>Yield Displacement (Right Push) (in.)</th>
<th>Yield Displacement (Left Push) (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bent 2</td>
<td>8.86</td>
<td>8.36</td>
</tr>
<tr>
<td>Bent 3</td>
<td>9.11</td>
<td>9.84</td>
</tr>
</tbody>
</table>

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.

\[ \Delta p = 22.15 \text{ in. for Bent 2 and } \Delta p = 24.93 \text{ in. for Bent 3}. \]

(1) Bent 2

\[ \text{Min } \mu_c = \frac{\Delta_c}{\Delta_y} = \left( \frac{8.86 + 22.15}{8.86} \right) = 3.5 > 3 \quad \text{OK.} \quad (SDC \text{ Section 3.1.4}) \]

(2) Bent 3

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

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

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

\[ T = 2.03 \text{ sec for which } S_a = 0.35g \]

\[ \Delta_D = 14.12 \text{ in.} \]

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.

\[ \text{Max } \mu_D = \frac{14.12}{8.36} = 1.7 < 5 \quad (\text{Bent 2}) \quad \text{OK.} \quad (SDC \text{ Section 2.2.4}) \]

\[ \text{Max } \mu_D = \frac{14.12}{9.11} = 1.5 < 5 \quad (\text{Bent 3}) \quad \text{OK.} \quad (SDC \text{ Section 2.2.4}) \]
21.3.10 Step 8 - Check $P - \Delta$ Effects

21.3.10.1 Transverse direction

We have relatively heavily loaded tall columns. $P-\Delta$ 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 $\Delta_r = 16.02$ in.

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

(2) Bent 3 Columns

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

Maximum Seismic Displacement $\Delta_r = 16.87$ in.

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

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

21.3.10.2 Longitudinal Direction

(1) Bent 2 Columns

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

(2) Bent 3 Columns

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

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 kips > 0.1(3,383) = 338 kips OK. \((SDC\ Section\ 3.5)\)

2.3.11.2 Longitudinal Direction

From the force deflection data shown in Appendix 21.3-13,  
Minimum lateral strength per column =  
0.22\(\frac{8,430}{2}\) = 927 kips > 0.1\(\frac{8,430}{2}\) = 422 kips OK. \((SDC\ Section\ 3.5)\)

21.3.12 Step 10 - Perform Column Shear Design

21.3.12.1 Transverse Bending

\((1)\) Bent 2  
\[M_0 = 1.2M_p = 1.2(14,964) = 17,957\text{ kip} - \text{ft} \quad \text{(includes overturning effects).}\]

Shear demand associated with overstrength moment is as:  
\[V_0 = \frac{M_0}{L} = \frac{17,957}{44} = 408\text{ kips}\]

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

Concrete Shear Capacity, \(V_c\)

For #8 hoops @ 5 in o.c.,  
\[A_b = 0.79\text{ in.}^2, \quad D' = 72 - 2 - 2 - \frac{1.13}{2} - \frac{1.13}{2} = 66.87\text{ in.}, \quad s = 5\text{ in.}\]

\[\rho_s = \frac{4A_b}{D_s} = 0.009451 \quad (SDC\ 3.8.1-1)\]

\[f_{y_b} = 60\text{ ksi}\]

\[\rho_s f_{y_b} = 0.009451(60) = 0.57 > 0.35\]

\[\therefore \quad \text{Use } \rho_s f_{y_b} = 0.35\text{ksi} \quad (SDC\ Section\ 3.6.2)\]

Using the maximum value of the displacement ductility demand, \(\mu_d = 1.82\) (see calculation for Bent 2 – Transverse pushover analysis), the shear capacity factor \(f_i\) is calculated as:
\[ f_1 = \frac{P_s f_{yyh}}{0.150} + 3.67 - \mu_d = \frac{0.35}{0.150} + 3.67 - 1.82 = 4.18 > 3 \]

\[ \therefore \text{Use } f_1 = 3 \quad (SDC \, 3.6.2-5) \]

\[ f_2 = 1 + \frac{P_c}{2000 A_t} = 1 + \frac{880 \times 10^3}{2000 \left( \frac{\pi}{4} \left(6 \times (12)\right)^2\right)} = 1.11 < 1.5 \quad \text{OK.} \]

\[ \therefore \text{Use } f_2 = 1.11 \]

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.

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

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

\[ \therefore V_c = v_c A_e = 21 \text{kip}(3,257) = 687,227 \text{lb} \approx 687 \text{kips} \]

**Transverse Reinforcement Shear Capacity, \( V_s \)**

\[ V_s = \left( \frac{n \pi A_s f_{yyh} D'}{2s} \right) = \frac{\pi}{2} \left( \frac{0.79(60)(66.87)}{5} \right) = 996 \text{kips} \]

Maximum shear strength is as:

\[ V_{s,\text{max}} = 8 \sqrt{f'_c A_e} = 8 \sqrt{4,000(3,257/1,000)} \]

\[ = 1,648 \text{kips} > 996 \text{kips} \quad \text{OK.} \quad (SDC \, 3.6.5.1-1) \]

Minimum shear reinforcement is as:

\[ A_{v,\text{min}} = 0.025 \left( \frac{D}{f_{yyh}} \right) \quad \text{OK.} \quad (SDC \, 3.6.5.2-1) \]

\[ = 0.025 \left( \frac{66.87(5)}{60} \right) = 0.14 \text{in}^2 < 0.79 \text{in}^2 \]

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

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

21.3.12.2 Longitudinal bending

(1) Bent 2

\[ V_0 = 1.2V_p = 1.2(645/2) = 377 \text{kips} \]

This corresponds to the maximum shear value of \( V_p = 323 \text{kips}/\text{column} \) obtained from the wFRAME pushover analysis.

For \( \mu_D = 1.7, \; f_1 = 4.3 > 3 \). Use \( f_1 = 3 \).

For dead load axial force, factor \( f_2 = 1.21 \)
\( v_c = 230 \text{ psi} \) which gives \( V_c = 749 \text{kips} \)
\( V_s = 996 \text{kips} \) as calculated earlier.

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

(2) Bent 3

\[ V_0 = 1.2V_p = 1.2(629/2) = 378 \text{kips} \]

This corresponds to the maximum shear value of \( V_p = 315 \text{kips}/\text{column} \) obtained from the wFRAME pushover analysis.

For \( \mu_D = 1.5, \; \text{factor} \; 1 = 4.5 > 3 \). Use \( f_1 = 3 \)

For dead load axial force, factor \( f_2 = 1.20 \)
\( v_c = 228 \text{ psi} \) which gives \( V_c = 743 \text{kips} \)
\( V_s = 996 \text{kips} \) as calculated earlier.

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

\[
A_{sk} = \frac{1.2(F_{sk} - 0.25P)}{f_y} \quad (SDC 7.6.7-1)
\]

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

Shear force associated with column overstrength moment is as:

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


Therefore, \( F_{sk} = 419 \) kips

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

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

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

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

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

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

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

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).

\[
\phi P_n = (\phi)(0.85)[0.85f_c'(A_g - A_{st}) + f_yA_{st}] \quad (AASHTO 5.7.4.4-2)
\]

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

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

\[
A_g = 809 \text{ in.}^2 \quad > \quad 419 \text{ in.}^2
\]
Therefore, \( A_{cv,reqd} = 809 \text{ in.}^2 \)

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

Use \( A_c \) diameter = 32 in. (see Figure 21.3-8)

![Figure 21.3-8 Column Shear Key](image)

**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 \( M^{+ve} = 21,189 \text{ kip-ft} \) and \( M^{-ve} = 19,436 \text{ kip-ft} \). Appendixes 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 \( \varepsilon_{SU}^R \) 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 \( M_0 \). As shown in Appendix 21.3-14 (right pushover), bent cap moment demands are:

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

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

The associated shear demand obtained from the above pushover analysis, \( V_o = 2,009 \text{ kips.} \)

### 21.3.14.2 Check Bent Cap Shear Capacity

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

\[
V_n = V_c + V_s + V_p \quad \text{(AASHTO 5.8.3.3-1)}
\]

and

\[
V_n = 0.25 f'_c b_v d_v + V_p \quad \text{(AASHTO 5.8.3.3-2)}
\]

where:

\[
V_c = 0.0316 \beta \sqrt{f'_c b_v d_v} \quad \text{(AASHTO 5.8.3.3-3)}
\]

\[
V_s = \frac{A_s f_y d_v \cot \theta}{s} \quad \text{(AASHTO C5.8.3.3-1)}
\]

\[ V_p = 0 \quad \text{(bent cap is not prestressed)} \]

\( b_v = \) 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 0.9\( d_v \) or 0.72\( h \) (see AASHTO Article 5.8.2.9).

