AASHTO T-3 TRIAL DESIGN BRIDGE DESCRIPTION

State: *California*

Trial Design Designation: *CA-1 (Caltrans Bridge Academy Example B)*

Bridge Name: *Typical California Bridge used by the Caltrans Bridge Academy*

Superstructure Type: *Continuous prestressed reinforced concrete box girder*

Span Length(s): *Three span 126ft.-168ft.-118ft.*

Substructure Type: *Two 6.ft. dia. reinforced concrete columns per bent*

Foundation: *Piles*

Abutments: *Seat type supported on piles*

Seismic Design Category (SDC): *D*

Seismic Design Strategy (Type 1, 2 or 3): *Type 1*

Design Spectral Acceleration at 1-second Period (S_{D1}): *0.97g*

Additional Description (Optional): *This bridge was originally designed in accordance with Caltrans Seismic Design Criteria (SDC) prior to doing the trial design in accordance with the NCHRP 20-07/193 Guidelines.*

LRFD Design Example B
(November 3, 2006 – Version 1.1)

Based on "Recommended LRFD Guidelines for the Seismic Design of Highway Bridges" (May 2006 Edition)

Bridge Design Academy Prototype Bridge

LRFD Design Example B
November 3, 2006 – Version 1.1

I. INTRODUCTION

I.A. Background/Problem Statement

This prototype bridge is used by various groups teaching the Caltrans Design Academy to illustrate the principles of bridge design including those of seismic design. This bridge will be designed using the proposed "Recommended LRFD Guidelines for the Seismic Design of Highway Bridges", drafted May 2006. Significant differences from Caltrans Seismic Design Criteria (SDC) version 1.4 will be noted where applicable.

This is a three span Prestressed Reinforced Concrete Box Girder bridge. 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 deg. The columns are pinned at the bottom. Figure 1 shows the General Plan for this structure.

The bridge is not considered to be a critical or essential bridge. Therefore, for design considerations, the prototype bridge is classified as a normal bridge. This bridge also meets the requirements of LRFD Section 3.1; consequently, the recommended guidelines set forth in the proposed LRFD code are applicable to this design.

I.B. Bridge Site Conditions and Design Requirements

This hypothetical structure crosses a roadway and railroad tracks. Because of poor soil conditions, the footing is supported on piles. The ground motion used for the Caltrans SDC design was based on the following assumptions:

Soil Profile:	Type C
Magnitude:	8.0 ± 0.25
Peak Rock Acceleration:	0.5g
Latitude and Longitude:	37.8800°, -122.522000°

The corresponding LRFD design response spectra is constructed using the procedures given in LRFD Section 3.4.1 and 3.4.2. The Type C soil profile is equivalent to Site Class C rock and values of S_s and S_1 obtained from NEHRP 2000 national ground motion maps at the location given above for Site Class B, are modified by F_a and F_v, respectively, as determined from LRFD Tables 3.4.2.3-1 and 2. Figure 2a shows the constructed design response spectra curve based upon 5% damping and 5% probability of exceedance in 50 years.

For comparison, Figure 2b shows the Caltrans ARS curve based upon 5% damping as taken from Appendix B of the SDC. Note the LRFD design spectra yields higher spectral accelerations for a given period as compared to the Caltrans ARS curve. Consequently, this will lead to higher design displacement demands subjected to the bridge.

The one-second period design spectral acceleration for the Life Safety Design Earthquake, S_{D1}, as obtained from Figure 2 is 0.97g. Since $S_{D1} > 0.50g$, according to

LRFD Table 3.5.1, this prototype bridge shall be designed for **Seismic Design Category (SDC) D**, which includes the following requirements:

- Identification of ERS
- Demand Analysis
- Displacement Capacity using Pushover Analysis
- Capacity Design including column shear
- SDC D Level of Detailing

The prototype bridge shall be designed such that plastic hinges will form in inspectable locations and the columns will provide all the resistance to the seismic motions. According to LRFD Section 3.3, these primary earthquake resisting elements (ERE) are categorized as permissible and consequently, the earthquake resisting system (ERS) is also permissible.

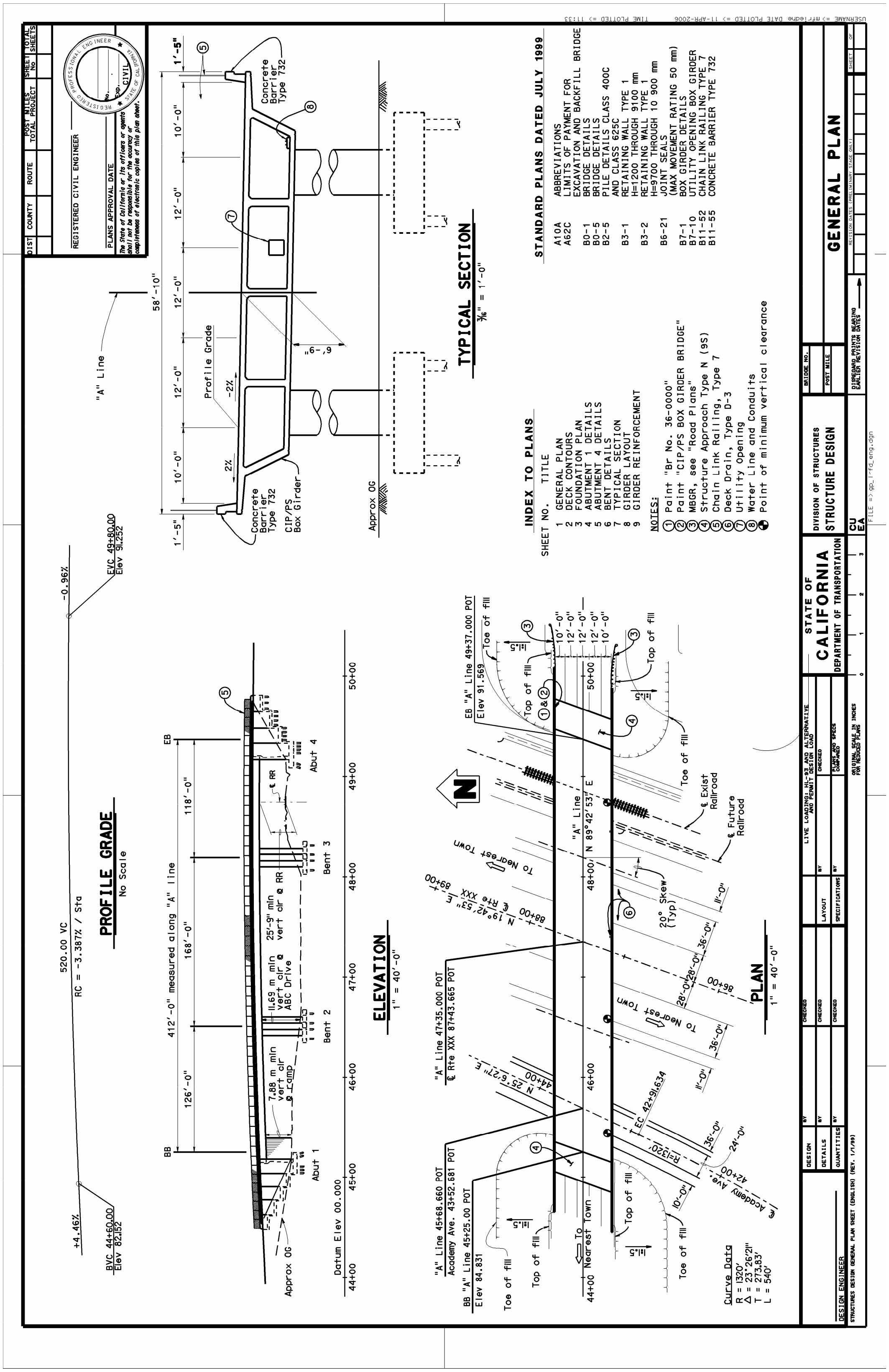
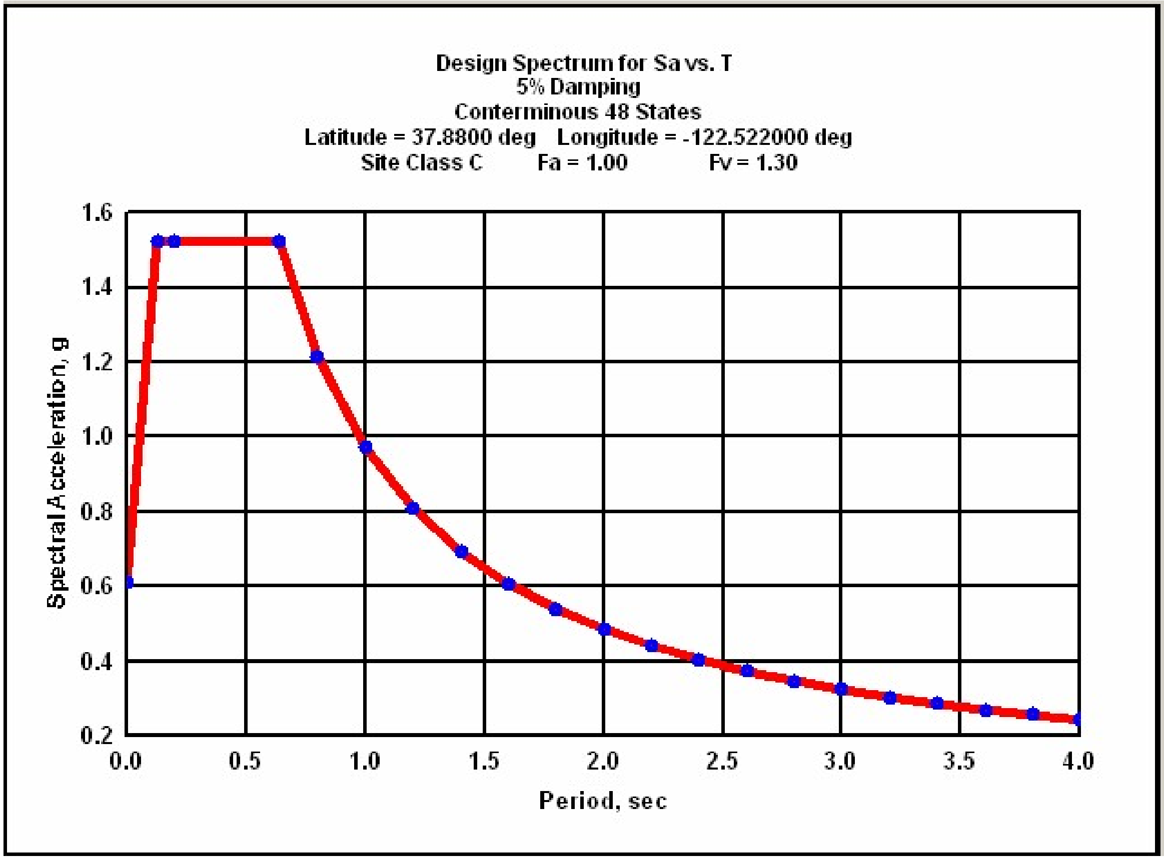


Figure 1 General Plan (Bridge Design Academy Prototype Bridge)



Graph Data	
Period, sec	Sa, g
0.00	0.6091
0.13	1.5227
0.20	1.5227
0.64	1.5227
0.80	1.2151
1.00	0.9721
1.20	0.8101
1.40	0.6944
1.60	0.6076
1.80	0.5401
2.00	0.4860
2.20	0.4419
2.40	0.4050
2.60	0.3739
2.80	0.3472
3.00	0.3240
3.20	0.3038
3.40	0.2859
3.60	0.2700
3.80	0.2558
4.00	0.2430

Figure 2a Design Response Spectrum, Construction Using Two-Point Method (See LRFD Figure 3.4.1-1)

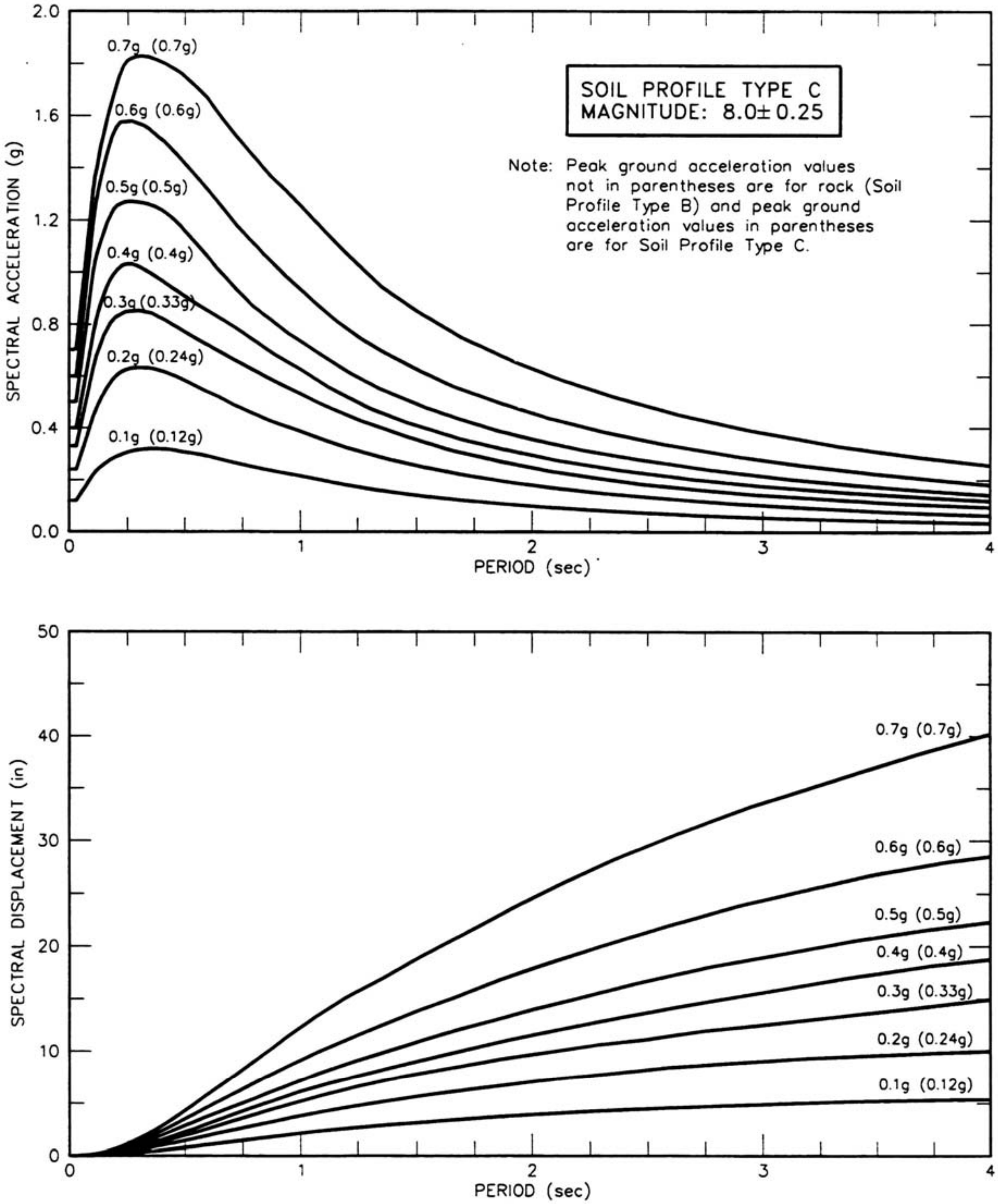


Figure 2b Caltrans ARS Curves For Soil Profile C (M = 8.0±0.25)
(See Caltrans SDC Figure B.6)

II. ANALYSIS AND DESIGN PROCEDURE

II.A. Preliminary Member and Span Configuration Determination

Bridge design is inherently an iterative process. It is common practice to design bridges for service loading and then, if necessary, to refine the design of various components to satisfy seismic performance requirements. In reality however, one needs to keep certain seismic requirements in mind even during a service design. This is especially true while selecting the span configuration, column size, column reinforcement requirements, and bent cap width.

Sizing the Column and Bent Cap

- Column Size

There are no guidelines in the LRFD specifiying minimum or maximum column sizes. As a starting point, we will use criteria provided in Caltrans SDC Section 7.6.1 and select a column size that satisfies the following criterion:

$$0.70 < \frac{D_c}{D_s} < 1.00 \qquad \text{SDC Eqn. (7.24)}$$

Where

D_c = Column diameter
D_s = Superstructure depth

If $D_c > D_s$, it may be difficult to meet the joint shear, superstructure capacity, and ductility requirements.

Given $D_s = 6.75$ ft from the service design, we select a column with $D_c = 6.00$ ft so that $D_c / D_s = 0.89$.

Max. Longitudinal Reinforcement Area, $A_{s\,max} \le 0.04 \times A_g$ LRFD Eqn.(8.31)
Min. Longitudinal Reinforcement Area, $A_{s\,min} \ge 0.01 \times A_g$ LRFD Eqn.(8.32b)

where A_g is the gross cross sectional area.

Normally choosing 1.5% main steel is a good starting point:

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

The maximum nominal longitudinal bar diameter is given by:

$$d_{bl} \le \frac{25 \times \sqrt{f_c^{'}}\left(L - 0.5D_c\right)}{f_{ye}} \qquad (in,\ psi) \qquad \text{LRFD Eqn. (8.35)}$$

where

f_{ye} = expected yield strength of reinforcing steel = 66 ksi (LRFD Eqn. (8.1))
f'_c = specified strength of concrete = 4000 psi
L = column length from point of maximum moment to point of contra-flexure

$$d_{bl,\max} = \frac{25 \times \sqrt{4000} \times \left(44 - 0.5 \times 6\right) \times 12}{66,000} = 11.79\ in$$

(There appears to be an error in the equation as the maximum value includes all possible sizes of steel rebar.)

Either spirals or hoops can be used as transverse (lateral) reinforcement in the column. However, according to LRFD Section C8.6.3, the use of spirals is recommended as the most effective and economical solution. Note, this is contrary to Memo-To-Designers 20-6, which states hoops are preferred because of their discrete nature in case of local failure. The amount of transverse reinforcement expressed as volumetric ratio:

$$\rho_s = \frac{4 \times A_{sp}}{D^{'} \times s} \text{ for circular columns} \qquad \text{LRFD Eqn. (8.23)}$$

A_{sp} = Area of transverse reinforcing hoop/spiral rebar
$D^{'}$ = Concrete section core diameter (typo in LRFD)
s = Transverse reinforcement spacing

shall be sufficient to ensure that the column meets the performance requirements as specified in LRFD Section 4.8. Additionally, such reinforcement should also meet the volumetric ratio requirements of AASHTO Sections 5.10.6 and 5.10.11.4.1 and the column shear requirements as specified in LRFD Section 8.6.

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

- Longitudinal Reinforcement
  Maximum Spacing = 8 inches — LRFD Figure C.8.6.3-1

  Minimum Spacing = Larger of : — AASHTO 5.10.3.1
    - 1.5 times the nominal diameter
    - 1.5 times maximum aggregate size
    - 1.5 in.

- Transverse Reinforcement

According to LRFD Section 8.4.1, the minimum size of transverse hoops and ties shall be equivalent to or greater than the following:

- #3 bars for #9 or smaller longitudinal bars,
- #5 bars for #10 or larger longitudinal bars, or
- #5 bars for bundled longitudinal bars.

In general, the spacing of transverse hoops and ties shall not exceed the least dimension of the compression member or 12 inches. Where two or more bars larger than #11 are bundled together, the spacing shall not exceed half the least dimension of the member or 6 inches.

For transverse reinforcement in the plastic hinge region, LRFD Section 8.8.9 specifies the maximum spacing shall not exceed the smallest of the following:

- 1/5 of the column dimension = 14.4 in, for confinement,
- 6 times the nominal diameter of the longitudinal bars = 10.2 in, to prevent longitudinal bar buckling, or
- 6 in. for single hoop or 8 in. for bundled hoops.

Keeping these requirements in mind, let us use the following reinforcement:

- #14 bars for longitudinal reinforcement
- #8 hoops @ 5 in for the plastic hinge region. Outside this region, the hoop spacing can be and should be increased to economize the design.

Assume a concrete cover of 2 inches (AASHTO Table 5.12.3-1)

d_M = Dia. of the longitudinal reinforcement loop

$$= 72 - 2 \times 2 - 2 \times 1.13 - \frac{1.88}{2} - \frac{1.88}{2} = 63.86 \text{ in}$$

Number of #14 bars = $\dfrac{61.07\, in^2}{2.25 \frac{in^2}{bar}} = 27.1 bars$

Let us use 26, #14 bars (1.44%) so that

Spacing = $\dfrac{\pi \times d_M}{26} = 7.7$ in

which meets the maximum spacing requirements outlined above. If the provided spacing turns out to be more that the maximum spacing allowed, then a smaller bar size can be used.

<u>Selecting Bent Cap Width</u>

This prototype bridge has an integral bent cap. The depth of such a bent cap is the same as the depth of the superstructure. Although a minimum cap width is not directly specified in the code, LRFD Section C8.8.4.3.2 suggests that cap beam widths one foot greater than the column diameter are encouraged so that the joint shear reinforcement is effective. Also, if additional joint shear reinforcement is required, per LRFD Section 8.13.4.2, the cap width should extend one foot from opposite sides of the column. For this example, we will use two feet greater than the column diameter to assure enough room for the possibly required joint shear reinforcement.

$$B_{cap} = D_c + 2 \text{ (ft)}$$

For our case, the bent cap width shall be 6 + 2 = 8 ft.

II.B. Balanced Stiffness Check and Preliminary Demand Assessment

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 simplest fundamental mode is to balance its stiffness and mass distribution. To this end, the LRFD Section 4.1.1 recommends that the ratio of effective stiffness between *any* two bents within a frame or between *any* two columns within a bent satisfy:

$$\frac{k_i^e \times m_j}{k_j^e \times m_i} \geq 0.5 \qquad \text{LRFD Eqn.(4.1b)}$$

The LRFD further recommends that the ratio of effective stiffness between *adjacent* bents within a frame or between *adjacent* columns within a bent satisfy:

$$\frac{k_i^e \times m_j}{k_j^e \times m_i} \geq 0.75 \qquad \text{LRFD Eqn. (4.2b)}$$

$k_i^e =$ Smaller effective bent or column stiff. $m_i =$ Tributary mass on column or bent *i*.

$k_j^e =$ Larger effective bent or column stiff. $m_j =$ Tributary mass on column or bent *j*.

Bent stiffness should also include the effects of foundation flexibility if it is determined to be significant by the geotechnical engineer.

It should be noted that LRFD Eqns. (4.1a) and (4.2a) are just special cases of Equations (4.1b) and (4.2b) and are used when the mass distribution across bents and columns is uniform. Most of the time, because of variable-width superstructures this is not the case. Therefore, it is suggested that the more general equations should be used.

If these requirements of balanced effective stiffness are not met, some of the 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.

In order to apply this check, we need to calculate the effective stiffness and tributary mass at each bent.

Balanced Frame Geometry
Although not applicable to this bridge, it is strongly recommended that the ratio of fundamental periods of vibration for adjacent frames in the longitudinal and transverse direction satisfy equation 4.3.

$$T_i / T_j \geq 0.7 \qquad \text{LRFD Eqn. (4.3)}$$

T_i = Natural period of the stiffer frame
T_j = Natural period of the flexible frame

The consequences of not meeting the fundamental period requirements of LRFD Eqn. (4.3) 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.

The computer program, *xSECTION*, is used to estimate the column effective section properties as well as the Moment-Curvature ($M - \phi$) relationship that will be needed later on to estimate member ductility.

As a first step towards calculating effective section properties for the column, the dead load axial force at column top (location of potential plastic hinge) is calculated. These column axial forces are obtained from the *CTBridge* output. Appendix A lists selective *CTBridge* input data. Selective output from this *CTBridge* run is given in Appendix B. These dead load axial forces include self-weight of the box girder, Type 732 concrete barrier, and weight of the future wearing surface (35 psf). The concrete unit weight used is 150 lb/ft^3. It should also be noted that these loads do not include weight of the integral bent cap. The *CTBridge* model has the regular superstructure cross-section with flared bottom slab instead of solid cap section. To be exact, only the weight of extra concrete should be added to the *CTBridge* output values to account for the full bent cap weight. The weight of whole solid cap was added to the *CTBridge* results (conservative).

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

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

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

$$\text{Average Bent Cap Length} = \frac{49.83 + 43.08}{2} \times \frac{1}{\cos(20^0)} = 49.44\,ft\ .$$

$$\text{Bent Cap Weight} = 8 \times 6.75 \times 49.44 \times 0.150 = 400\ kips$$

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

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

LRFD Section 8.7.2 specifies the maximum axial load in a column designed to be ductile shall not be greater than $0.2f'_{ce}A_g$. However, efforts should be made to keep the dead load axial forces in columns around 10% of their ultimate compressive capacity, $P_u = f'_c A_g$. This is recommended to make sure that the column does not experience brittle compression failure and also that any potential $P - \Delta$ effects remain within acceptable limits. In our case, axial forces are about 10% of such ultimate compressive capacity. When this ratio starts approaching 15%, increasing column size or adding extra columns should be considered.

<u>Material and Effective Column Section Properties (I_e)</u>

<u>Material Properties</u>

- Concrete

As per Caltrans common design practice, $f_c^{'} = 4{,}000\ psi$ is used for superstructure, columns, piers, and pile shafts. For other components like abutments, wingwalls, and footings, use of $f_c^{'} = 3{,}600\ psi$ is specified.

As per LRFD Section 8.4, expected material properties are to be used to calculate section capacities for all ductile members. To be consistent between the demand and capacity, expected materials will also be used to calculate member stiffness. For concrete, the expected yield strength, f'_{ce}, is taken as:

$$f'_{ce} = Greater\ of \begin{pmatrix} 1.3 \times f'_c \\ 5{,}000\ psi \end{pmatrix} \qquad \text{LRFD Eqn. (8.7)}$$

In our case,

since $\left[f'_{ce} = 1.3 \times 4{,}000 = 5{,}200\ psi \right] > 5{,}000\ psi$, $f'_{ce} = 5{,}200\ psi$ will be used.

Other concrete properties used are listed in LRFD Section 8.4.4.

- Steel

Grade A706 will be used for reinforcing bar steel. The material properties for such steel are given in LRFD Section 8.4.2. Note the slight differences from Caltrans SDC.

It is well known that concrete cover spalls off at very low ductility levels. Therefore, the effective (cracked) moment of inertia values will be used to assess the seismic response of all ductile members as specified in LRFD Section 5.6.2. However, per LRFD Section 5.6.3, no stiffness reduction is recommended for prestressed concrete box girder sections.

The following values for the column section, and the concrete and steel properties are used as input into the *xSection* program:

- Column Dia. = 72.0 in
- Concrete Cover = 2 in
- Main Reinforcement: #14 bars, tot. 26.
- Lateral Reinforcement: #8 hoops @ 5 in c/c.
- $f'_{ce} = 5{,}200\, psi$
- The program calculates the modulus of elasticity of concrete internally.
- $E_s = 29{,}000\ ksi$
- $f_{ye} = 1.1 f_y = 1.1 \times 60\ ksi = 66\, ksi \quad (CT \quad 68\, ksi)$ LRFD Eqn. (8.1)
- $f_{ue} = 1.4 f_{ye} = 1.4 \times 66\ ksi = 92\, ksi \quad (CT \quad 95\, ksi)$ LRFD Eqn. (8.2)
- $\varepsilon_{sh} = \begin{cases} 0.0150 & for\ \#8\ bars \\ 0.0075 & for\ \#14\ bars \end{cases}$
- $\varepsilon^R_{su} = \begin{cases} 0.06 & for\ \textbf{column}\ longitudinal\ reinforcement \quad Longitudinal\ Steel \\ 0.120 & for\ \#10\ bars\ or\ smaller \quad (CT \quad 0.09) \quad Transverse\ Steel \end{cases}$

Bent 2 Column Axial Force, P_c=1,694 kips.

Bent 3 Column Axial Force, P_c=1,653 kips.

Using these section and material properties, a section analysis is now performed using *xSECTION* program.

- An input file to *xSECTION* for the Bent 2 Column is shown in Appendix C. An input file for Bent 3 Column will be similar except for different column axial loads.

- Output for this *xSECTION* run is shown in Appendix D.

- Moment-Curvature ($M-\phi$) diagram for Bent 2 Column is shown in Appendix E.

For a single pinned-fixed column, the lateral bending stiffness is given as

$$k_2^e = \frac{3 \times E_c \times I_e}{L^3}$$

L = Column height, measured from the pin at top of footing to the soffit of the bridge (LRFD Figure 4.2)

The concrete modulus of Elasticity, E_c, is given by

$$E_c = 33000 \times (w_c)^{1.5} \times \sqrt{f_c^{'}} \; (ksi) \qquad \text{AASHTO Eqn. (5.4.2.4-1)}$$

where w_c is the unit weight of concrete in k/ft^3. Using expected value, $f_{ce}^{'}$,

$$E_c = 33000 \times (0.150)^{1.5} \times \sqrt{5.2} = 4,372 \; ksi$$

- Bent 2 Stiffness

From the $M-\phi$ analysis results, the cracked moment of inertia, $I_e =$ 23.872 ft^4 (See Appendices D and E).

$$k_2^e = (2\; Columns)\left[\frac{(3)\times(4,372)\times(23.872)\times(12^4)}{(44\times 12)^3}\right]$$

or $$k_2^e = 88.22 \; \frac{k}{in}$$

- Bent 3 Stiffness

Again from $M-\phi$ analysis results, $I_e =$ 23.772 ft^4

$$k_3^e = (2\ Columns)\left[\frac{(3)\times(4,372)\times(23.772)\times(12^4)}{(47\times 12)^3}\right]$$

or $$k_3^e = 72.08\ \frac{k}{in}$$

Total tributary mass at Bent 2 = $\frac{(2\ Columns)\times(1,694)}{(32.2)\times(12)} = 8.77\ kips - s^2 / in$

Total tributary mass at Bent 3 = $\frac{(2\ Columns)\times(1,653)}{(32.2)\times(12)} = 8.56\ kips - s^2 / in$

$$\frac{k_i^e \times m_j}{k_j^e \times m_i} = \frac{(72.08)\times(8.77)}{(88.22)\times(8.56)} = 0.84 \qquad \text{OK}$$

Therefore, the current span layout configuration satisfies the LRFD balanced stiffness criteria for adjacent bents in a frame.