0.72 \( h = 0.72 \times 81 = 58.3 \text{ in.} \)

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

\( d_v = \) cap effective depth = 81-5-1.63/2 = 75.2 in.

0.9\( d_v = 0.9 \times 75.2 = 67.7 \text{ in.} > 58.3 \text{ in.} \)

Therefore, \( d_{v,\text{min}} = 67.7 \text{ in.} \)

Method 1 of AASHTO Article 5.8.3.4 (AASHTO 2012) is used to determine the values of \( \beta \) and \( \theta \) (the bent cap section is non-prestressed and the effect of any axial tension is assumed to be negligible).

\[ \therefore \quad \text{Use } \beta = 2.0 \text{ and } \theta = 45^\circ \text{ per AASHTO Article 5.8.3.4.1.} \]

\[
V_c = 0.0316 \beta \sqrt{f'_c b_v d_v} = 0.0316(2)(\sqrt{4})(96)(67.7) = 821 \text{ kips}
\]

Assuming 6-legged, #6 stirrups @ 7 in. o.c. transverse reinforcement (see Figure 21.3-19).
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>
<thead>
<tr>
<th>Bent</th>
<th>Skew (Degree)</th>
<th>DL $M_z$</th>
<th>DL $M_y$</th>
<th>DL $M_{long}$</th>
<th>ADL $M_z$</th>
<th>ADL $M_y$</th>
<th>ADL $M_{long}$</th>
<th>Sec. PS $M_z$</th>
<th>Sec. PS $M_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>20</td>
<td>-1,189</td>
<td>91</td>
<td>-1,148</td>
<td>-213</td>
<td>17</td>
<td>-207</td>
<td>82</td>
<td>-371</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td>1,305</td>
<td>-1</td>
<td>1,227</td>
<td>234</td>
<td>-1</td>
<td>220</td>
<td>-127</td>
<td>287</td>
</tr>
</tbody>
</table>

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

$V_s = \frac{A_v f_y d_v \cot \theta}{s} = \frac{6(0.44)(60)(67.7)\cot(45)}{7} = 1,532$ kips

$V_n = V_c + V_s = 821 + 1,532 = 2,353$ kips

$V_n = 0.25 f_c b_v d_v = 0.25(4)(96)(67.7) = 6,499$ kip > 2,353 kips

$\therefore V_n = 2,353$ kips

$\phi \times V_n = 0.92(2,353) = 2,118$ kips $> V_0 = 2,009$ kips

OK.
Figure 21.3-9 Coordinate Transformation from Skewed to Unskewed Configuration

\[
\begin{align*}
M'_z &= M_z \cos \theta - M_y \sin \theta \\
M'_y &= M_z \sin \theta + M_y \cos \theta \\
T'_z &= T_z
\end{align*}
\] (Longitudinal Moment) (Transverse Moment) (Torsional Moment)

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

\[
\begin{align*}
\text{Deck Soffit} & \quad 4.6 \text{ kips} \\
\text{Column} & \\
\text{Column} & \quad 4.6 \text{ kips}
\end{align*}
\]

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

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

- Secondary Prestress Moments
  Column moment at base, \( M_{\text{ps}}^{\text{col, bottom}} = 0 \) kip-ft (CTBridge Output)
  Column Moment at deck soffit, \( M_{\text{ps}}^{\text{col, top @ joint}} = -218 \) 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 1/10<sup>th</sup> 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 1/10<sup>th</sup> 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

<table>
<thead>
<tr>
<th>Location</th>
<th>Whole Superstructure Width</th>
<th>Per Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$x/L$</td>
<td>( x ) (ft)</td>
</tr>
<tr>
<td>Support</td>
<td>1.5</td>
<td>619</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>710</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>12158</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>14741</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>14857</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>12508</td>
</tr>
<tr>
<td></td>
<td>0.6</td>
<td>7693</td>
</tr>
<tr>
<td></td>
<td>0.7</td>
<td>88.2</td>
</tr>
<tr>
<td></td>
<td>0.8</td>
<td>100.8</td>
</tr>
<tr>
<td></td>
<td>0.9</td>
<td>142.8</td>
</tr>
<tr>
<td>Support</td>
<td>123</td>
<td>-32599</td>
</tr>
<tr>
<td>Span 1</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>142.8</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>159.6</td>
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<td></td>
<td>0.3</td>
<td>176.4</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>193.2</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>210</td>
</tr>
<tr>
<td></td>
<td>0.6</td>
<td>226.8</td>
</tr>
<tr>
<td></td>
<td>0.7</td>
<td>243.6</td>
</tr>
<tr>
<td></td>
<td>0.8</td>
<td>260.4</td>
</tr>
<tr>
<td></td>
<td>0.9</td>
<td>277.2</td>
</tr>
<tr>
<td>Support</td>
<td>291</td>
<td>-31614</td>
</tr>
<tr>
<td>Span 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>297</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>305.8</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>317.6</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>329.4</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>341.2</td>
</tr>
<tr>
<td></td>
<td>0.6</td>
<td>353</td>
</tr>
<tr>
<td></td>
<td>0.7</td>
<td>364.8</td>
</tr>
<tr>
<td></td>
<td>0.8</td>
<td>376.6</td>
</tr>
<tr>
<td></td>
<td>0.9</td>
<td>388.4</td>
</tr>
<tr>
<td>Support</td>
<td>400.2</td>
<td>6077</td>
</tr>
<tr>
<td>Span 3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>410.5</td>
</tr>
</tbody>
</table>

Chapter 21 – Seismic Design of Concrete Bridges 21-70
(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.

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

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

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

Column moment generated by seismic loading at column soffit is:

\[ M_{eq}^{\text{col @ soffit}} = 1.2 M_p^{\text{col @ soffit}} (M_{dl}^{\text{col @ soffit}} + M_{ps}^{\text{col @ soffit}}) \]

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

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.

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

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

\[ M_{eq}^{col \@ soffit} = 1.2M_p^{col} - (M_{dl}^{col} + M_{ps}^{col}) \]
\[ = 1.2(-13,838) - (-1,355 + 204) = -32,060 \text{ kip ft} \]

Bent 3

\[ M_{eq}^{col \@ soffit} = 1.2M_p^{col} - (M_{dl}^{col} + M_{ps}^{col}) \]
\[ = 1.2(-13,777) - (1,447 - 0) = -34,512 \text{ kip ft} \]

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 $M_{eq,soffit}$ into the Superstructure

The static non-linear “push-over” frame analysis program wFRAME is used to distribute the column earthquake moments $M_{eq,soffit}$ 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:

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

Based on the column location and the girder spacing, it can easily be concluded that the girder aligned along the centerline of the bridge lies outside the effective width. Therefore, at the face of 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.

Case 1

$$M_{el}^L = \{(-6,520) + (-1,164)\}(4) = -30,736 \text{ kip-ft}$$
$$M_{er}^L = \{(-6,731) + (-1,202)\}(4) = -31,732 \text{ kip-ft}$$
$$M_{ps}^L = \{+1,734\}(4) = +6,936 \text{ kip-ft}$$
$$M_{ps}^R = \{+1,694\}(4) = +6,776 \text{ kip-ft}$$
$$M_{eq}^L = -15,015 \text{ kip-ft} \quad \text{(see Table 21.3-3)}$$
$$M_{eq}^R = +21,135 \text{ kip-ft} \quad \text{(see Table 21.3-3)}$$
The superstructure moment demand is then calculated as:

\[ M_D^L = M_{eq}^L + M_{ps}^L + M_{eq}^L = (-30,736) + (6,936^*) + (-15,015) = -45,751 \text{ kip-ft} \]
\[ M_D^R = M_{eq}^R + M_{ps}^R + M_{eq}^R = (-31,732) + (6,776) + (21,135) = -3,821 \text{ kip-ft} \]

Table 21.3-4 lists these superstructure seismic moment demands.