The columns within each bent are of the same height and they support equal gravity loads, thus LRFD Equation (4.1b) is automatically satisfied.

As mentioned earlier, if foundation flexibility is significant, its effect must be considered while performing these checks.

In case the bents/frames do not meet the LRFD requirements for balanced stiffness, one or more of the following techniques (LRFD 4.1.3) can be considered for adjusting the fundamental period of vibration:

- Use of oversized shafts.
- Adjust the effective column length. Examples include lowering footings, using isolation casings.
- Modify end fixities.
- Redistribute superstructure mass.
- Vary column size and/or longitudinal reinforcement.
- 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 Caltrans MTD 6-1 may help:

- Pin columns in multi-column bents and selected single columns at base adjacent to abutments.

- Use higher strength column concrete.

- Shorten spans and add bents.

- Use pile shafts in lieu of footings.

- Add more additional columns per bent.

Before we proceed with a comprehensive analysis to consider the effects of change in columns axial forces due to seismic overturning moments and also the effects of soil overburden on column footings, let us now check the component ductility capacity, μ_c, for ductile members to make sure that basic ductility requirements are met. Note a minimum local member ductility capacity is not explicitly required by the current LRFD. However, it is Caltrans' practice that each ductile member have a minimum μ_C of 3 to ensure dependable rotational capacity in the plastic hinge regions regardless of displacement demands (SDC Section 3.1.4.1). Catrans practice is to use an idealized bilinear $M-\phi$ curve to estimate the idealized yield displacement and deformation capacity of ductile members.

For Bent 2 columns,

L = 44 ft.
$\phi_Y = 0.000078\ rad/in$ as read from the $M-\phi$ data listed in Appendix D.

The analytical plastic length, L_p, is estimated using LRFD Eqn. (4.12) as

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

With L = 528 in, f_{ye} =66 ksi, and d_{bl} =1.693 in,

$$L_p = 0.08 \times 528 + 0.15 \times 66 \times 1.693 = 59.00\ in > (0.3 \times 66 \times 1.693) = 33.52 \text{ in.}$$

$$\therefore L_p = 59.00 \text{ in.}$$

The idealized column yield displacement is now calculated as:

$$\Delta_Y = \frac{1}{3} \times (528)^2 \times 0.000078 = 7.25 \text{ in.}$$

Plastic curvature, $\phi_p = 0.000969$ rad/in as read from the $M - \phi$ data shown in Appendices D and E.

Plastic rotation, $\theta_p = L_p \times \phi_p = 59.00 \times 0.000969 = 0.057171$ rad.

Plastic displacement, $\Delta_p = 0.057171 \times \left(528 - \frac{59.00}{2} \right) = 28.50$ in.

Total Displacement Capacity, $\Delta_c = 7.25 + 28.50 = 35.75$ in. (CT 29.40 in.)

Local Displacement Ductility Capacity for Bent 2 Columns is now calculated as

$$\mu_c = \frac{\Delta_c}{\Delta_Y} = \left(\frac{35.75}{7.25} \right) = 4.93 > 3$$ (CT 4.1) OK. SDC Eqn. (3.6)

Similarly, for Bent 3 columns,

$$\mu_c = \frac{\Delta_c}{\Delta_Y} = \left(\frac{40.40}{8.27} \right) = 4.89 > 3$$ (CT 4.0) OK. SDC Eqn. (3.6)

Thus, the column section size and reinforcement meets the local displacement ductility capacity requirements of the SDC Section 3.1.4.1.

<u>Displacement Demand</u>

As this bridge is a regular 3-span bridge with uniformly distributed stiffness, per LRFD Section 4.2, the seismic demands can be estimated using Procedure 1 – Equivalent Static Analysis – based on a simple lumped-mass method**.** 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. As a preliminary check, seismic demands based on the previously calculated bent stiffnesses will be compared to the local displacement ductility and capacity of each column:

For Bent 2

The period of fundamental mode of vibration, T_2, is calculated as

$$T_2 = 2\pi \sqrt{\frac{m_2}{k_2^e}} = 2\pi \sqrt{\frac{8.77}{88.22}} = 1.98 \text{ sec.}$$

Similarly, for Bent 3,

$T_3 = 2.17$ sec. The longer period is expected because the Bent 3 columns are taller and support more gravity load.

From the design spectrum curve shown in Figure 2, the values of spectral acceleration for two bents are read to be

$$a_2 = 0.49g \qquad (CT \quad 0.36g)$$
$$a_3 = 0.45g \qquad (CT \quad 0.33g)$$

The displacement demand can now be estimated as:

$$\Delta_D = \frac{m \times a}{k_e}$$

For Bent 2 Columns, the displacement demand is:

$$\Delta_D = \frac{8.77 \times 0.49 \times 32.2 \times 12}{88.22} = 18.82 \text{ in.} \qquad (\text{CT } 13.92 \text{ in.})$$

As this is a preliminary calculation, it is assumed that the displacement demand is resisted only by the columns (i.e. $\Delta_D^{global} \leq \left[\Delta_C^{system} = \Delta_Y^{col} + \Delta_{pd}^{col}\right]$). Subsequently, the plastic displacement demand on the column is estimated to be:

$$\Delta_{pd} = \Delta_D - \Delta_Y = 18.82 - 7.25 = 11.57 \; in$$

The displacement ductility demand is:

$$\mu_D = 1 + \frac{\Delta_{pd}}{\Delta_y} = 1 + \frac{11.57}{7.25} = 2.60 \leq 8.0 \quad OK. \quad (CT \;\; 1.9 \leq 5.0) \quad \text{LRFD Section 4.9}$$

Also $\Delta_D = 18.82 \; in < \Delta_C = 35.75 \; in$ OK LRFD Eqn. (4.6)

Similarly, for Bent 3 Columns, the displacement demand is:

$$\Delta_D = \frac{8.56 \times 0.45 \times 32.2 \times 12}{72.08} = 20.65 \text{ in.} \qquad (\text{CT } 15.25 \text{ in.})$$

The plastic displacement demand is estimated to be:

$$\Delta_{pd} = 20.65 - 8.27 = 12.38\, in$$

The displacement ductility demand is:

$$\mu_D = 1 + \frac{12.38}{8.27} = 2.50 \ \le \ 8.0 \qquad OK. \qquad (CT \ \ 1.8 \le 5.0) \qquad \text{LRFD Section 4.9}$$

Also $\Delta_D = 20.65\, in < \Delta_C = 40.40\, in$ OK LRFD Eqn. (4.6)

The displacement ductility demand criteria is slightly different between LRFD and Caltrans SDC. The proposed LRFD guidelines relates the maximum displacement ductility demand criteria, $\mu_D = 1 + \frac{\Delta_{pd}^{col}}{\Delta_Y^{col}}$, to only local member displacements whereas Caltrans SDC defines the maximum displacement ductility demand, $\mu_D = \frac{\Delta_D^{system}}{\Delta_Y^{system}}$, with respect to the global system or sub-system. In both the LRFD and Caltrans SDC, the total displacement demand, $\Delta_{D,}$ includes components attributed to foundation flexibility, $\Delta_{F,}$ flexibility of capacity protected members such as bent caps, $\Delta_{B,}$ and the flexibility attributed to elastic and inelastic response of ductile members, $\Delta_{Y,}$ and $\Delta_{P,}$ respectively. However, the LRFD equation pertains only to the demand on a specific column as implied by:

$$1 \equiv \frac{\Delta_Y^{col}}{\Delta_Y^{col}}$$

$\Delta_{pd} \equiv$ Column plastic displacement demand after column yields in **system**

$$= \Delta_D^{system} - \Delta_Y^{system}$$

The Caltrans SDC equation extends the ductility capacity to the entire system when defining the yield displacement. The LRFD and Caltrans SDC will yield the same ductility demand only when $\Delta_Y^{col} = \Delta_Y^{system}$ (i.e. there is no additional flexibility due to the foundation or other capacity protected members in the system). In general, Caltrans SDC will yield lower ductility demands since the system's yield displacement increases due to inclusion of other flexible members.

However, the maximum allowable displacement ductility demand for multi-column bents supported on fixed or pinned footings is increased to 8 in LRFD from 5 in Caltrans SDC. For Bent 2, the calculated LRFD and SDC ductility demands are 2.6 and 1.9, respectively. The fact that the column is at 33% of the allowable ductililty using LRFD versus 38% using SDC suggests that the column has slightly more reserve displacement capacity. Though the calculated ductility demand is lower using SDC, it is offset by the lower available ductility. In general, the LRFD criteria allows more ductile response from the column, and consequently, more damage relative to SDC.

II.C. Transverse Pushover Analysis and Design

II.C.1. Modeling Assumptions Including Soil Springs

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 cap beam is not infinitely rigid. The flexibility provided by the bent cap alters the column end condition. Also, the effect of soil-structure interaction can be included . Such effect can be significant in the case where footings are buried deep in the ground.

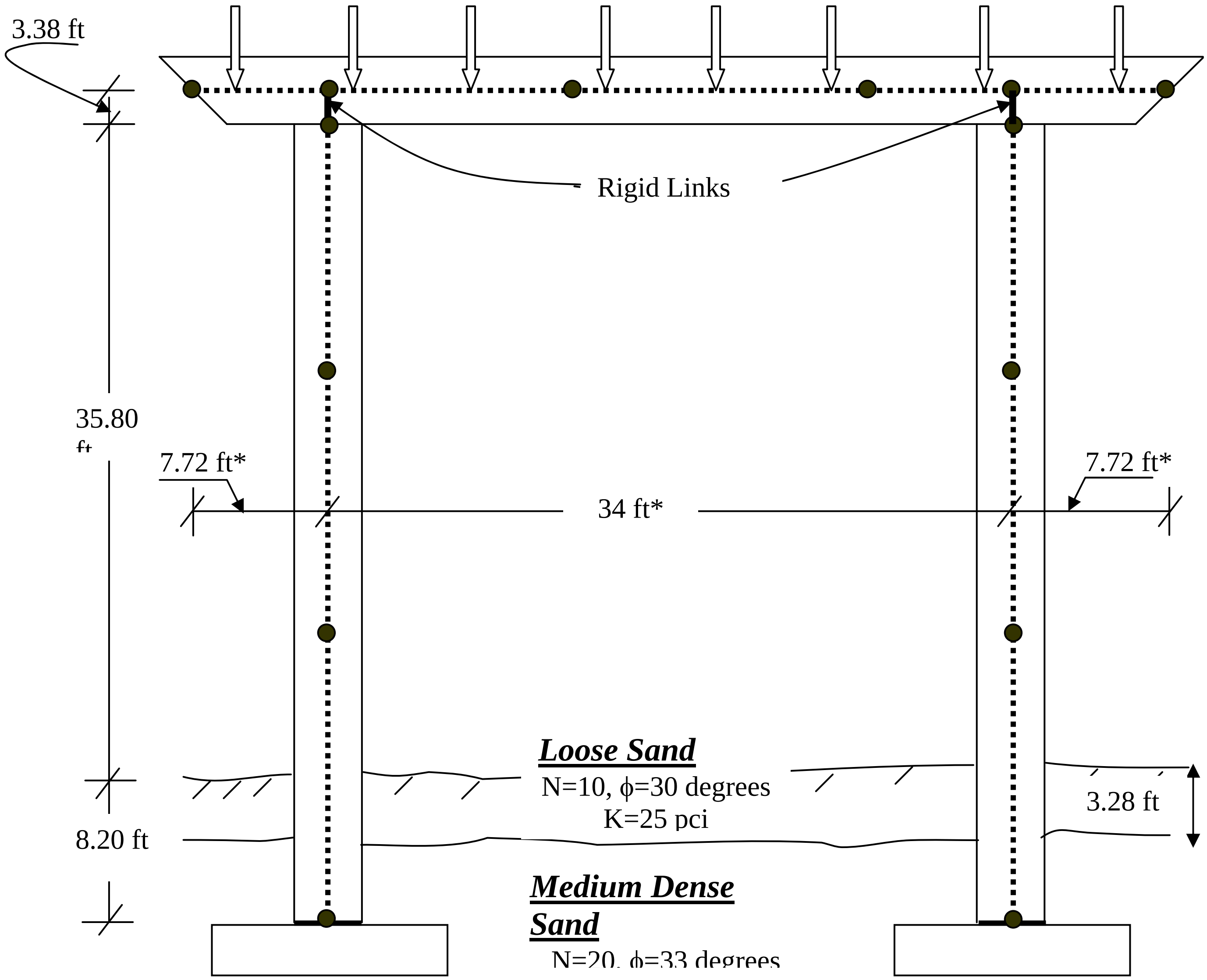
Per LRFD Section 4.8.2, Inelastic Quasi-static Pushover Analysis is used to determine the reliable displacement capacity of a structure. Pushover is mainly a capacity estimating procedure but it can also be used to estimate demand for structures having characteristics previously outlined. A similar procedure outlined in LRFD Section 4.11.4, without the column overstrength factors, will be followed to determine the displacement capacity of the structure. However, the computer program *wFRAME* is used to perform pushover analysis so that bent flexibility can be included. The following conventions are used:

- The model is two dimensional with beam elements along the center of cap beam 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 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 that is two times higher than the regular bent section. The moment capacity used for such element is two times 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 top of the column but below the soffit.
- The soil effect is included as $p-y$ springs applied to the column portion below the ground. The data used for this site is shown in Appendix F.

Figure 3 on the following page schematically shows such a model.

The following values of effective section properties and idealized column plastic moment capacity (under dead loads only) are used as input to *wFRAME* program for pushover analysis.

$P_c(kips)$	$M_p(ft-kips)$	$I_e(ft^4)$	$\phi_Y(rad/in)$	$\phi_p(rad/in)$
1,694	13,808	23.872	0.000078	0.000969



* Dimensions along the skewed bent line.

Figure 3 Transverse Pushover Analysis Model

The effective bent cap width is calculated as per LRFD Section 8.11. The Appendix G shows the *xSECTION* model of the bent cap. The Appendices H1 and H2 show selective portions of *xSECTION* output showing cap section properties for positive and negative bending. The following section properties are used:

$A = 62.62\ ft^2$

$$\left.\begin{aligned} I_{eff}^{+ve} &= 52.95\ ft^4 \\ I_{eff}^{-ve} &= 48.64\ ft^4 \end{aligned}\right\} \quad I_{eff}^{avg} = 50.80\ ft^4$$

As per LRFD Section 8.9, capacity protected concrete components such as the bent cap, superstructure and footing shall be designed to remain essentially elastic when the column reaches its overstrength capacity. This is required in order to make sure that no plastic hinge forms in these components. Per LRFD Section 4.11.1 and 4.11.2, it is required that the bent cap flexure and shear capacity equals or exceeds the demand imposed by the column overstrength moment. Appendix I lists *wFRAME* input file.

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 back side experiences the same value in tension, reducing the net axial load. Based upon their behavior, these columns are usually known as compression and tension columns, respectively.

At the instant when the first plastic hinge forms (in this case at the top of the compression column), the superstructure displacement and corresponding lateral force values are obtained from *wFRAME* output. Appendix J shows *wFRAME* output.

$\Delta_y = 8.44\, in$

$F_y = 0.169 \times (3,382) = 572\ kips$

where 3,382 lb 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 911 kips and 2,470 kips, respectively. These values can be quickly checked using simple hand calculations as described below:

$$M_{overturning} = 572 \times 47.38 = 27,101\ ft-kips.$$

The axial compression corresponding to such overturning is given as

$$\Delta P = \pm \frac{27,101}{34} = \pm 797\ kips$$

The axial force in the compression column will increase to $1,694 + 797 = 2,491\, kips$. The tension column will see its axial compression drop to $1,694 - 797 = 897\ kips$. These values compare very well with the *wFRAME* results. Small differences are probably due to the presence of soil in the more realistic *wFRAME* model.

Now we know that the overturning caused by seismic forces results in significant change in the column axial forces. We also know that the effective section properties and column yield moments are influenced by the level of axial force. Therefore, for these updated axial forces, the section properties are calculated again using *xSECTION*. See Appendix K for these results.

Column Type	$P_c(kips)$	$M_p(ft-kip)$	$I_e(ft^4)$	$\phi_Y(rad/in)$	$\phi_p(rad/in)$
Tension	911	12,502	21.647	0.000078	0.000980
Compression	2470	14,906	25.728	0.000078	0.000885

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

The effect of change in the axial force in a column section due to overturning moments can be summarized as below:

- M_P changes.
- The tension column has become more ductile while the compression column has become less ductile.
- The required flexural capacity of cap beam that is needed to make sure that the hinge forms at column top is now obviously larger.

With updated values of M_P and I_e, we run a second iteration of the *wFRAME* model. As the frame is pushed laterally, the compression column yields at the top. The tension column has not reached its capacity yet. See Appendix L for these results. At this moment,

$$\Delta_{y(1)} = 8.72\ in$$

At this stage, the column axial forces are read to be 881 kips, 2,501 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 iteration two 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 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. Appendix L shows selective portions of the *wFRAME* output file. The Force-Displacement relationship is shown in Appendix M.

$$\Delta_{y(2)} = 10.29\ in.$$

This is an updated value of the idealized yield Δ_Y which was calculated previously based upon the assumption that the cap beam is infinitely rigid.

$$F_{y(2)} = 0.190 \times (3{,}382) = 643\ kips$$

II.C.2.i. Displacement Capacity and Demand

Using the procedure previously described to calculate the plastic deformation and using the section properties listed above, the section capacities for both columns are calculated to be:

Tension Column

$$L_p = 59.00\ in$$
$$\Delta_p = 28.82\ in$$
$$\Delta_c = 10.29 + 28.82 = 39.11\ in$$
$$\left(CT : \mu_c = \frac{39.11}{10.29} = 3.80 \right)$$

Compression Column

$$L_p = 59.00\ in$$
$$\Delta_p = 26.03\ in$$
$$\Delta_c = 8.72 + 26.03 = 34.75\ in$$
$$\left(CT : \mu_c = \frac{34.75}{8.72} = 3.99 \right)$$

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

Hinge Location	Yield Displacement (in)	Plastic Deformation (in)	Total Displacement Capacity (in)
Compression Column Top	8.72 (CT 8.79)	26.03 (CT 20.22)	34.75* (CT 29.01)
Tension Column Top	10.29 (CT 10.52)	28.82 (CT 24.79)	39.77 (CT 35.31)

* Critical bent displacement capacity, Δ_C . The bent capacity calculated previously was to size up the members before proceeding with more realistic and comprehensive analysis that includes the effects of bent cap flexibility.

Estimating the Seismic Demand

As discussed previously, since the prototype bridge is well balanced with uniformly distributed stiffness, the response of the structure can be captured by a predominant translational mode of vibration. Per LRFD Section 5.4.2, Procedure 1 – Equivalent Static Analysis – is suitable to determine the seismic demands. In this example, individual bents rather than the entire structure is examined, resulting in a simple lumped-mass model. Though the model lacks the interaction between the adjacent bents and abutments that would be captured using the Uniform Load Method as suggested in Section 5.4.2, the estimated demands wll be more conservative since the system is more flexible.

Per LRFD Section 4.3, the global seismic displacement demands, Δ_D^T, should be determined independently along two perpendicular axes using Procedure 1, 2, or 3, to account for the directional uncertainty of earthquake mortions and the simultaneous occurrences of earthquake forces in two perpendicular horizontal directions.. The subsequent orthogonal displacements should then be combined using the 100%-30% rule as described in LRFD Section 4.4, resulting in two independent load cases:

Load Case 1: 100% Δ_D^{Long} + 30% Δ_D^{Trans}
Load Case 2: 30% Δ_D^{Long} + 100% Δ_D^{Trans}

However, because the bents are skewed 20°, the orthogonal axis requirement complicates the determination of the capacities and demands along each of these axis. One can choose to align the analysis along the centerline of bent or the centerline of the superstructure; however, the resulting orthogonal analysis to either choice would require a more complicated, three-dimensional pushover analysis.

Caltrans SDC Section 2.1.2 provides an alternative to the method above which allows the application of the ground motion along the principal axes of individual components. Consequently, ground motion must be applied at a sufficient number of angles to capture the maximum deformation of all critical elements . In other words, the demand can be determined along both the bent centerline (transverse analysis) and the superstructure centerline (longitudinal analysis) and compared to the corresponding displacement capacities as calculated previously. Although this option is not currently presented in the LRFD, this is a viable alternative that will simplify the capacity analysis while yielding realistic seismic demands for design of the columns.

The effective bent stiffness is estimated as

$$k_2^e = \frac{F_y}{\Delta_y} = \frac{643}{10.29} = 62.49\,\frac{k}{in}$$

and the period of vibration, T, is calculated to be

$$T = 2\pi \times \sqrt{\frac{8.77}{62.49}} = 2.35\sec$$

From the design spectrum curve, the spectral acceleration a_2 is read to be 0.41g (CT 0.30g). The maximum seismic displacement demand is estimated as

$$\Delta_d = \frac{8.77 \times (0.41 \times 32.2 \times 12)}{62.49} = 22.23\,in \qquad (CT \quad 16.63\,in)$$

The displacement ductility demand for the more critical compression column is:

$$\Delta_{pd} = \Delta_D - \Delta_Y^{system} = 22.23 - 8.72 = 13.51\,in$$

$$\mu_D = 1 + \frac{\Delta_{pd}}{\Delta_Y^{column}} = 1 + \frac{13.51}{7.25} = 2.86 \ \leq \ 8.0 \qquad OK. \quad (CT \ \ 1.6 \leq 5) \quad \text{LRFD Section 4.9}$$

and also $\Delta_d = 22.23\,in < \Delta_c = 34.75\,in.$ LRFD Equation (4.6)

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 out of the 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 in Section II.C.2.ii, the so-called $P-\Delta$ controls the column flexural design.

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

Tension Column

$$L_p = 61.88\,in$$
$$\Delta_p = 32.56\,in$$
$$\Delta_c = 11.21 + 32.56 = 43.77\,in$$
$$\left(CT : \mu_c = \frac{43.77}{11.21} = 3.90 \right)$$

Compression Column

$$L_p = 61.88\,in$$
$$\Delta_p = 29.29\,in$$
$$\Delta_c = 9.63 + 29.29 = 38.92\,in$$
$$\left(CT : \mu_c = \frac{38.92}{9.63} = 4.04 \right)$$

Estimating the Seismic Demand

$$k_e^3 = \frac{F_y}{\Delta_y} = \frac{0.191 \times 3,278}{11.21} = 55.85\,\frac{k}{in}$$

and the period of vibration, T, is calculated to be

$$T = 2\pi \times \sqrt{\frac{8.56}{55.85}} = 2.45\,\text{sec}$$

From spectrum design curve, the spectral acceleration a_3 is read to be 0.40g (CT 0.29g). The maximum seismic displacement demand is estimated as:

$$\Delta_d = \frac{8.56 \times (0.40 \times 32.2 \times 12)}{55.85} = 23.69\,in \qquad (CT \quad 17.41\,in)$$

The displacement ductility demand for the more critical compression column is:

$$\Delta_{pd} = \Delta_D - \Delta_Y^{system} = 23.69 - 9.63 = 14.06\,in$$

$$\mu_D = 1 + \frac{\Delta_{pd}}{\Delta_Y^{column}} = 1 + \frac{14.06}{8.27} = 2.70 \ \le \ 8.0 \qquad OK. \quad (CT \ \ 1.5 \le 5) \quad \text{LRFD Section 4.9}$$

and also $\Delta_d = 23.69 \ in < \Delta_c = 38.92 \ in.$ LRFD Equation (4.6)

II.C.2.ii. $P - \Delta$ Check

We have relatively heavily loaded tall columns. $P - \Delta$ effects could be significant for this type of situation. However, per LRFD Section 4.11.5, $P - \Delta$ effects can be ignored if these are limited to 25% of the column capacity i.e.

$$P_{dl} \times \Delta_r \le 0.25 \times M_p \qquad \text{LRFD Eqn. (4.9a)}$$

$$\left(CT: \quad P_{dl} \times \Delta_D \le 0.20 \times M_p^{col} \right)$$

where

P_{dl} = Dead load axial force.

Δ_r = The lateral offset between the point of contra-flexure and the furthest end of the plastic hinge.

<u>For Bent 2 Columns</u>

Column Axial Dead Load = 1,694 kips.
Plastic Moment Capacity = 13,808 ft-kips.
Maximum Seismic Displacement = 22.23 in.

$$\frac{P_{dl} \times \Delta_r}{M_p} = \frac{1,694 \times 22.23}{(13,808 \times 12)} = 0.23 < 0.25 \qquad (CT \quad 0.17 \le 0.20) \qquad \text{OK}$$

<u>For Bent 3 Columns</u>

Column Axial Dead Load = 1,653 kips.
Plastic Moment Capacity = 13,747 ft-kips.
Maximum Seismic Displacement = 23.69 in.

$$\frac{P_{dl} \times \Delta_r}{M_p} = \frac{1,653 \times 23.69}{(13,747 \times 12)} = 0.24 < 0.25 \qquad (CT \quad 0.18 \le 0.20) \qquad \text{OK}$$

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.

II.C.2.iii. Minimum Lateral Strength Capacity (0.1P_{DL})

According to the LRFD Section 8.7.1, the minimum lateral strength of each column shall be 0.1 P_{DL}. From the force deflection data shown in Appendix M, the minimum lateral strength of Bent 2 is 0.190g or 0.095g for each column (close to 0.1g OK).

II.C.3. Column Shear and Bent Cap Capacity Check

II.C.3.i. Column Shear Check

According to LRFD Section 8.6.1, the seismic demand shall be based upon the overstrength shear, V_0, associated with the column overstrength moment M_0 (LRFD Section 8.5). Since shear failure tends to be brittle, shear capacity for ductile members shall be conservatively determined using nominal material properties. Note, the pushover analysis that was used to determine the displacement capacity followed the method outlined in LRFD Section 4.11.4. This method specifies a procedure for mulit-column bents to obtain column design shear forces and superstructure/bent cap design forces. The results from the pushover analysis will now be used to determine the column overstrength shear.

According to the LRFD

$$\phi \times V_n \geq V_0 \qquad \phi = 0.85 \qquad \text{LRFD Eqn. (8.9)}$$

where nominal shear capacity, V_n, is given as summation of concrete and steel shear capacities i.e.

$$V_n = V_c + V_s \qquad \text{LRFD Eqn. (8.10)}$$

- Shear Demand V_0

For Bent 2, $M_0 = 1.2 \times M_p = 1.2 \times 14,906 = 17,887 \ ft - kips$

This overstrength moment includes the effects of overturning.

Shear demand associated with overstrength moment can be estimated as:

$$V_0 = \frac{M_0}{L} = \frac{17,887}{44} = 407 \, kips \qquad (CT \quad 408 \, kips)$$

Alternately, the maximum shear demand can also be determined from *wFRAME* results. The maximum column shear demand reported by such analysis is multiplied by a factor of 1.2 to obtain the shear demand associated with the overstrength moment. From *wFRAME* output, the maximum column shear demand equals $1.2 \times 348 = 418\ kips$ (CT 419 kips). See *wFRAME* output results in Appendix L.

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 *wFRAME* output.

- Concrete Shear Capacity

$V_c = v_c \times A_e$ LRFD Eqn. (8.11)

where

v_c = Allowable concrete shear stress

$A_e = 0.8 \times A_g$ LRFD Eqn. (8.12)

Now

$$\left[\upsilon_c = \alpha' \left(1 + \frac{P}{2000 A_g} \right) \sqrt{f_c'} \right] \leq 3.5 \sqrt{f_c'}$$ LRFD Eqn. (8.13)

As one can see from the equations for concrete shear capacity, the plastic hinge region is more critical as the capacity will be lower in the this region. Further, the shear capacity will be smallest when the axial load is low. The controlling shear capacity will be found in the tension column. Now

$$\alpha' = \frac{0.03}{\mu_D} \rho_s f_{yh}$$ LRFD Eqn. (8.16)

where for circular column, the confinement reinforcement ratio is given as

$$\rho_s = \frac{4 A_{sp}}{D' s}$$ LRFD Eqn. (8.23)

For our case,

$A_{sp} = 0.79\ in^2$

$D' = 72 - 2 - 2 - \frac{1.13}{2} - \frac{1.13}{2} = 66.87\ in$

$s = 5\ in$

Making these substitutions in above equation yields

$\rho_s = 0.009451$

From the pushover analysis results, the displacement ductility, $\mu_d = 2.86$.

Using $f_{yh} = 60\ ksi$,

$$\alpha' = \frac{0.03}{2.86} \times (0.009451) \times (60000) = 5.95$$

Similarly,

$$1 + \frac{P}{2,000 \times A_g} = 1 + \frac{911 \times 10^3}{2,000 \times \frac{\pi}{4} \times (6 \times 12)^2} = 1.11$$

The maximum allowable concrete shear stress is calculated as

$$\left[\upsilon_c = 5.95 \times 1.11 \times \sqrt{4,000} = 418\ psi\right] > \left[3.5\sqrt{4,000} = 221\ psi\right]$$

Use $\upsilon_c = 221\ psi \qquad (CT \quad 211\ psi)$.

$$\therefore V_c = \frac{221 \times 0.8 \times \frac{\pi}{4} \times (6 \times 12)^2}{1,000} = 720\ kips \qquad (CT \quad 687\ kips)$$

- Transverse Reinforcement Shear Capacity

$$V_s = \frac{\pi}{2}\left(\frac{A_v f_{yh} D'}{s}\right)$$ LRFD Eqn (8.25)

where $A_v = n \times A_{sp}$ LRFD Eqn (8.26)

n = number of individual interlocking spiral or hoop core sections

As specified in the LRFD Section 8.6.6, the minimum spiral reinforcement ratio, ρ_s, for each individual core should not be less than:

$\rho_{s,\min} = 0.4\% = 0.004$

$\rho_{s,provided} = 0.009451$ (#8 hoop @ 5 in c-c). OK

$$\therefore V_s = \frac{\pi}{2} \times \frac{0.79 \times 60 \times 66.87}{5} = 996\ kips$$

According to LRFD Section 8.6.5, the shear strength, V_s, provided by reinforcing steel shall not be more than $8 \times \sqrt{f_c'} \times A_e = 1,648$ kips. Therefore,

$\left[\phi \times V_n = 0.85 \times (720 + 996) = 1,459\ kips\right] > \left[V_0 = 418\ kips\right]$. OK.