**Case 2**

\[ M_{eq}^L = +13,201 \text{ kip-ft}; \quad M_{eq}^R = -20,299 \text{ kip-ft} \]
\[ M_D^L = (-30,736) + (6,936) + (13,201) = -10,599 \text{ kip-ft} \]
\[ M_D^R = (-31,732) + (6,776^*) + (-20,295) = -52,027 \text{ kip-ft} \]

*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 \( w\text{FRAME} \) Output

<table>
<thead>
<tr>
<th>Location</th>
<th>( M_{EQ} ) (kip-ft)</th>
<th>( w\text{FRAME} ) Convention</th>
<th>Standard Convention</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case 1</td>
<td>Case 2</td>
<td>Case 1</td>
</tr>
<tr>
<td>0.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Support</td>
<td>-183</td>
<td>161</td>
<td>-183</td>
</tr>
<tr>
<td>0.1</td>
<td>-1538</td>
<td>1352</td>
<td>-1538</td>
</tr>
<tr>
<td>0.2</td>
<td>-3076</td>
<td>2705</td>
<td>-3076</td>
</tr>
<tr>
<td>0.3</td>
<td>-4614</td>
<td>4057</td>
<td>-4614</td>
</tr>
<tr>
<td>0.4</td>
<td>-6152</td>
<td>5409</td>
<td>-6152</td>
</tr>
<tr>
<td>0.5</td>
<td>-7691</td>
<td>6761</td>
<td>-7691</td>
</tr>
<tr>
<td>0.6</td>
<td>-9229</td>
<td>8114</td>
<td>-9229</td>
</tr>
<tr>
<td>0.7</td>
<td>-10767</td>
<td>9466</td>
<td>-10767</td>
</tr>
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\[\text{Standard Positive Convention}\]

Chapter 21 – Seismic Design of Concrete Bridges

21-76
### Table 21.3-4  
**Moment Demand Envelope**

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*Note: The table details specific values for moment demand envelopes under various conditions and locations.*
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, $M_{eq}^{(1)} = 0$ kip-ft

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

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

Span 1, Case 2

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

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

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

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

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

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

* 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

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Table 21.3-6 Earthquake Shear Forces from wFRAME Output

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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

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* Area of mild steel based on minimum seismic requirement only

(remaining limit state requirements need to be satisfied, \( A_{p_{sv}} = 56.6 \text{ in.}^2 \) at right face of Bent 2)
## Table 21.3-9 Section Flexural Capacity Calculation Data

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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

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21.3.21 Step 19 - Design Joint Shear Reinforcement

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

![Diagram of bent cap-to-column joint]

Figure 21.3-17 Bent Cap-to-Column Joint

Cap beam short stub length, \( S = 14.3 - 8.38 - 3 = 2.92 \text{ ft} < D_c = 6 \text{ ft} \) (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, \( l_{ac,\text{provided}} = 66 \) in.

From the \( x\)SECTION 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.

Calculate principal stresses, \( p_t \) and \( p_c \)

Vertical Shear Stress, \( v_{jv} \)
\[ T_c \approx 1.2 \times (2862) \text{ kips} = 3,434 \text{ kips} \]  
(Using \( x\)SECTION results of Appendix 21.3-7)
\[ A_{jv} = l_{ac} B_{cap} = (66)(96) = 6,336 \text{ in.}\(^2\) \]  
(\( SDC \) 7.4.4.1-4)
\[ v_{jv} = \frac{T_c}{A_{jv}} = \frac{3,434}{6336} = 0.542 \text{ ksi} \]  
(\( SDC \) 7.4.4.1-3)

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 \( f_h = (P_b / B_{cap}D_s) = 0 \).
\[
p_r = \frac{f_h + f_v}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_j^2}
\]
\[
= \frac{(0.00 + 0.168)}{2} - \sqrt{\left(\frac{0.00 - 0.168}{2}\right)^2 + 0.542^2}
\]
\[
= -0.464 \text{ ksi}
\]

(SDC 7.4.4.1-1)

\[
p_c = \frac{(0.00 + 0.168)}{2} + \sqrt{\left(\frac{0.00 - 0.168}{2}\right)^2 + 0.542^2}
\]
\[
= 0.632 \text{ ksi}
\]

(SDC 7.4.4.1-2)

Check Joint Size Adequacy

Principal compression, \( p_c = 0.632 \text{ ksi} \leq [0.25 f'_c = 0.25 (4.0) = 1 \text{ ksi}] \)

OK (SDC 7.4.2-1)

Principal tension, \( p_t = 0.464 \text{ ksi} \leq [12 \sqrt{f'_{ct}} = 12\sqrt{4000}/1000 = 0.76 \text{ ksi}] \)

OK (SDC 7.4.2-2)

Check the Need for Additional Joint Reinforcement

Since \( p_t = 0.464 \text{ ksi} > [3.5 \sqrt{f'_{ct}} = 3.5\sqrt{4000}/1000 = 0.221 \text{ ksi}] \), 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 \textit{wFRAME} push-over analysis results (see Appendix 21.3-6),
Column axial force (including the effect of overturning), \( P_c = 907 \text{ kips}^* \)
Column plastic moment, \( M_p = 12,636 \text{ kip-ft}^* \)

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

\[
T_c \approx 1.2 \text{ (3,148 kips) } = 3,778 \text{ kips using } \textit{xSECTION} \text{ results.}
\]

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

\[
\nu_{jv} = \frac{3,778}{6,336} = 0.596 \text{ ksi}
\]
Check Joint Size Adequacy

Principal compression, \( p_c = 0.628 \text{ ksi} < [0.25 \times 4.0 = 1 \text{ ksi}] \) OK

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

Check the Need for Additional Joint Reinforcement

Since \( p_t = 0.566 \text{ ksi} > [3.5\sqrt{4000}/1000= 0.221 \text{ ksi}] \), 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.
Joint Shear Requirement

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

$$A_{s}^{U-Bar}_{\text{required}} = 0.33 A_{st} = 0.33 (58.5) = 19.3 \text{ in.}^2 \quad (SDC 7.4.5.1-3)$$

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

- Top Reinforcement  
  #11, total 22 bars giving $A_{st} = 34.32 \text{ in.}^2$
- Bottom Reinforcement  
  #11, total 24 bars giving $A_{st} = 37.44 \text{ in.}^2$

$A_{s}^{U-Bar}_{\text{provided}} = 12 (1.56) = 18.72 \text{ in.}^2$ (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)](image)

- Vertical Stirrups in Joint Region

$$A_{s}^{Jv}_{\text{required}} = 0.2 A_{st} = 0.20 (58.5) = 11.7 \text{ in.}^2 \quad (SDC 7.4.5.1-4)$$

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

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

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

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

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

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

---

**Figure 21.3-20 Location of Vertical Stirrups, $A_{jv}^y$**

- **Horizontal Side Reinforcement**

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

  $A_{cap}^{\text{top}} = 34.32 \text{ in.}^2$

  $A_{cap}^{\text{bot}} = 37.44 \text{ in.}^2$
As shown in Figures 21.3-21 and 21.3-22, provide #6, total 5 (“U” shaped), giving:

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

- Horizontal Cap End Ties

\[ A_{jhc}^{jhc} = 0.33A_{s}^{jhc-bar} = 0.33(19.3) = 6.37 \text{ in.}^2 \quad (SDC 7.4.5.1-7) \]

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 \( A_{jhc}^{jhc} \)

- J-Dowels

Strictly following SDC guidelines, there is no need for J-Dowels for this bridge. Let us provide it anyway.

\[ A_{s}^{j-bar} = 0.08A_{s} = 0.08(58.5) = 4.68 \text{ in.}^2 \quad (SDC 7.4.5.1-8) \]

Use 16, #5 J-Dowels.