$\left(CT: \left[\phi \times V_n = 1,431\ kips\right] > \left[V_0 = 419\ kips\right]\right)$

Similarly for Bent 3 columns, the shear demand corresponding the overstrength moment is estimated as

$$V_0 = \frac{M_0}{L} = \frac{1.2 \times 14,838}{47} = 379\ kips \qquad (CT \quad 380\,kips)$$

From the *wFRAME* analysis results, the maximum column shear demand = $1.2 \times 339 = 407\ kips$ (CT 409 kips).

Going through similar calculations, we determine that

$$\left[\phi \times V_n = 0.85 \times (720 + 996) = 1,459\ kips\right] \quad > \quad \left[V_0 = 407\ kips\right]. \qquad \text{OK}$$

$$\left(CT: \quad \left[\phi \times V_n = 1,425\ kips\right] \quad > \quad \left[V_0 = 409\ kips\right]\right)$$

Although no calculations are done here, the column shear key shall be designed for axial and shear forces associated with column overstrength moment including the effects on overturning. As recommended in LRFD Section 8.15, the key reinforcement shall be located as close to the center of the column as possible in order to minimize developing a force couple within the shear key reinforcement. Steel pipes may be used to relieve congestion and reduce the moment generated within the key.

II.C.3.ii. Bent Cap Flexural and Shear Capacity

According to LRFD Section 8.9, 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 is determined based on a stress-strain compatibility analysis using a $M - \phi$ diagram. The expected nominal moment capacity shall be based upon the expected concrete and steel strength values when either concrete strain reaches 0.005 (CT 0.003) or the steel strain reaches ε_{SU} $(CT \ \ \varepsilon_{SU}^{R})$ as derived from the applicable stress-strain relationship. Appendix G shows *xSECTION* model of the bent cap. As mentioned earlier, effective bent cap width is calculated as per LRFD Section 8.11. The design for service loading 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 flexural capacity of bent cap is estimated to be

$$M_{ne}^{+ve} = 20,580\ ft-kips \qquad M_{ne}^{-ve} = 18,899\,ft-kips$$

$$\left(CT: \quad M_{ne}^{+ve} = 21,189\ ft-kips \qquad M_{ne}^{-ve} = 19,436\,ft-kips\right)$$

The Appendices H1 and H2 show such values. 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* push over analysis of Bent 2 with column moment capacity to be M_0. As shown in Appendix N (right side push over), bent cap moment demands are:

$$M_D^{+ve} = 14,196\ ft-kips \qquad M_D^{-ve} = 15,021\ ft-kips$$
$$\left(CT: \quad M_D^{+ve} = 14,350\ ft-kips \qquad M_D^{-ve} = 15,072\ ft-kips\right)$$

Similar to the design for moment, the bent cap needs to be designed for the maximum seismic shear demand that corresponds to the column overstrength moment as stated in LRFD Section 4.11.1.

Though not explicitly stated by the LRFD, as the bent cap is capacity protected, the shear capacity of the bent cap is calculated as per AASHTO 5.8.3.3 using nominal values:

[$\phi V_n = \phi(V_c + V_s)$] $\geq$ V_u

where $\phi = 0.9$

- Maximum V_u and Associated M_u

  The overstrength moment is considered indirect loading so the critical section for shear will be taken at the face of support. The maximum seismic shear demand corresponding to the column overstrength moment can be found in Appendix N.

  V_u = 2001 kips

  Associated M_u = -15021 k-ft (Negative)

- Determine b_v and d_v

  b_v = 8 ft = 96 in (Width of web)

$$d_v = d_e - \frac{a}{2}$$

or

$$d_v = Greater\ of \left\{ \begin{matrix} 0.9d_e = 0.9 \times 74.5\,in = 67.05\,in \\ or \\ 0.72h = 0.72 \times 81\,in = 58.32\,in \end{matrix} \right\} = 67.05\,in$$

Note since the moment is in the negative direction, d_e is taken from the bottom of the bent to the centroid of the top main reinforcement.

- Determine θ and β using CA Amendment equations

$$\theta = 29 + 7000\varepsilon_x$$
$$\beta = \frac{4.8}{1+1500\varepsilon_x}$$
$$where\ \varepsilon_x = \frac{\left(\frac{M_u}{d_v} + 0.5V_u \cot\theta\right)}{2\left(E_s A_s\right)}$$

Assume 0.5cotθ ≅ 1.0.

Use absolute values for M_u and V_u.

$$A_s = 22\,bars \times 1.56\frac{in^2}{bar} = 34.32\,in^2$$

$$\varepsilon_x = \frac{\left(\frac{15021k - ft \times 12\,\frac{in}{ft}}{67.05\,in} + 1.0 \times 2001\,kips\right)}{2\left(29000\,ksi \times 34.32\,in^2\right)} = 0.002356$$

$$\theta = 29 + 7000\left(0.002356\right) = 45.49°$$
$$\beta = \frac{4.8}{1+1500 \times 0.002356} = 1.06$$

- Compute required stirrup spacing

Concrete contribution –

$$\begin{aligned} V_c &= 0.0316\beta\sqrt{f'_c}\,b_v d_v \\ &= 0.0316 \times 1.06 \times \sqrt{4} \times 96\,in \times 67.05\,in = 431\,kips \end{aligned}$$

Demand on stirrups –

$$V_{s,req} = \frac{V_u}{\phi} - V_c$$

$$= \frac{2001\,kips}{0.9} - 431\,kips = 1792\,kips$$

Required shear stirrups –

$$\left[V_{s,provided} = \frac{A_v f_y d_v}{s} \cot\theta \right] \geq V_{s,req}$$

$$\left(\frac{A_v}{s} \right)_{req} \geq \left[\frac{V_{s,req}}{f_y d_v \cot\theta} = \frac{1792\,kips}{60\,ksi \times 67.05\,in \times \cot 45.49°} = 0.453 \frac{in^2}{in} \right]$$

Minimum reinforcement and spacing –

$$\left(\frac{A_v}{s} \right)_{\min} = 0.0316 \frac{\sqrt{f'_c}}{f_y} b_v = 0.0316 \times \frac{\sqrt{4\,ksi}}{60\,ksi} \times 96\,in = 0.10 \frac{in^2}{in}$$

$$\left[\left(\frac{A_v}{s} \right)_{req} = 0.456 \frac{in^2}{in} \right] \geq \left[\left(\frac{A_v}{s} \right)_{\min} = 0.10 \frac{in^2}{in} \right]$$

Provided shear stirrups –

As shown in Figure 16 in the joint shear calculations section, the shear reinforcement in this region of maximum shear consists of 6-legged, #6 stirrups @ 8 in c/c.

$$\left[\left(\frac{A_v}{s} \right)_{provided} = \frac{6\,legs \times 0.44 \frac{in^2}{bar}}{8\,in} = 0.33 \frac{in^2}{in} \right] \leq \left[\left(\frac{A_v}{s} \right)_{req} = 0.453 \frac{in^2}{in} \right] \quad \therefore NG!$$

Since the provided shear reinforcement is not sufficient, either increase the area per stirrup or decrease the spacing between stirrups. The longitudinal reinforcement in the bent cap can be increased as well which will decrease the demand on the stirrups. In this case, we will decrease the stirrup spacing from 8 in to 5 in.

Use 6-legged, #6 stirrups @ 5 in c/c:

$$\left[\left(\frac{A_v}{s}\right)_{provided} = \frac{6\,legs \times 0.44\,\frac{in^2}{bar}}{5\,in} = 0.53\frac{in^2}{in}\right] \geq \left[\left(\frac{A_v}{s}\right)_{req} = 0.453\frac{in^2}{in}\right] \quad \therefore OK!$$

Maximum stirrup spacing –

$$\begin{aligned} &If\ \ v_u < 0.125\,f_c^{'},\ then:\\ &\quad s_{max} = 0.8d_v \leq 24.0\ in.\\ &If\ \ v_u \geq 0.125\,f_c^{'},\ then:\\ &\quad s_{max} = 0.4d_v \leq 12.0\ in. \end{aligned}$$

$$v_u = \frac{V_u}{\phi b_v d_v} = \frac{2001\,kips}{0.9 \times 96\,in \times 67.05\,in} = 0.345\,ksi$$

$$\left[0.125\,f_c^{'} = 0.125 \times 4.0\,ksi = 0.5\,ksi\right] \geq \left[v_u = 0.345\,ksi\right]$$

$$\therefore s_{max} = Lesser\ of \begin{Bmatrix} 0.8d_v = 0.8 \times 67.05\,in = 53.64\,in \\ 24.0\,in \end{Bmatrix} = 24.0\,in$$

$$\left[s_{prov} = 5.0\,in\right] \leq \left[s_{max} = 24.0\,in\right] \qquad \therefore OK!$$

Note these 6-legged #6 stirrups can also be used to satisfy joint shear reinforcement requirements. The required spacing as determined from joint shear criteria will be calculated in a later section and compared to the spacing of 5 in. calculated above. The smaller spacing governs the seismic design at location.

II.D. Longitudinal Pushover Analysis and Design

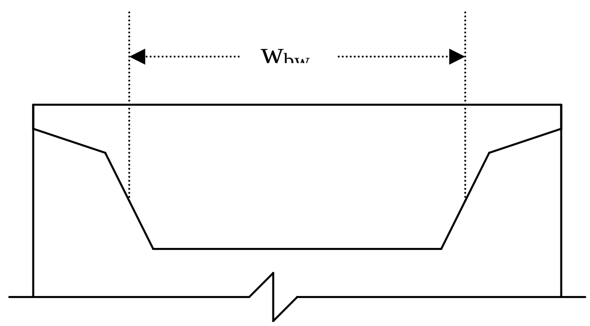
II.D.1. Abutment Soil Springs

This prototype bridge is supported on seat type abutments. It is Caltrans design practice to design the abutment backwall so that it breaks off in shear during the seismic event. For this bridge, the abutment is assumed to provide no contribution to the Earthquake Resisting System. In other words, the columns provide all the resistance to the seismic loads without any contribution from the abutments in either direction. Per LRFD Section 5.2.3.2, this is a Case 1 situation and the abutment soil springs will be excluded from the longitudinal analysis. This ensures that the columns will be able to resist the lateral seismic loads.

As a check for the previous assumption, the Caltrans SDC is used to model the soil springs and to determine the effectiveness of the abutment in resisting seismic loads. The linear-elastic abutment model is based upon the effective stiffness that accounts for expansion gap and incorporates realistic values for the embankment fill response. The abutment embankment fill stiffness is non-linear and is highly dependent upon the properties of the backfill. SDC Section 7.8 describes the procedure of modeling longitudinal stiffness of abutments.

For our case of seat type abutment, the effective area, A_e is given as:

$$A_e = h_{bw} \times w_{bw}$$



where
$h_{bw} =$ Back wall height
$w_{bw} =$ Superstructure width.

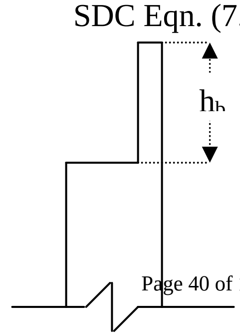
For our case,
$$A_e = 6.75 \times 46.46 = 313.6 \ ft^2$$

As per SDC Figure 7.14C, the effective abutment width is taken as average normal width of the superstructure.

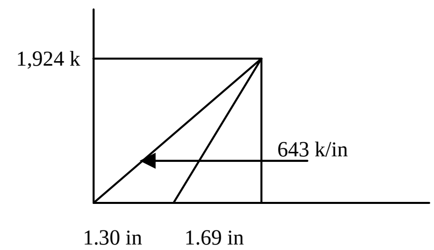
The maximum passive pressure, P_w , resisting the abutment is given as

$$P_w = A_e \times 5 \ ksf \times \left(\frac{h_{bw}}{5.5}\right) kips$$ SDC Eqn. (7.44)

$$= 313.6 \times 5 \times \left(\frac{6.75}{5.5}\right) = 1{,}924 \ kips$$



Based upon initial embankment fill stiffness, $K_i \approx 20\left(\frac{kips/in}{ft}\right)$, initial abutment stiffness is adjusted proportional to the backwall height as:



$$K_{abut} = K_i \times w \times \left(\frac{h}{5.5}\right)$$

$$K_{abut} = 20 \times 46.46 \times \left(\frac{6.75}{5.5}\right) = 1{,}140\ \frac{k}{in}$$

$$\Delta = \frac{F}{K} = \frac{1{,}924}{1{,}140} = 1.69\ in$$

$\Delta_{effective} = \Delta + \Delta_{gap} = 1.69 + 1.30 = 2.99\ in.$ See Appendix O for calculations for Δ_{gap}, the combined effect of thermal movement and anticipated shortening. Average contributory length is used for this purpose.

$$K_{initial}^{Abut} = \frac{1{,}924}{2.99} = 643\ kips/in$$

This value is used as the starting abutment stiffness for the longitudinal push over analysis. The Appendix P lists *wFRAME* input file. When the structure has reached its plastic limit state, we calculate the longitudinal bridge stiffness as

$k_{long} = \frac{0.38 \times 8,430}{9.05} = 353\ kips/in.$ See Appendix Q1.

Mass, $m = \frac{W}{g} = \frac{8{,}430}{32.2 \times 12} = 21.82\ kips - s^2/in$

$$T = 2 \times \pi \times \sqrt{\frac{m}{k_{long}}} = 2 \times \pi \times \sqrt{\frac{21.82}{353}} = 1.56\ \sec$$

$$S_a = 0.62g$$

$$\Delta_D = \frac{F}{K} = \frac{m \times a}{K} = \frac{21.82 \times 0.62 \times 32.2 \times 12}{353} = 14.81\ in$$

$$R_A = \frac{\Delta_D}{\Delta_{effective}} = \frac{14.81}{2.99} = 4.95$$

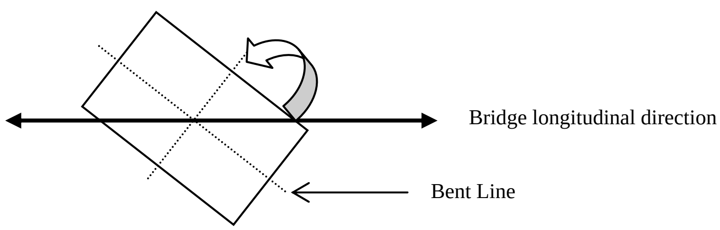
According to SDC Sec. 7.8.1, since $R_A > 4$, the elastic model is insensitive to the abutment stiffness. The abutment contribution to the overall bridge response is small and the abutments are insignificant to the longitudinal seismic performance of the structure. This confirms are previous assumption of not including the abutment response in the ERS. Consequently, subsequent longitidunal analysis will assume zero stiffness for the abutments per LRFD Section 5.2.3.2.