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

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

- Check Anchorage for Main Column Reinforcement

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

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

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

\[
T_c \approx 1.2(2,948) \text{ kips} = 3,538 \text{ kips using } x\text{SECTION results}
\]

\[
A_{jv} = l_c B_{cap} = 66 \times 96 = 6,336 \text{ in}^2
\]

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 \( f_h = (P_b/B_{cap}D_c) = 0 \).

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

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

Check Joint Size Adequacy

Principal compression, \( p_c = 0.618 \text{ ksi} \leq [0.25 \times 4.0 = 1 \text{ ksi}] \) OK

Principal tension, \( p_t = 0.503 \text{ ksi} \leq [\sqrt{12/4000/1000} = 0.760 \text{ ksi}] \) OK
Check the Need for Additional Joint Reinforcement

\[ p_t = 0.503 \text{ ksi} > \sqrt[3]{\frac{3.5}{4000}/1000} = 0.221 \text{ ksi} \], 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

\[ A_{jv}^{\text{required}} = 0.2 A_{st} = 0.2 (58.5) = 11.7 \text{ in.}^2 \]

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

\[ A_{jv}^{\text{provided}} = (2 \text{ legs})(14 \text{ sets})(0.44) = 12.32 \text{ in.}^2 > 11.7 \text{ in.}^2 \] \hspace{1cm} 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, \( N_A = 30 \text{ in.} \) \hspace{1cm} (SDC Section 7.8.3)

\[ N_A \geq \Delta_{phs} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_q + 4 \text{ (in.)} \] \hspace{1cm} (SDC 7.8.3-1)

The combined effect of \( \Delta_{phs} \), \( \Delta_{cr+sh} \), and \( \Delta_{temp} \), 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.,
\[ \Delta_{eq,\text{longitudinal}} = 14.12 + 16.87 \sin(20^\circ) = 19.89 \text{ in.} \]

\[ N_{A, \text{required}} \text{ (normal to centerline of bearing)} = (19.89 + 2.6) \cos(20^\circ) + 4 = 25.13 \text{ in.} < 30 \text{ in.} \]

Provide abutment seat width \( N_A = 36 \text{ in.} > 30 \text{ in.} \) 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)
- \( f_c' = 3.6 \text{ ksi} \)
- 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)

\[ V_{piles} = 14 (40) = 560 \text{ kips} \]

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 \( d_v = 0.72 (12 \text{ in.}) = 8.64 \text{ in.} \)

Effective width \( b_v = [6.75 + \frac{\sqrt{A}}{3}(7.25)](12) \text{ in.} = 110 \text{ in.} \)

\[ V_{ww} = 0.0316\beta \sqrt{f_c'b_vd_v} = 0.0316(2)\sqrt{3.6(110)(8.64)} = 114 \text{ kips} \]

Assuming \( \alpha = 0.75 \), \( F_{sk} = 0.75[0.75(560) + 114] = 401 \text{ kips} \)

We shall use the Isolated Shear Key Method for this example.

Vertical shear key reinforcement:

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

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

Hanger bars,

\[ A_{sh} = 2.0 A_{sk,\text{provided}} = 2 \times (3.52) = 7.04 \text{ in.}^2 \] \( (SDC 7.8.4.1B-1) \)

Provide 5 #11 hooked bars \( (A_{sk,\text{provided}} = 7.8 \text{ in.}^2 > 7.04 \text{ in.}^2) \) OK
Place the vertical shear key bars, $A_{vk}$ at least $L_{\text{min}}$ from the end of the lowest layer of the hanger bars, where

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

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

$$a = \text{(Bearing pad thickness + 6 in.)} = 11 \text{ in.}$$

Assuming 2 in cover and #4 distribution bars for the hanger bars,

$$b = 2 + 0.56 + 0.5(1.63) = 3.4 \text{ in.} \quad (\text{see SDC Figure 7.8.4.1-1A for definition of dimension “}b”)$$

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

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

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

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

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

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

04/17/2006, 11:45
************************************************************
*                                                          *
*  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                                   *
*                                                          *
*  Copyright (C) 1994, 1995, 1999 By Mark Seyed Mahan.    *
*                                                          *
*  A proper license must be obtained to use this software. *
*  For GOVERNMENT work call 916-227-8404, otherwise leave a *
*  message at 530-756-2367. 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.                                   *
*                                                          *
************************************************************

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
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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
Force Equilibrium Condition of the x-section:

Max. Strain
step  Conc. | Max. Neutral Steel | Max. Steel
          Axis | Strain Conc. | force | P/S | Net Curvature Moment
          in.  | Tens. Comp. | Tens. force | rad/in | (K-ft)
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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 -2566 0 | -1.69 0.000215 | 12926
28      0.00435 | 18.07 -0.0134 | 3388 1008 -2703 0 | -0.79 0.000269 | 13267
29      0.00532 | 18.11 -0.0148 | 3413 1037 -2753 0 | 0.59 0.000298 | 13362
30      0.00588 | 18.15 -0.0164 | 3461 1048 -2816 0 | -0.56 0.000330 | 13495
31      0.00650 | 18.21 -0.0183 | 3515 1060 -2881 0 | -0.42 0.000366 | 13660
32      0.00718 | 18.27 -0.0203 | 3570 1072 -2948 0 | -0.93 0.000406 | 13834
33      0.00794 | 18.30 -0.0225 | 3630 1087 -3021 0 | 1.38 0.000449 | 14017
34      0.00878 | 18.33 -0.0249 | 3686 1103 -3096 0 | -1.20 0.000497 | 14194
35      0.00971 | 18.34 -0.0275 | 3743 1122 -3171 0 | -0.61 0.000550 | 14368
36      0.01073 | 18.34 -0.0304 | 3792 1148 -3246 0 | 0.07 0.000608 | 14536
37      0.01186 | 18.34 -0.0336 | 3834 1181 -3321 0 | -0.67 0.000672 | 14695
38      0.01312 | 18.38 -0.0373 | 3874 1217 -3371 0 | -0.48 0.000745 | 14841
39      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) = 0.000075
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:

<table>
<thead>
<tr>
<th>Method</th>
<th>Conc.</th>
<th>Yield symbol Plastic</th>
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</thead>
<tbody>
<tr>
<td>ID</td>
<td>Strain</td>
<td>Curv. Moment</td>
</tr>
<tr>
<td>in/in</td>
<td>rad/in (K-ft)</td>
<td>rad/in (K-ft) moment rad/in</td>
</tr>
<tr>
<td>Strain @ 0.003</td>
<td>0.000155</td>
<td>12388 0.000070</td>
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<tr>
<td>Strain @ 0.004</td>
<td>0.000219</td>
<td>12957 0.000073</td>
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<tr>
<td>Strain @ 0.005</td>
<td>0.000279</td>
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<tr>
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<td>UCSD@5phy 0.00483</td>
<td>0.000270</td>
<td>13271 0.000075</td>
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</table>
APPENDIX 21.3-2 Moment – Curvature Relationship

Moment-Curv. Curve:
\[ M_{\text{max}} \text{ ft-k} = 14975.5 \]
\[ \phi_{\text{max}} = 0.0000825 \]

Rebar Yield info:
\[ M_y \text{ (k-ft)} = 9537.0 \]
\[ \phi_{y} = 0.000054 \]
\[ E_c \text{ (ksi)} = 1313 \]

Idealized values:
\[ M_p \text{ (k-ft)} = 13838.4 \]
\[ \phi_{p} = 0.0000747 \]
\[ I_{cr} \text{ (ft}^4) = 23.72 \]
APPENDIX 21.3-3 Soil Spring Data

![P-y Data - Bent 2 (Location 1)](image1)

![P-y Data - Bent 2 (Location 2)](image2)
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

Copyright (C) 1994, 1995, 1999 By Mark Seyed Mahan.

A proper license must be obtained to use this software.
For GOVERNMENT work call 916-227-8404, otherwise leave a
message at 530-756-2367. 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.

******************************************************************************
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:

<table>
<thead>
<tr>
<th>Strain</th>
<th>Yield symbol</th>
<th>Plastic Curv. Moment</th>
</tr>
</thead>
<tbody>
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<td>0.000520</td>
<td>21189</td>
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<td>0.004</td>
<td>0.000684</td>
<td>21635</td>
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<tr>
<td>0.005</td>
<td>0.000806</td>
<td>21635</td>
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<td>CALTRANS 0.00187</td>
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<tr>
<td>UCSD@5phy0.00126</td>
<td>0.000184</td>
<td>17426</td>
</tr>
</tbody>
</table>
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 *
* *
* Copyright (C) 1994, 1995, 1999 By Mark Seyed Mahan. *
* *
* A proper license must be obtained to use this software. *
* For GOVERNMENT work call 916-227-8404, otherwise leave a *
* message at 530-756-2367. 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. *
* *
*****************************************************************************

This output was generated by running:
 xSECTION
VER._2.40, MAR-14-99

Concrete Type Information:

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<thead>
<tr>
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<th>e0</th>
<th>e2</th>
<th>ecc</th>
<th>eu</th>
<th>f0</th>
<th>f2</th>
<th>fcc</th>
<th>fu</th>
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<th>W</th>
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<td>0.0020</td>
<td>0.0050</td>
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Steel Type Information:

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<th>eh</th>
<th>eu</th>
<th>fy</th>
<th>fu</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0023</td>
<td>0.0150</td>
<td>0.0900</td>
<td>68.00</td>
<td>95.00</td>
<td>29000</td>
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<tr>
<td>2</td>
<td>0.0023</td>
<td>0.0075</td>
<td>0.0600</td>
<td>68.00</td>
<td>95.00</td>
<td>29000</td>
</tr>
</tbody>
</table>

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) = 48.93B

Idealization of Moment-Curvature Curve by Various Methods:

<table>
<thead>
<tr>
<th>Points on Curve</th>
<th>Idealized Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method</td>
<td>Conc.</td>
</tr>
<tr>
<td>Strain @ 0.003</td>
<td>0.000593</td>
</tr>
<tr>
<td>Strain @ 0.004</td>
<td>0.000000</td>
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<td>Strain @ 0.005</td>
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<tr>
<td>CALTRANS</td>
<td>0.00159</td>
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<tr>
<td>UCSD@5phy</td>
<td>0.00117</td>
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</tbody>
</table>
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
<table>
<thead>
<tr>
<th>#</th>
<th>name</th>
<th>strength</th>
<th>displacement</th>
<th>rotation</th>
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<tbody>
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<td>PO1X01</td>
<td>136.37</td>
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</table>

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

### Element Information:

#### Element:

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### Lat. Force / Deflection:

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### Cumulative Results of analysis at end of stage:

- Plastic Action
- Lat. Force / Deflection

### GLOBAL:

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**Element Table**

**Table Legend**

- **name**: Reference number for the element.
- **node**: Node number for the element.
- **displ.x**: Displacement in the x-direction.
- **displ.y**: Displacement in the y-direction.
- **rotation**: Rotation angle.
- **axial**: Axial force.
- **shear**: Shear force.
- **moment**: Bending moment.

**Table Notes**

- **RIDGE**: Indicates an engineering term or concept.
- **D**: Indicates a design or dimensioning factor.

---

**Chapter 21 – Seismic Design of Concrete Bridges**

21-107
### Bridge Design Practice • February 2015

Cumulative Results of analysis at end of stage 1

Plastic Action at:

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Chapter 21 – Seismic Design of Concrete Bridges 21-108
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  DUCTILITY and STRENGTH of
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  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
  Copyright (C) 1994, 1995, 1999 By Mark Seyed Mahan.
  A proper license must be obtained to use this software.
  For GOVERNMENT work call 916-227-8404, otherwise leave a
  message at 530-756-2367. 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.

  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:
  Type e0 e2 ecc eu f0 f2 fcc fu E W
  1 0.0020 0.00 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:
  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 type xc yc area
  No. in in in^2
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  2 2 31.00 7.64 2.25
  ... 25 2 28.27 14.84 2.25
  26 2 31.00 7.64 2.25
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26  | 0.00356 | 15.28 | -0.0081 | 3813  | 991   | -2330 | 0     | 0.43  | 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  | 0.00522 | 16.23 | -0.0129 | 4008  | 1092  | -2623 | 0     | 1.90  | 0.000269 | 14242  |
31  | 0.00578 | 16.38 | -0.0144 | 4043  | 1106  | -2695 | 0     | -0.34 | 0.000300 | 14544  |
32  | 0.00650 | 16.52 | -0.0161 | 4089  | 1121  | -2765 | 0     | 1.91  | 0.000334 | 14706  |
33  | 0.00778 | 16.66 | -0.0180 | 4135  | 1137  | -2797 | 0     | 0.76  | 0.000372 | 14879  |
34  | 0.00794 | 16.77 | -0.0200 | 4180  | 1156  | -2862 | 0     | 0.39  | 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:

<table>
<thead>
<tr>
<th>Points on Curve</th>
<th>Idealized Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method</td>
<td>Conc.</td>
</tr>
<tr>
<td>ID</td>
<td>Strain</td>
</tr>
<tr>
<td>Strain @ 0.003</td>
<td>0.000138</td>
</tr>
<tr>
<td>Strain @ 0.004</td>
<td>0.000198</td>
</tr>
<tr>
<td>Strain @ 0.005</td>
<td>0.000253</td>
</tr>
<tr>
<td>UCSD@5phy0.00558</td>
<td>0.000283</td>
</tr>
</tbody>
</table>
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
* VER._2.40,_MAR-14-99
* Copyright (C) 1994, 1995, 1999 By Mark Seyed Mahan.
* A proper license must be obtained to use this software.
* For GOVERNMENT work call 916-227-8404, otherwise leave a
* message at 530-756-2367. 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.
******************************************************************************

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 m2 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

Chapter 21 – Seismic Design of Concrete Bridges

21-112
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:

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<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>step epscmax</td>
<td>in.</td>
<td>Tens. Comp.</td>
<td>Comp. Tens.</td>
<td>force</td>
<td>rad/in</td>
</tr>
<tr>
<td>0</td>
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<td>0.00000</td>
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<td>1</td>
<td>0.00029</td>
<td>2.87</td>
<td>-0.0003</td>
<td>949</td>
<td>131</td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th>Points on Curve</th>
<th>Idealized Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method</td>
<td>Conc. Yield symbol Plastic</td>
</tr>
<tr>
<td>Strain @ 0.003</td>
<td>0.000000 0.01154</td>
</tr>
<tr>
<td>Strain @ 0.004</td>
<td>0.0000248 11800 0.000074</td>
</tr>
<tr>
<td>Strain @ 0.005</td>
<td>0.0000313 12168 0.000076</td>
</tr>
<tr>
<td>CALTRANS 0.00673</td>
<td>0.0000421 12636 0.000079</td>
</tr>
<tr>
<td>UCSD@5phy 0.00412</td>
<td>0.0000256 11855 0.000074</td>
</tr>
</tbody>
</table>
APPENDIX 21.3-9 \textit{wFRAME}, Output File

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

******************************************************************************  
*  
* \textit{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

<table>
<thead>
<tr>
<th>node</th>
<th>name</th>
<th>coordinates</th>
<th>--------fixity--------</th>
<th>X</th>
<th>Y</th>
<th>X-dir.</th>
<th>Y-dir.</th>
<th>Rotation</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0.00</td>
<td>r</td>
<td>r</td>
<td>r</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>S01.01</td>
<td>4.72</td>
<td>r</td>
<td>r</td>
<td>r</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>S01.02</td>
<td>7.72</td>
<td>r</td>
<td>r</td>
<td>r</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>4</td>
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<td>7.72</td>
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<td>r</td>
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<td></td>
</tr>
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<td>7.72</td>
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<td></td>
<td></td>
</tr>
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<td></td>
<td></td>
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<td>8</td>
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<td>-41.22</td>
<td>s 1.4e+002</td>
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<td>41.72</td>
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<td>r</td>
<td>r</td>
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<tr>
<td>22</td>
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<td>s 1.4e+002</td>
<td>r</td>
<td></td>
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</tr>
<tr>
<td>23</td>
<td>P02.02</td>
<td>41.72</td>
<td>-43.27</td>
<td>s 4.1e+002</td>
<td>r</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>41.72</td>
<td>-45.32</td>
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<td>r</td>
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<td>f 0.0000</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Spring Information at node points:

\[ k's = k/ft \text{ or } ft-k/rad.; \quad d's = ft \text{ or } rad. \]
Cumulative Results of analysis at end of stage 0