II.D.2. Ductility Analysis

II.D.2.i. Ductility Capacity and Ductility Demand Check

Although the process of calculating the section capacity and the estimated seismic demand is similar to that for the transverse direction, there are some significant differences:

- Columns are lumped together.
- Because superstructure is prestressed, gross moment of inertia is used for the superstructure.
- Bent overturning is ignored.
- The abutment contribution is excluded from the analysis as previously discussed.
- The calculations for determining section capacity for the longitudinal bending are similar because the columns have circular cros-section. If the section were rectangular, section properties along the longitudinal direction of the bridge must be calculated and used. This can be achieved by specifying, in *xSECTION* input file, the angle between the column section coordinate system and the longitudinal direction of the bridge as shown below.



Both left and right push over analyses are performed. The yield displacements of Bent 2 and Bent 3 are determined to be:

Location	Yield Disp. (Right Push)	Yield Disp. (Left Push)
Bent 2	8.79 in (CT 8.86)	8.29 in (CT 8.35)
Bent 3	9.02 in (CT 9.10)	9.75 in (CT 9.82)

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.

$\Delta_p = 28.50\,in$ for Bent 2 (CT 22.15)

and

$\Delta_p = 32.13\,in$ for Bent 3 (CT 24.93)

Now with $\Delta_c = \Delta_Y + \Delta_P$

For Bent 2

$$Min \quad \mu_c = \frac{\Delta_c}{\Delta_Y} = \left(\frac{8.79 + 28.50}{8.79}\right) = 4.24 > 3 \qquad \text{SDC Sec. 3.1.4} \qquad \text{OK.}$$

Similarly, for Bent 3 Column,

$$Min \quad \mu_c = \frac{\Delta_c}{\Delta_Y} = \left(\frac{9.75 + 32.13}{9.75}\right) = 4.30 > 3 \qquad \text{SDC Sec. 3.1.4} \qquad \text{OK.}$$

Appendix Q2 lists force-displacement relationship from *wFRAME* analysis. The bridge (frame) longitudinal stiffness is calculated from this plot when both columns have yielded. This stage represents the collapse mechanism.

$$k_{long} = \frac{0.153 \times 8,430}{9.02} = 143 \, kips / in.$$

$$T = 2 \times \pi \times \sqrt{\frac{21.82}{143}} = 2.45 \sec \qquad (CT \quad 2.2)$$

for which $S_a = 0.40g \quad (CT \quad 0.31g)$

$$\Delta_D = \frac{21.82 \times 0.40 \times 32.2 \times 12}{143} = 23.58 \, in \qquad (CT \quad 15.11)$$

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, which can result from rotational and/or translational foundation movements.

<u>Check Displacement Ductility</u>

Bent 2:

$$Max \, \Delta_{pd} = \Delta_D - \Delta_Y^{system} = 23.58 - 8.29 = 15.29 \, in$$

$$Max \, \mu_D = 1 + \frac{\Delta_{pd}}{\Delta_Y^{column}} = 1 + \frac{15.29}{7.25} = 3.11 \le 8.0 \qquad OK. \qquad (CT \;\; 1.8 \le 5)$$

Bent 3:

$$Max \, \Delta_{pd} = \Delta_D - \Delta_Y^{system} = 23.58 - 9.02 = 14.56 \, in$$

$$Max \, \mu_D = 1 + \frac{\Delta_{pd}}{\Delta_Y^{column}} = 1 + \frac{14.56}{8.27} = 2.76 \le 8.0 \qquad OK. \qquad (CT \;\; 1.6 \le 5)$$

II.D.2.ii. Check $P-\Delta$

For Bent 2 Columns

$$\frac{P_{dl}\times\Delta_r}{M_p}=\frac{1,694\times 23.58}{(13,808\times 12)}=0.24<0.25 \qquad (CT \quad 0.15<0.20) \qquad \text{OK}$$

For Bent 3 Columns

$$\frac{P_{dl}\times\Delta_r}{M_p}=\frac{1,653\times 23.58}{(13,747\times 12)}=0.23<0.25 \qquad (CT \quad 0.15<0.20) \qquad \text{OK}$$

II.D.2.iii. Minimum Lateral Strength

Per LRFD Section 8.7.1, the minimum lateral strength for a column is $0.1P_{DL}$. For a Bent 2 column, $0.1P_{dl}=0.1\times 1694=169\ kips$. Likewise, for a Bent 3 column, $0.1P_{dl}=0.1\times 1653=165\ kips$. The lateral strength, as read from Appendix Q2, for the bridge is $0.151g\times M$ (CT 0.19g), or $0.151g\times\left(\frac{8430}{g}\right)=1273\ kips$. This results in a bent lateral strength of $\frac{1273\ kips}{2\ bents}=637\ kips$ and, subsequently, a column lateral strength of $\frac{673\ kips}{2\ columns}=337\ kips$. The lateral strength of the columns satisfy the minimums calculated above.

II.D.3. Column Shear Check

As per LRFD Section 8.6.1, the maximum shear demand corresponds to V_0 ., the shear corresponding to the overstrength moment. Note that wFRAME output numbers represent total shear for both columns at each bent. The column shear capacity is calculated following the procedures outlined in the transverse analysis.

Bent 2

$V_0=1.2\times V_P=388\ kips$ (CT 388). It corresponds to max shear value of V_P =323 kips/column obtained from the wFRAME push over analysis.

$$\left[\upsilon_c=\alpha'\left(1+\frac{P}{2000A_g}\right)\sqrt{f_c'}\right] \leq 3.5\sqrt{f_c'} \qquad \text{LRFD Eqn. (8.13)}$$

$$\alpha' = \frac{0.03}{\mu_D}\rho_s f_{yh}$$ LRFD Eqn. (8.16)

As previously calculated, $\rho_s = 0.009451$

From the longitudinal pushover analysis results, the displacement ductility, $\mu_d = 3.11$.

Using $f_{yh} = 60\ ksi$,

$$\alpha' = \frac{0.03}{3.11}\times(0.009451)\times(60000) = 5.47$$

Similarly,

$$1+\frac{P}{2,000\times A_g} = 1+\frac{1694\times10^3}{2,000\times\frac{\pi}{4}\times(6\times12)^2} = 1.21$$

The maximum allowable concrete shear stress is calculated as

$$\left[\upsilon_c = 5.47\times1.21\times\sqrt{4,000} = 419\ psi\right] > \left[3.5\sqrt{4,000} = 221\ psi\right]$$

Use $\upsilon_c = 221\ psi \qquad (CT \quad 230\ psi)$.

$$\therefore V_c = \frac{221\times0.8\times\frac{\pi}{4}\times(6\times12)^2}{1,000} = 720\ kips \qquad (CT \quad 749\ kips)$$

V_s = 996 kips, which was calculated previously for the transverse analysis.

$$\left[\phi\times V_n = 0.85\times(720+996) = 1,459\ kips\right] > \left[V_0 = 388\ kips\right].$$ OK.

$$\left(CT: \left[\phi\times V_n = 1,483\ kips\right] > \left[V_0 = 388\ kips\right]\right)$$

Bent 3

$V_0 = 1.2\times V_P = 378\ kips\ (CT \ 378)$. It corresponds to max shear value of 315 kips/column obtained from the wFRAME push over analysis.

Similar to the previous calculations, V_c = 720 kips and V_s = 996 kips.

$$\left[\phi\times V_n = 0.85\times(720+996) = 1,459\ kips\right] > \left[V_0 = 378\ kips\right].$$ OK.

$$\left(CT: \left[\phi\times V_n = 1,478\ kips\right] > \left[V_0 = 378\ kips\right]\right)$$

II.D.4. Seismic Strength of Concrete Bridge Superstructures

We often seem to forget that 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 can occur in the superstructure leading to potential failure of the superstructure. It is understood from LRFD Section 4.11.1 and 8.10 that a capacity design approach is adopted to ensure that the superstructure has an appropriate strength reserve above demands generated from probable column plastic hinging. Though the LRFD does not specify an explicit procedure, Caltrans Memo to Designers (MTD) 20-6 describes a procedure for determining the seismic demands in the superstructure that implements this capacity design philosophy. It also recommends a method for determining the flexural capacity of the superstructure at all critical locations. The Caltrans procedure will be applied herein supplemented by corresponding LRFD guidelines where appropriate.

II.D.4.i. General Assumptions

As discussed in MTD 20-6, the following are some of the assumptions that are made for simplifying the process of calculating the seismic demands in the superstructure:

- 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 prestressed superstructure elements.
- Because of the uncertain magnitude and distribution of secondary presress, the effects of secondary prestress shall only be included when it results in increased demands in the superstructure.
- Superstructure dead load and secondary prestress demands are assumed to be uniformly distributed to each girder, except in case of highly curved or highly skewed structures.
- While assessing the superstructure member demands and available section capacities, an effective width as defined in the LRFD Section 8.10 will be used.

II.D.4.ii. Determining Seismic Demand in the Superstructure

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 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 Prestress Loads
- Seismic Loads

It should be noted that 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 to determine extreme seismic demands in the superstructure considers each of these load cases separately and the final member demand is obtained by the superposition of the individual load cases. Since we shall be using different tools to calculate these demands, it is *very important* to use a consistent sign convention while interpreting results. We shall adopt the following sign convention for positive moments and positive shears. The *CTBridge* program uses a similar sign convention. It should be kept in mind that the *wFRAME* program uses a sign convention that is different to this adopted sign convention.

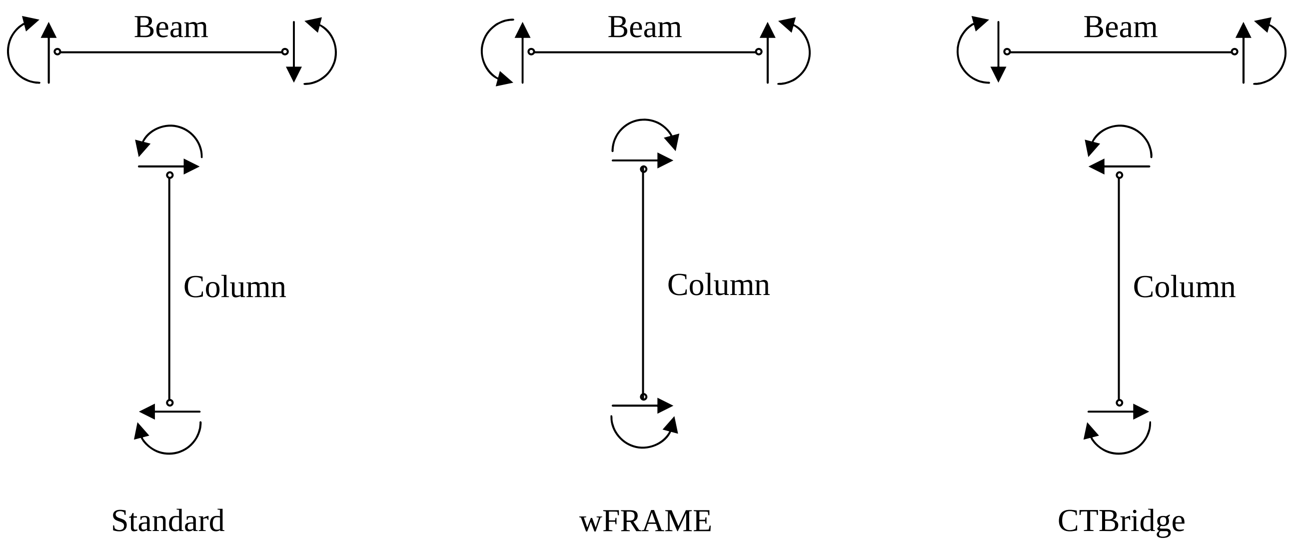


Figure 4 Sign Convention for Positive Moment and Positive Shear for Various Programs

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 upon the direction of these “pre-load” moments and the direction of the seismic loading under consideration.

The load and secondary prestress effects to reach its overstrength moment capacity of the columns. Figure 5 shows schematically this approach of calculating columns seismic forces.

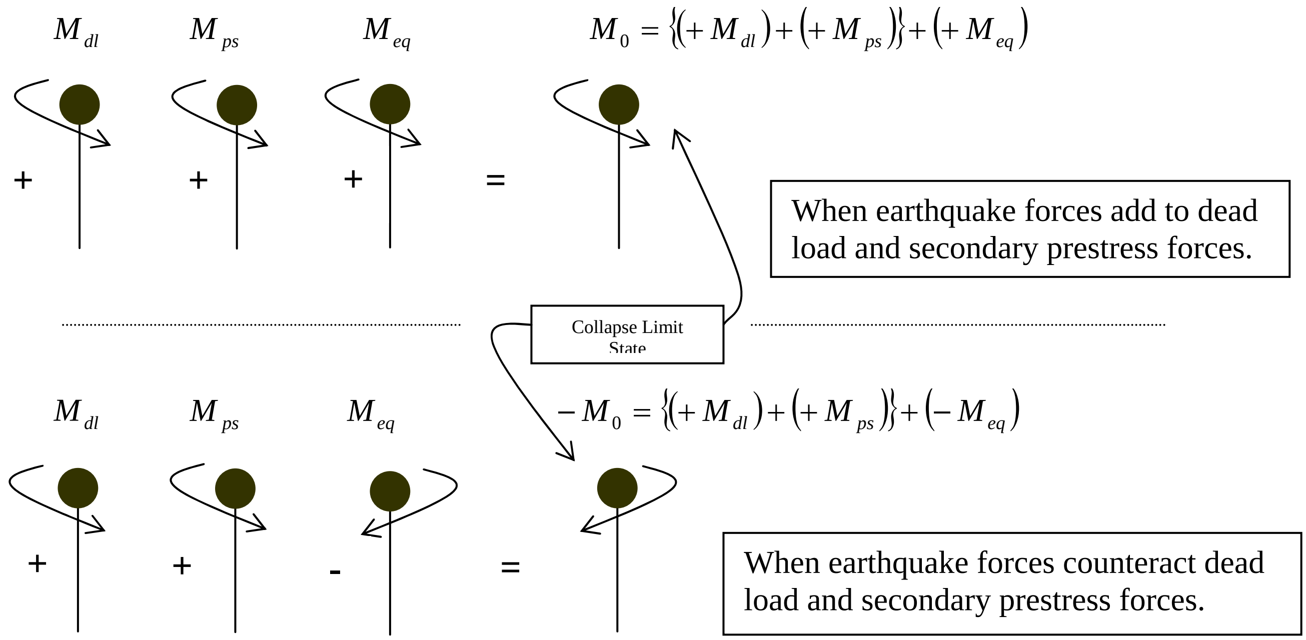
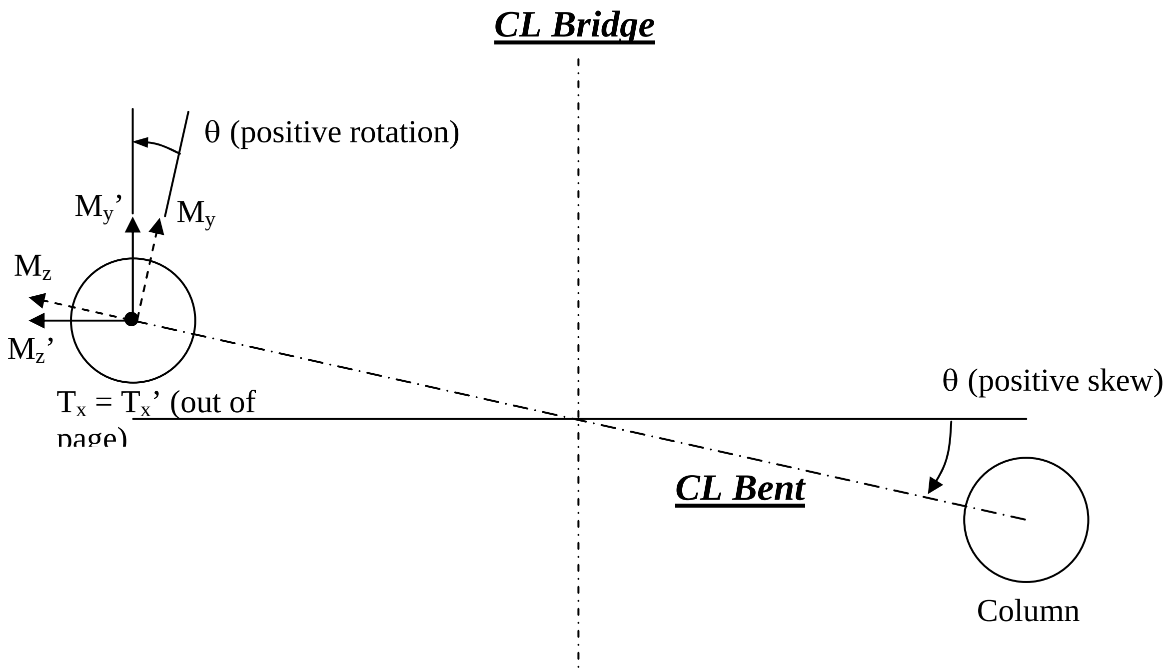


Figure 5 Column Forces Corresponding to Two Seismic Loading Cases

Once the column moment, M_{eq}, is known at each potential plastic hinge location below the joint regions, the seismic moment demand in the superstructure can be determined using currently available Caltrans analysis tools by either of the following approaches. In the first method, these moments are applied at the column-superstructure joints and the *SAP2000* program can be used to compute the moment demand in the superstructure members. The second method involves using the *wFRAME* program to perform a longitudinal push over analysis by specifying the required seismic moments in the columns as the plastic hinge capacities of the column ends which are moment-connected to the superstructure. The push over is continued until all the plastic hinges have been formed. In our case, we shall use this method to compute the distribution of seismic moments in the superstructure members.