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

Element Information:

<table>
<thead>
<tr>
<th>#</th>
<th>name</th>
<th>depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S01-01</td>
<td>7.00 6.8 62.6 629528 62953</td>
</tr>
<tr>
<td>2</td>
<td>S01-02</td>
<td>7.00 6.8 62.6 629528 62953</td>
</tr>
<tr>
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<td>C01-01</td>
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</tr>
<tr>
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<td>7.00 6.8 62.6 629528 62953</td>
</tr>
<tr>
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<td>7.00 6.8 62.6 629528 62953</td>
</tr>
<tr>
<td>6</td>
<td>C01-04</td>
<td>7.00 6.8 62.6 629528 62953</td>
</tr>
</tbody>
</table>

Displacement:

- Displ.x  Displ.y  Rotation

- Lat. Force / Deflection

- Element/ Stage/ Code/ *g (DL= 3381.7) / (in)

- Element node  --- local --- element ---

- Element: node

- Displ.x  displ.y  rotation  axial  shear  moment

- Chapter 21 – Seismic Design of Concrete Bridges

Seismic Design of Concrete Bridges

<table>
<thead>
<tr>
<th>#</th>
<th>name</th>
<th>dir</th>
<th>depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S01-01</td>
<td>7.00 6.8 62.6 629528 62953</td>
<td>52.25 -68.40 29928 29928 0.02 e</td>
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<td>43.00 0.00 29928 29928 0.02 e</td>
</tr>
<tr>
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<td>C01-04</td>
<td>7.00 6.8 62.6 629528 62953</td>
<td>21.50 0.00 12636 12636 0.02 e</td>
</tr>
</tbody>
</table>

- Element: node

- Displ.x  displ.y  rotation  axial  shear  moment

- Chapter 21 – Seismic Design of Concrete Bridges

Seismic Design of Concrete Bridges
<table>
<thead>
<tr>
<th>Element/ Stage/ Code/ *g (DL=3381.7)</th>
<th>Lat. Force</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>S02</td>
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<tr>
<td>S03</td>
<td>0.00000</td>
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</tr>
<tr>
<td>P01</td>
<td>0.00000</td>
<td>0.00000</td>
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</tbody>
</table>

Cumulative Results of analysis at end of stage 1

Plastic Action at:

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<tr>
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<th>node#</th>
<th>name</th>
<th>displ.x</th>
<th>displ.y</th>
<th>rotation</th>
<th>axial</th>
<th>shear</th>
<th>moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>S02</td>
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<td>C01</td>
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<td>0.00141</td>
<td>0.00000</td>
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<td>761.93</td>
</tr>
<tr>
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<td>02</td>
<td>C01</td>
<td>-0.00606</td>
<td>-0.00012</td>
<td>0.00141</td>
<td>0.00000</td>
<td>322.85</td>
<td>761.93</td>
</tr>
<tr>
<td>P01</td>
<td>03</td>
<td>C01</td>
<td>-0.00606</td>
<td>-0.00012</td>
<td>0.00141</td>
<td>0.00000</td>
<td>322.85</td>
<td>761.93</td>
</tr>
<tr>
<td>P01</td>
<td>04</td>
<td>C01</td>
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<td>-0.00012</td>
<td>0.00141</td>
<td>0.00000</td>
<td>322.85</td>
<td>761.93</td>
</tr>
</tbody>
</table>

Chapter 21 – Seismic Design of Concrete Bridges
12 S02-02/02/02  12 0.73219 -0.01192 -0.00274 -78.08 146.67 -10141.02
13 S02-03/03 13 0.73220 -0.02352 -0.00059 78.08 332.13 9491.90
14 S02-04/04 14 0.73220 -0.02138 0.00106 -6.47 810.93 5491.19
15 S02-05/05 15 0.73218 -0.01148 0.00151 -91.08 1289.73 1861.15
16 S02-06/06 16 0.73215 -0.00478 0.00003 -175.49 1768.53 12565.08
17 S02-01/01 17 0.73213 -0.00665 -0.00137 -2501.90 348.38 16141.41
18 S02-02/02 18 0.00618 0.72477 -0.00300 -2501.84 348.05 14964.00
19 S02-03/03 19 0.00450 0.62890 -0.01255 -2501.84 -348.05 -10811.87
20 S02-04/04 20 0.00283 0.43749 -0.01903 -2501.84 -348.04 -6659.75
21 S02-01/01 21 0.00115 0.18720 -0.00224 -2501.84 -348.03 -2507.76
22 S02-02/02 22 0.00086 0.14093 -0.00270 -2501.84 -347.72 -1795.38
23 S02-03/03 23 0.00058 0.09420 -0.00288 -2501.84 -348.53 -1121.56
24 S02-04/04 24 0.00029 0.04717 -0.00299 -2501.84 -289.56 -528.43
25 S03-01/01 25 0.73213 -0.00665 -0.00137 -74.72 528.17 2038.64
26 S03-02/02 26 0.73214 -0.01097 -0.00149 -28.84 322.85 761.91
27 S03-02/02 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:

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<thead>
<tr>
<th>Element/ Stage/ Code/ *g (DL= 3381.7)</th>
<th>Lat. Force / Deflection</th>
</tr>
</thead>
<tbody>
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</tr>
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<tr>
<td>P01X01</td>
<td>3 2</td>
</tr>
<tr>
<td>P02X02</td>
<td>4 2</td>
</tr>
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Chapter 21 – Seismic Design of Concrete Bridges

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Chapter 21 – Seismic Design of Concrete Bridges 21-118
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

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

Note: Specific instructions are included as footnotes.

EA  DISTRICT  COUNTY  ROUTE  PM (KP)  BRIDGE NAME AND NUMBER
F/0076  59  ES  999  99  Prototype Bridge

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

1) TEMPERATURE EXTREMES [from Preliminary Report]

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ITEM(1) DESIGNER:  DATE  ITEM(2) CHECKED BY:  DATE

DESIGNER:  CHECKER:  To be filled in by Office of Structures Design

To be filled in by SR:  Date:

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<td>(3X4/100) to 1/2&quot; or Open Joint</td>
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Anticipated Shortening = \( \frac{1.26}{100} \times \left( \frac{202 + 210}{2} \right) = 2.60 \text{ in.} \)
APPENDIX 21.3-12 \textit{wFRAME} Longitudinal Push Over – Force/Displacement Relationship, Right Push
APPENDIX 21.3-13 *wFRAME* Longitudinal Push Over – Force vs. Displacement Relationship

```
*wFRAME*
VER_3.06_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 (\(\times g\)) = 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
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6  C01.03  7.72 -27.24     r         r         r
7  C01.04  7.72 -39.17     r         r         r
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9  P01.02  7.72 -43.27     s 4.1e+002  r         r
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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
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21 C02.04 41.72 -39.17     r         r         r

Cumulative Results of analysis at end of stage  6

Plastic Action at:

Element/ Stage/ Code/ Lat. Force / Deflection
              g (DL= 3381.7) / (in)
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P01X02  2  2   0.1958   9.7870
P02X02  3  2   0.1966   9.8292
P01X02  4  2   0.2059  10.3036
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APPENDIX 21.3-15 \textit{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)

******************************
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* 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:
- \textit{s}=spring and value
- \textit{r}=complete release
- \textit{f}=complete fixity with imposed displacement

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**Chapter 21 – Seismic Design of Concrete Bridges**

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**Seismic Design of Concrete Bridges**

Chapter 21 – Seismic Design of Concrete Bridges

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**Note:** Year 1 data is not available for this route.
APPENDIX 21.3-16 PSSECx Input File

PSSEC300, OCT_26_2005
Bridge Design Academy - Prototype Superestructure 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 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 compressive parameters
type#: Ef;fy;strain hard. factor;fu;ult. strain;PS-code: 0 tendons, 1 otherwise
  Ef;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 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)
1

Analysis Control: p- Positive moment, n- Negative moment
APPENDIX 21.3-17 PSSECx Model for Superstructure

PSSEC300, OCT 31, 2005
Bridge Design Academy, Evanston
Superstructure Capacity SI
LINEG
X Sec. Geometry and Rebar
Negative Moment Analysis
Comp. & Bottom Fibers
Axial Force = 0.8
Dimension Chart Units:
Min. X
= 3.510 in.
Max. X
= 33.00 in.
Min. Y
= 45.55 in.
Max. Y
= 35.03 in.
APPENDIX 21.3-18 Partial Output from PSSECx Run

******** SECx ********

05-15-2006

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 Sustructure 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

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<th>Ultimate Stress (ksi)</th>
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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

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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

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<td>34.62</td>
<td>292.83</td>
</tr>
<tr>
<td>200</td>
<td>1.0</td>
<td>0.00</td>
<td>35.03</td>
<td>292.83</td>
</tr>
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</table>
### Prestressing Steel Fiber Data

<table>
<thead>
<tr>
<th>Fiber Material</th>
<th>x</th>
<th>y</th>
<th>area</th>
<th>P/S force</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No.</td>
<td>No.</td>
<td>(in)</td>
<td>(in^2)</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>0.00</td>
<td>25.44</td>
<td>38.28</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Fiber Material</th>
<th>x</th>
<th>y</th>
<th>area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No.</td>
<td>No.</td>
<td>(in)</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>0.00</td>
<td>31.80</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>0.00</td>
<td>-42.13</td>
</tr>
</tbody>
</table>

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 0.0017950

Undefomed 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:

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Step</td>
<td>Conc.</td>
<td>Neutral Steel</td>
<td>Conc.</td>
</tr>
<tr>
<td>Strain</td>
<td>Axis</td>
<td>Strain</td>
<td>Conc.</td>
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<tr>
<td>epsecmax</td>
<td>in.</td>
<td>Tens.</td>
<td>Comp.</td>
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<tr>
<td>0</td>
<td>-0.00001</td>
<td>-41.50</td>
<td>-0.00000</td>
</tr>
<tr>
<td>1</td>
<td>-0.00001</td>
<td>-42.26</td>
<td>0.00000</td>
</tr>
<tr>
<td>2</td>
<td>-0.00001</td>
<td>-43.05</td>
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<tr>
<td>3</td>
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<td>-43.86</td>
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<tr>
<td>4</td>
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<td>-44.70</td>
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<tr>
<td>5</td>
<td>0.000000</td>
<td>-45.56</td>
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</tr>
<tr>
<td>6</td>
<td>0.000100</td>
<td>9055.25</td>
<td>0.00000</td>
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<tr>
<td>7</td>
<td>0.000111</td>
<td>362.50</td>
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<tr>
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<tr>
<td>10</td>
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<td>78.67</td>
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<tr>
<td>11</td>
<td>0.000118</td>
<td>59.45</td>
<td>0.00000</td>
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<tr>
<td>12</td>
<td>0.000200</td>
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<tr>
<td>13</td>
<td>0.000222</td>
<td>37.74</td>
<td>0.00000</td>
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<tr>
<td>14</td>
<td>0.000225</td>
<td>29.97</td>
<td>0.00000</td>
</tr>
<tr>
<td>15</td>
<td>0.000228</td>
<td>14.77</td>
<td>0.00000</td>
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<tr>
<td>16</td>
<td>0.00032</td>
<td>2.23</td>
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<tr>
<td>18</td>
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<td>19</td>
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<tr>
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<td>-20.61</td>
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<tr>
<td>21</td>
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<td>0.000171</td>
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<tr>
<td>23</td>
<td>0.00071</td>
<td>-26.11</td>
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<td>24</td>
<td>0.00079</td>
<td>-27.79</td>
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<tr>
<td>25</td>
<td>0.00089</td>
<td>-30.09</td>
<td>0.000356</td>
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<tr>
<td>26</td>
<td>0.00100</td>
<td>-32.67</td>
<td>0.000499</td>
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<tr>
<td>27</td>
<td>0.00112</td>
<td>-34.65</td>
<td>0.000682</td>
</tr>
<tr>
<td>28</td>
<td>0.00126</td>
<td>-36.06</td>
<td>0.000979</td>
</tr>
</tbody>
</table>
### Prestress Tendon Strain on the x-section:

Max. Conc. Neutral P/S Steel 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 .00000 -.45.56 1 -.005648
6 .000010 .9055.25 1 -.005701
7 .000011 .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.

<table>
<thead>
<tr>
<th>Curvature(rad/in)</th>
<th>Moments (ft-k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield</td>
<td>0.000040</td>
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<tr>
<td>Nominal</td>
<td>67871</td>
</tr>
<tr>
<td>See forc equilibrium table at concrete strain of .003</td>
<td></td>
</tr>
<tr>
<td>Ultimate</td>
<td>0.000000</td>
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</tbody>
</table>

End
NOTATION

\[ \text{AASHTO} = \text{AASHTO LRFD Bridge Design Specifications with Interims and California Amendments} \]

\[ A_b = \text{area of individual reinforcing steel bar (in.}^2) \]

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

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

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

\[ A_e = \text{effective shear area; effective abutment wall area (in.}^2) \]

\[ A_g = \text{gross cross section area (in.}^2) \]

\[ A_{jh} = \text{effective horizontal area of a moment resisting joint (in.}^2) \]

\[ A_{jv} = \text{effective vertical area for a moment resisting joint (in.}^2) \]

\[ A_{ps} = \text{prestressing steel area (in.}^2) \]

\[ A_s = \text{area of supplemental non-prestressed tension reinforcement (in.}^2) \]

\[ A_{jh}^{\text{bar}} = \text{area of horizontal joint shear reinforcement required at moment resisting joints (in.}^2) \]