Note that *CTBridge* is a three-dimensional analysis program where force results are oriented in the direction of each member's local axis. Since we will be using *wFRAME,* a two-dimensional frame analysis program, to determine the distribution of seismic forces to the superstructure, we need to make sure 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. Consequently for this prototype bridge, the top of bent support results from *CTBridge* will need to be transformed to a consistent planar coordinate system (i.e. the plane formed by the centerline of bridge and the vertical axis). To do so, the following coordinate transformation will be applied to the top of column moments from *CTBridge*:



$M_z' = M_z \cos\theta - M_y \sin\theta$ (Longitudinal Moment)
$M_y' = M_z \sin\theta + M_y \cos\theta$ (Transverse Moment)
$T_x' = T_x$ (Torsional Moment)

Bent	Location	Skew	DL			ADL			Sec. PS		
			M_z	M_y	M_{long}	M_z	M_y	M_{long}	M_z	M_y	M_{long}
2	**Soffit**	20	-1189	91	-1148	-213	17	-206	82	-371	204
3	**Soffit**	20	1305	-1	1227	234	-1	220	-127	287	-218

It should be kept in mind that the above values are for both columns in each bent.

As recommended in MTD 20-6, due to the uncertainty of 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.

Now these methodologies are applied to our bridge to calculate the extreme seismic forces in the superstructure corresponding to the Collapse Limit State of the bridge.

II.D.4.iii. Determine Dead Load and Additional Dead Load Moments

These dead load moments are readily available from the *CTBridge* output. Table 1.1 lists these moments at every 1/10th point of the span length and at the face of supports. These moments are assumed to be uniformly distributed along each girder.

<u>At Bent 2</u>

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

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

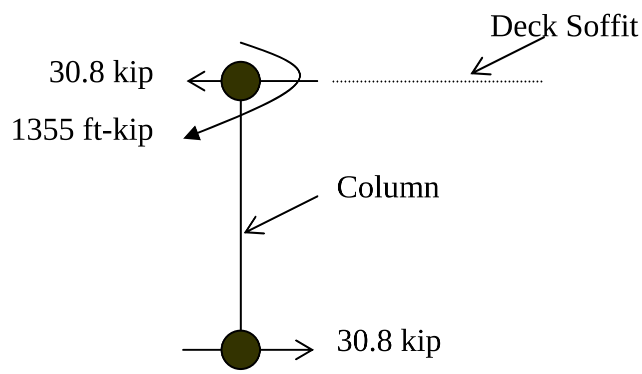


Figure 6 Free Body Diagram Showing Equilibrium of Dead Loading at Bent 2

At Bent 3

Column moment at base, $M_{dl}^{col,top@joint} = 0\ ft-kip$ (*CTBridge* Output)
Column moment at deck soffit, $M_{dl}^{col,top@joint} = \{(+1{,}227)+(+220)\} = +1{,}447\ ft-kip$

II.D.4.iv. Determine Prestress Secondary Moments

Once again, the secondary prestress moments are obtained directly from the *CTBridge* output. These moments are assumed to be uniformly distributed along each girder. Table 1.1 lists these moments at every 1/10th point of the span length and at the face of supports.

At Bent 2

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

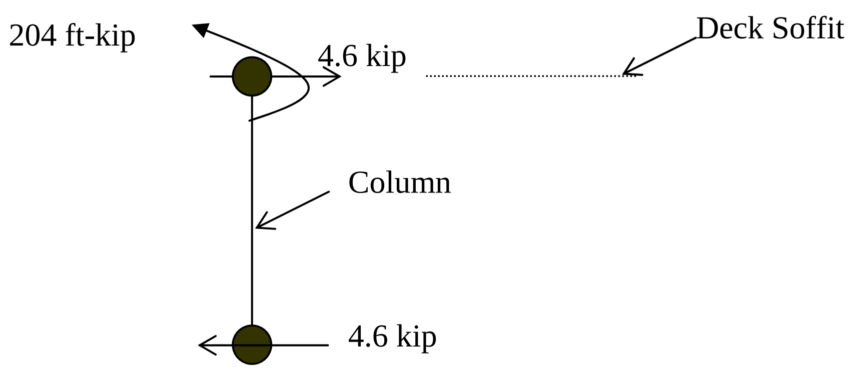


Figure 7 Free Body Diagram Showing Equilibrium of Secondary Prestress Forces at Bent 2

At Bent 3

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

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

II.D.4.v. Determine Earthquake Moments

II.4.D.v.a. Determine the seismic loading needed to ensure 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 an overstrength plastic moment capacity, M_{po}^{col}.

$$M_{po}^{col@soffit} = M_{dl}^{col@soffit} + M_{ps}^{col@soffit} + M_{eq}^{col@soffit}$$

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

The column seismic load moments, M_{eq}^{col}, are calculated from this equation based upon the principle of superposition as follows:

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

In these equations, the overstrength column moment as given as

$$M_{po}^{col} = 1.2 \times M_{p}^{col}$$ LRFD Section 4.11.2

Two cases of longitudinal earthquake loading are considered.

Case 1) The Bridge Moves from Abutment 1 towards Abutment 4

As shown in Figure 8, such loading results in positive moments in the columns according to the sign convention used here.

Bent 2

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

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

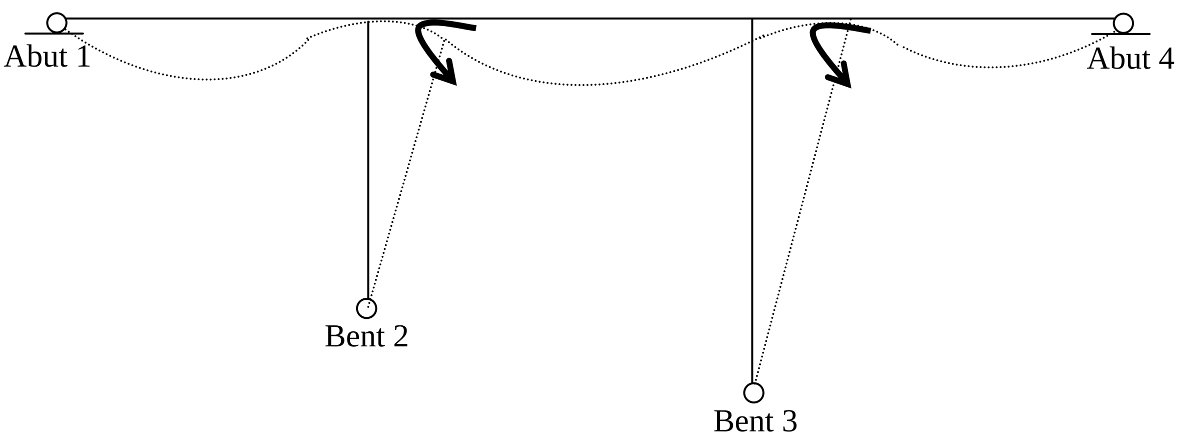


Figure 8 Seismic Loading Case "1" Producing Positive Moments in Columns

Now, the amount of column moment that will be generated by the seismic loading so that the column reaches its overstrength moment capacity will be

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

$$= 1.2 \times \left(2\,Columns\right) \times 13{,}808 - \{(-1{,}355) + 0)\} = +34{,}494\,ft - kip \qquad (CT \quad 34{,}566)$$

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. As recommended in MTD20-6 and discussed earlier, this practice is considered conservative because of the uncertainty associated with the magnitude and distribution of prestress secondary moments.

Figure 9 schematically explains this superposition approach.

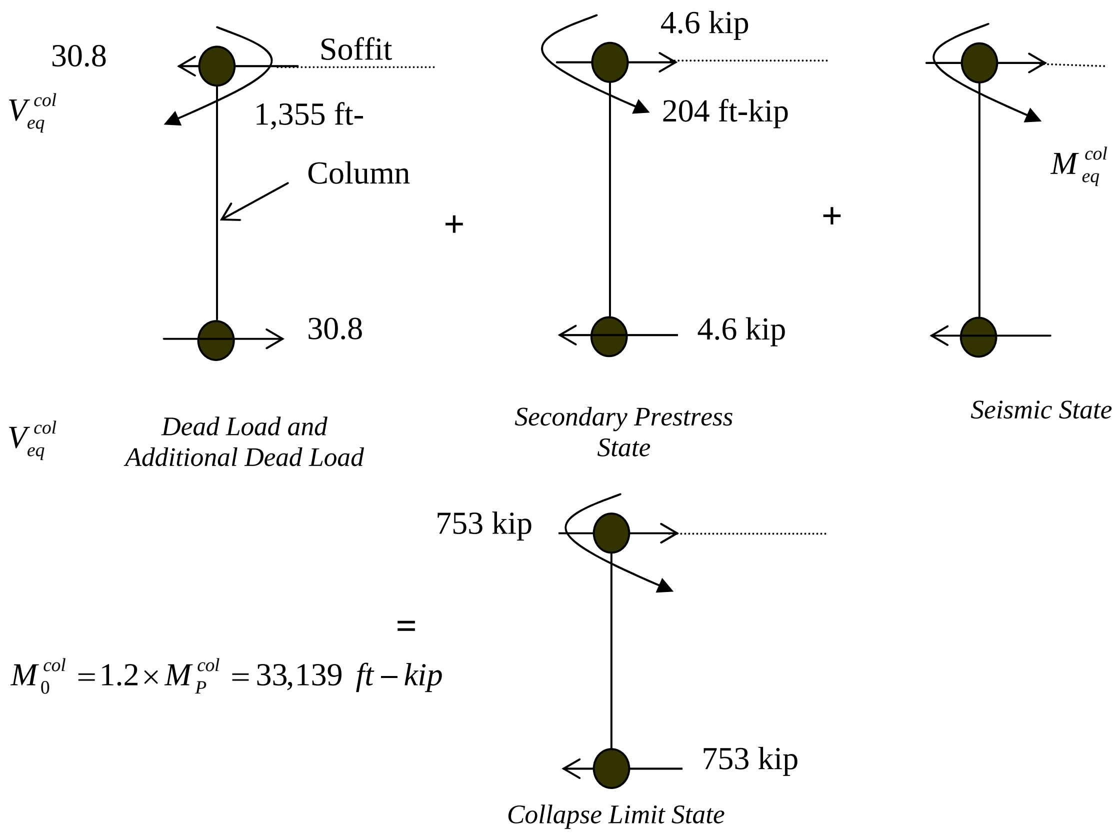


Figure 9 Superposition of Column Forces at Bent 2 for Loading Case "1"

<u>Bent 3</u>

Following a similar approach, the amount of column moment that will be generated by the seismic loading so that the column reaches its overstrength moment capacity will be

$$M_{eq}^{col@soffit} = 1.2 \times (2\ Columns) \times 13,747 - (1,447 - 218) = 31,764\ ft - kip \qquad (CT \quad 31,835)$$

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

Case 2) The Bridge Moves from Abutment 4 toward Abutment 1

As shown in Figure 10, such loading results in negative moments in the columns according to our sign convention.

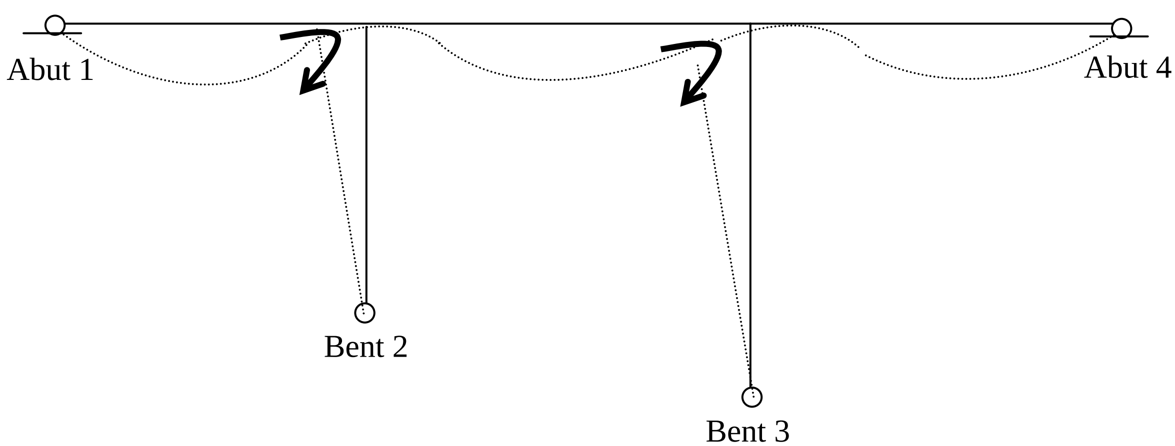


Figure 10 Seismic Loading Case "2" Producing Negative Moments in Columns

Following the same procedure as outlined earlier, the maximum column seismic moments at bents 2 and 3 are calculated to be

$$M_{eq}^{col@soffit} = 1.2 \times M_{p}^{col} - (M_{dl}^{col} + M_{ps}^{col}) = 1.2 \times \left(2\ Columns\right) \times (-13,808) - (-1,355 + 204) = -31,988\ ft-kip \qquad (CT \quad -32,060)$$

$$M_{eq}^{col@soffit} = 1.2 \times M_{p}^{col} - (M_{dl}^{col} + M_{ps}^{col}) = 1.2 \times \left(2\ Columns\right) \times (-13,747) - (1,447 - 0) = -34,440\ ft-kip \qquad (CT \quad -34,512)$$

respectively.