\[ A_{jhc} = \text{total area of horizontal ties placed at the end of the bent cap in Case 1 Knee joints (in.}^2) \]

\[ A_{jv}^{\text{bar}} = \text{area of vertical joint shear reinforcement required at moment resisting joints (in.}^2) \]

\[ A_{j-h}^{\text{bar}} = \text{area of vertical “J” bar reinforcement required at moment resisting joints with a skew angle > 20° (in.}^2) \]

\[ A_{sf} = \text{area of bent cap side face steel required at moment resisting joints (in.}^2) \]

\[ A_{sk} = \text{area of interface shear reinforcement crossing the shear plane (Vertical shear key reinforcement) (in.}^2) \]

\[ A_{st,\text{max}} = \text{maximum longitudinal reinforcement area (in.}^2) \]

\[ A_{st,\text{min}} = \text{minimum longitudinal reinforcement area (in.}^2) \]

\[ A_{st} = \text{total area of column longitudinal reinforcement anchored in the joint; total area of column/pier wall longitudinal reinforcement (in.}^2) \]
B_ridge Design Practice • February 2015

\[ A_{\text{bar}} = \text{area of bent cap top and bottom reinforcement bent in the form of “U” bars in Knee joints (in.}^2) \]

\[ A_{\text{sh}} = \text{area of horizontal shear key reinforcement (hanger bars) (in.}^2) \]

\[ A_{\text{sk(provided)}} = \text{area of interface shear reinforcement provided for isolated shear key (in.}^2) \]

\[ A_{\text{Non-iso sk(provided)}} = \text{area of interface shear reinforcement provided for non-isolated shear key (in.}^2) \]

\[ A_e = \text{area of shear reinforcement perpendicular to flexural tension reinforcement (in.}^2) \]

\[ B_{\text{cap}} = \text{bent cap width (in.)} \]

\[ BDD = \text{Caltrans Bridge Design Details} \]

\[ B_{\text{eff}} = \text{effective width of the superstructure for resisting longitudinal seismic moments (in.)} \]

\[ D_c = \text{column cross sectional dimension in the direction of interest (in.)} \]

\[ D_{\text{fg}} = \text{depth of footing (in.)} \]

\[ D_s = \text{depth of superstructure at the bent cap (in.)} \]

\[ DSH = \text{Design Seismic Hazards} \]

\[ D' = \text{cross-sectional dimension of confined concrete core measured between the centerline of the peripheral hoop or spiral (in.)} \]

\[ D'_c = \text{confined column cross-section dimension, measured out to out of ties, in the direction parallel to the axis of bending (in.)} \]

\[ E_c = \text{modulus of elasticity of concrete (ksi)} \]

\[ EDA = \text{Elastic Dynamic Analysis} \]

\[ ESA = \text{Elastic Static Analysis} \]

\[ F_{sk} = \text{abutment shear key force capacity; Shear force associated with column overstrength moment, including overturning effects (ksi)} \]

\[ I_{\text{eff}}, I_e = \text{effective moment of inertia for computing member stiffness (in.}^4) \]

\[ ISA = \text{Inelastic Static Analysis} \]

\[ K_{\text{abut}} = \text{abutment backwall stiffness (kip/in./ft)} \]

\[ K_{\text{eff}} = \text{effective abutment backwall stiffness (kip/in./ft)} \]

\[ K_i = \text{Initial abutment backwall stiffness (kip/in./ft)} \]
\( L \) = member length from the point of maximum moment to the point of contra-
flexure (in); length of bridge deck between adjacent expansion joints

\( L_{\text{min, headed}} \) = minimum horizontal length from the end of the lowest layer of headed
hanger bar to the intersection with the shear key vertical reinforcement (in.)

\( L_{\text{min, hooked}} \) = 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)

\( M_{eq} \) = column moment when coupled with any existing \( M_{dl} \) & \( M_{ps} \) will equal the
column’s overstrength moment capacity, \( M_{o,col} \) (kip-ft)

\( M_{eq}^{RL} \) = 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)

\( M_{supRL}^{ne} \) = 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 \( M-\phi \) analysis
(kip-ft)

\( M_{ps} \) = moment attributed to secondary prestress effects (kip-ft)

\( M_y \) = 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)
\[ P_{\text{dia}} = \text{passive pressure force resisting movement at diaphragm abutment (ksf)} \]
\[ P_{\text{dl}} = \text{superstructure dead load reaction at the abutment plus weight of the abutment and its footing (kip)} \]
\[ P_{\text{sup}} = \text{superstructure axial load resultant at the abutment (kip)} \]
\[ P/S = \text{prestressed concrete; prestressing strand} \]
\[ P_{\text{jack}} = \text{total prestress jacking force (kip)} \]
\[ P_{n} = \text{nominal axial resistance (kip)} \]
\[ P_{bw} = \text{passive pressure force resisting movement at seat abutment (ksf)} \]
\[ R_{A} = \text{abutment displacement coefficient} \]
\[ S = \text{cap beam short stub length (ft)} \]
\[ SDC = \text{Seismic Design Criteria} \]
\[ T = \text{natural period of vibration, (seconds), } T = 2\pi\sqrt{m/k} \]
\[ T_{c} = \text{total tensile force in column longitudinal reinforcement associated with } M_{o,\text{col}} \text{ (kip)} \]
\[ T_{i} = \text{natural period of the stiffer frame (sec.)} \]
\[ T_{j} = \text{natural period of the more flexible frame (sec.)} \]
\[ V_{c} = \text{nominal shear strength provided by concrete (kip)} \]
\[ V_{n} = \text{nominal shear strength (kip)} \]
\[ V_{o} = \text{overstrength shear associated with the overstrength moment } M_{o} \text{ (kip)} \]
\[ V_{o,\text{col}} = \text{column overstrength shear, typically defined as } M_{o,\text{col}}/L \text{ (kip)} \]
\[ V_{p} = \text{column plastic shear, typically defined as } M_{p,\text{col}}/L \text{ (kip)} \]
\[ V_{s} = \text{nominal shear strength provided by shear reinforcement (kip)} \]
\[ V_{ww} = \text{shear capacity of one wingwall (kip)} \]
\[ a = \text{demand spectral acceleration} \]
\[ b_{v} = \text{effective web width taken as the minimum web width within the shear depth } d_{v} \text{ (in.)} \]
\[ d_{bl} = \text{nominal bar diameter of longitudinal column reinforcement (in.)} \]
\( d_v \) = effective shear depth defined as the distance between resultants of tensile and compressive forces due to flexural, but need not be taken less than 0.9\( d_e \) or 0.72\( h \) (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)

\( f_{sh} \) = 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)

\( f_{1s}, f_{2s} \) = concrete shear factors for ductile members

\( g \) = acceleration due to gravity, 32.2 \( \text{ft/}^2 \)

\( h \) = 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.)

\( k_i', k_j' \) = 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)
\[ p_t = \text{nominal principal tension stress in a joint (psi)} \]
\[ s = \text{spacing of shear/transverse reinforcement (in.)} \]
\[ t = \text{top or bottom slab thickness (in.)} \]
\[ v_{vp} = \text{nominal vertical shear stress in a moment resisting joint (psi)} \]
\[ v_c = \text{permissible shear stress carried by concrete (psi)} \]
\[ w = \text{width of the backwall or diaphragm, as appropriate (in.)} \]
\[ \alpha = \text{factor defining the range over which } F_{sk} \text{ is allowed to vary} \]
\[ \beta = \text{factor indicating ability of diagonally cracked concrete to transmit tension and shear} \]
\[ e_{yu} = \text{reduced ultimate tensile strain for A706 reinforcement} \]
\[ \Delta_e = \text{local member displacement capacity (in.)} \]
\[ \Delta_{el} = \text{displacement attributed to the elastic and plastic deformation of the column (in.)} \]
\[ \Delta_c = \text{global displacement capacity (in.)} \]
\[ \Delta_{cr+sh} = \text{displacement due to creep and shrinkage (in.)} \]
\[ \Delta_t = \text{local member displacement demand (in.)} \]
\[ \Delta_D = \text{global system displacement (in.)} \]
\[ \Delta_{eff} = \text{effective longitudinal abutment displacement at idealized yield (in.)} \]
\[ \Delta_{eq} = \text{relative longitudinal displacement demand at an expansion joint due to earthquake (in.)} \]
\[ \Delta_p = \text{idealized plastic displacement capacity due to rotation of the plastic hinge (in.)} \]
\[ \Delta_{ps} = \text{displacement due to prestress shortening (in.)} \]
\[ \Delta_r = \text{relative lateral offset between the point of contra-flexure and the base of the plastic hinge (in.)} \]
\[ \Delta_{tem} = \text{displacement due to temperature variation (in.)} \]
\[ \Delta_Y = \text{idealized yield displacement of the subsystem at the formation of the plastic hinge (in.)} \]
\[ \Delta_{Y(i)} = \text{idealized yield displacement of the subsystem at the formation of plastic hinge (i) (in.)} \]
$\Delta_{y}^{\text{col}}$ = idealized yield displacement of a column at the formation of the plastic hinge (in.)

$\theta$ = angle of inclination of diagonal compressive stresses (radians)

$\theta_p$ = plastic rotation capacity (radians)

$\theta_{sk}$ = skew angle (degree)

$\rho_s$ = amount of transverse reinforcement expressed as volumetric ratio

$\phi$ = resistance factor

$\phi_p$ = idealized plastic curvature (1/in.)

$\phi_u$ = ultimate curvature capacity (1/in.)

$\phi_y$ = yield curvature corresponding to the first yield of the reinforcement in a ductile component (1/in.)

$\phi_T$ = idealized yield curvature (1/in.)

$\mu_d$ = local displacement ductility demand

$\mu_D$ = global displacement ductility demand

$\mu_c$ = local displacement ductility capacity
REFERENCES