Please note the negative sign associated with the column overstrength moment capacity., indicating that the seismic loading being considered here produces negative column moments according to our sign convention.

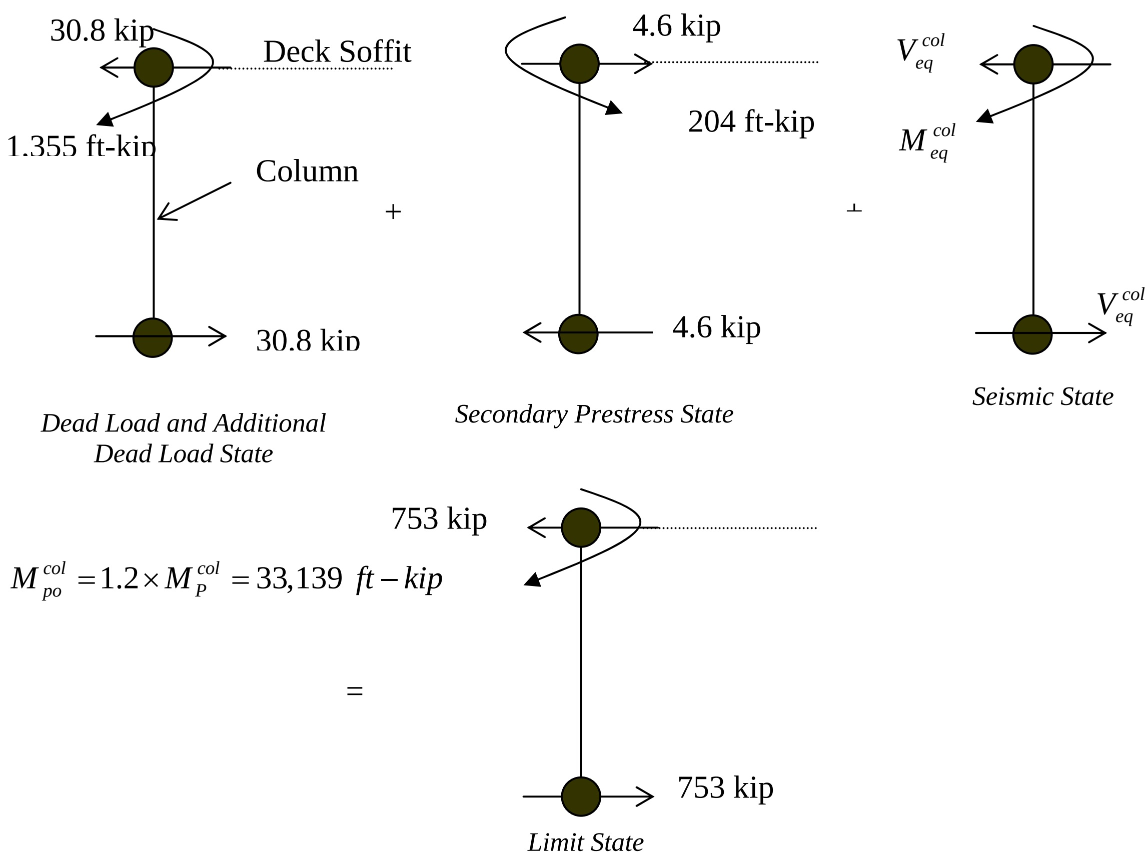


Figure 11 Superposition of Column Forces at Bent 2 for Loading Case "2"

Figure 11 schematically shows the Free Body Diagram at Bent 2 for this seismic loading case.

Now that we know the extreme seismic moments in columns, let us find distribution of these moments in the superstructure.

II.D.4.v.b. Determine the earthquake moment in the superstructure

The static non-linear "push-over" frame analysis program *wFRAME* will now be used to distribute the column earthquake moments and shears into the superstructure.
The sign convention for positive moment and shear forces used in *wFRAME* is opposite to the one being used here. Appendix R shows the input file to the *wFRAME* program. Note that the superstructure dead load has been removed from the *wFRAME* model. As can be seen from this input file the positive column earthquake moments corresponding to "Case 1" loading are used as negative column moment capacities for "push-over" analysis while the negative column earthquake moments corresponding to "Case 2" are modeled as positive column moment capacities.

Table 1.2 lists the distribution of earthquake moments in the superstructure as obtained from these "push-over" analyses. See Appendix S for Case 1 results in Table 1.2.

II.D.4.vi. Compute Moment and Shear Demand at Location of Interest

The extreme seismic moment demand in the superstructure is now calculated as the summation of all the moments calculated in steps 3.1 through 3.3, taking into account the proper direction of bending in each case as well as the effective section width. The superstructure demand moments are defined as

$$M_D^L = M_{dl}^L + M_{ps}^L + M_{eq}^L$$

and

$$M_D^R = M_{dl}^R + M_{ps}^R + M_{eq}^R$$

at the left and right sides of the column, respectively. Dead load and prestress moment demands in the superstructure are proportioned based upon the number of girders falling within the effective section width. The full earthquake moment imparted by the column is also assumed to act within the same effective section width.

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

<u>Bent 2: At the left face of Bent Cap</u>

The effective section width,

$$b_e = D_c + 2 \times D_s = 6.00 + 2 \times 6.75 = 19.50\, ft \qquad \text{LRFD Eqn. (8.36)}$$

Based upon 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.

The per girder values used below have been listed in Table 1.1.

Case 1)

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

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

Now the $M_{eq}^{L} = -14{,}946\, ft - kip \quad (CT\ -15{,}015)$. This value is listed in Table 1.2

The superstructure moment demand is then calculated as

$$M_{D}^{L} = (-30{,}736) + \left(\cancelto{0}{6{,}936^*}\right) + (-14{,}946) = -45{,}682\, ft - kip \quad (CT\ -45{,}751)$$

Similarly,

$$M_{D}^{R} = (-31{,}730) + (6{,}774) + (21{,}136) = -3{,}820\, ft - kip \quad (CT\ -3{,}821)$$

Table 1.3 lists these superstructure seismic moment demands.

Case 2)

$$M_{eq}^{L} = +13{,}136\, ft - kip \quad (CT\ +13{,}201)$$

The superstructure moment demand in this case becomes

$$M_{D}^{L} = (-30{,}736) + (6{,}936) + (13{,}136) = -10{,}664\, ft - kip \quad (CT\ -10{,}599)$$

The superstructure demand on the right side of the column is calculated to be

$$M_{D}^{R} = (-31{,}730) + \left(\cancelto{0}{6{,}774^*}\right) + (-20{,}293) = -52{,}023\, ft - kip \quad (CT\ -52{,}025)$$

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

The superstructure moment demands around Bent 3 are calculated to be:

$$M_D^L = \begin{cases} -49{,}703\,ft-kip \quad \text{Case 1} \qquad (CT \quad -49{,}702) \\ -2{,}996\,ft-kip \quad \text{Case 2} \qquad (CT \quad -3{,}002) \end{cases}$$

and

$$M_D^R = \begin{cases} -9{,}500\,ft-kip \quad \text{Case 1} \qquad (CT \quad -9{,}431) \\ -43{,}839\,ft-kip \quad \text{Case 2} \qquad (CT \quad -43{,}914) \end{cases}$$

The seismic moment demands along the superstructure length have been summarized in the form of moment envelope values tabulated in Table 1.3

Now a similar procedure can be followed to calculate the seismic shear force demand in the superstructure.

Once again the shear forces in the superstructure member due to dead load, additional dead load, and secondary prestress are readily available from the *CTBridge* output. Table 1.4 lists these values.

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. In our case, the values of V_{eq} for Span 1 are calculated to be:

<u>Case 1)</u>

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

Seismic Moment at Bent 2, $M_{eq}^{(2)} = -15{,}311\,ft-kip \qquad (CT \quad -15{,}381)$

Shear force in Span , $V_{eq} = \dfrac{\left(M_{eq}^{(2)} - M_{eq}^{(1)}\right)}{\text{Length of Span 1}} = \dfrac{\left(-15{,}311-0\right)}{126} = -122kip$

<u>Case 2)</u>

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

Seismic Moment at Bent 2, $M_{eq}^{(2)} = 13{,}456\,ft-kip \qquad (CT \quad 13{,}523)$

Shear force in Span , $V_{eq} = \frac{\left(M_{eq}^{(2)} - M_{eq}^{(1)}\right)}{\text{Length of Span 1}} = \frac{(13,456-0)}{126} = 107kip$

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

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

Span 3, $V_{eq} = \begin{cases} -115kip & \text{Case 1} \\ +132kip & \text{Case 2} \qquad (CT \quad +133) \end{cases}$

Table 1.5 lists these values. Once again, the extreme seismic shear force demand in the superstructure is now 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 are defined as $V_D^L = V_{dl}^L + V_{ps}^L + V_{eq}^L$ and $V_D^R = V_{dl}^R + V_{ps}^R + V_{eq}^R$ at the left and right side of the column, respectively. Once again, the effect due to the secondary prestress will be considered only when doing so results in increased seismic demand.

Table 1.6 lists the maximum shear demand summarized as a shear envelope.

II.D.4.vii. Superstructure Section Capacity

Now that we have calculated the extreme moment and shear seismic demands, let us calculate the corresponding section capacity to make sure that the superstructure has sufficient capacity to resist the demands.

II.D.4.vii.a. Superstructure Flexure Capacity

MTD 20-6 describes the philosophy behind the flexural section capacity calculations. The member strength and curvature capacities are assessed using a stress-stain compatibility analysis. Failure is reached when either the ultimate concrete, mild steel or prestressing ultimate strain is reached. Figure 12 shows such equilibrium:

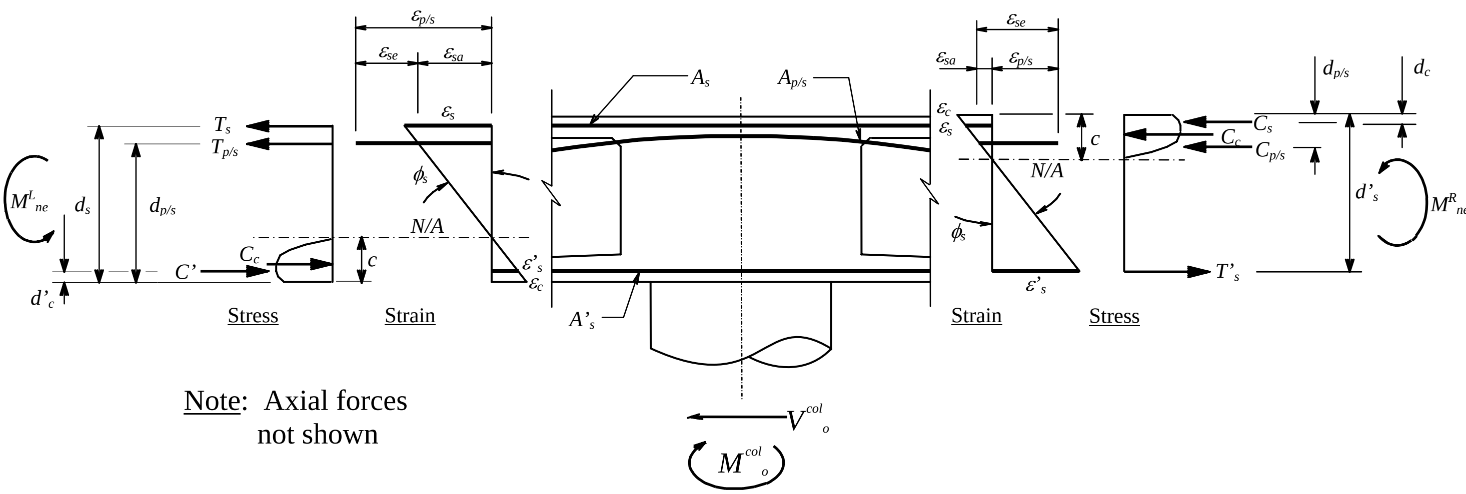


Figure 12 Superstructure Capacity Provided by Internal Couple

As stated in MTD20-6, the empirical relationships in AASHTO LRFD Section 5.7.3 do not accurately reflect prestress member strength or address the issue of bonded tendon ductility, and therefore, are not used in this example. The computer program *PSSECx* is used to calculate the section flexural capacity. The program has the option to use either a "simple" model or "Mander's unconfined" model to represent the unconfined concrete stress-strain relationship. The material properties used for 270ksi prestressing strands are given in LRFD Section 8.4.3. According to MTD20-6, at locations where additional longitudinal mild steel is not required by analysis, as a minimum, an equivalent of #8 @12 (maximum spacing not to exceed 12") should be placed in the top and bottom slabs at the bent cap. Such reinforcement will extend beyond the inflection points for the seismic moment demand envelope. For A706 reinforcing steel , the material properties are given in LRFD Section 8.4.2. As specified in LRFD Section 8.9, the expected nominal moment capacity, M_{ne}, for capacity protected concrete components shall be determined by $M-\phi$ analysis. Also, expected material properties are to be used. Expected nominal moment capacity for these capacity-protected concrete members shall be based upon the expected concrete and steel strengths when either the concrete strain reaches its ultimate value, ε_{cu} =0.005 (CT 0.003), based upon the stress-strain model or the reduced ultimate prestress steel strain, ε_{su}^{R} =0.04 (CT 0.03), as specified in LRFD Section 8.4.3. Besides these material properties, the following additional information also needs to be supplied:

<u>Prestressing Steel</u>

- Eccentricity of Prestressing Steel - Obtained from the *CTBridge* output file. This value is referenced from the CG of the section.

- Prestressing Force - Obtained from the *CTBridge* output file under the "P/S Response After Long Term Losses" tables.

- Prestressing Steel Area, A_{ps} — Calculated for 270ksi steel as

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

<u>Mild Steel</u>

- Amount of Top Slab Steel — Known as per design including #8 @12 that is put in a priori.
- Location of Top Slab Steel — Referenced from CG of section. Known from section depth, assumed cover, etc.

- Amount of Bottom Slab Steel — Known as per design including #8 @12 that is put in a priori.
- Location of Bottom Slab Steel — Referenced from CG of section. Known from section depth, assumed cover, etc.

Table 1.7 lists these data that will be used to calculate the flexural section capacity.

The computer program *PSSECx* was run repeatedly to calculate superstructure flexural capacities at various points along the span length. Both negative (tension at the top) and positive (tension at the bottom) capacities were calculated at various sections along the length of the bridge. Table 1.8 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. It is suggested that $\phi_{flexure} = 1.0$ be used as we are dealing with extreme conditions corresponding to column overstrength.

Appendix T lists the *PSSECx* input for the superstructure section that lies just left of Bent 2. The model is shown in Appendix U. The results for negative capacity calculations are shown in Appendix W. As stated earlier, the flexural capacity is determined when either the steel or concrete strain reaches its respective maximum value. In this case, the maximum allowable value of steel strain is reached before concrete reaches its maximum. The worst ratio 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 @12 may be revised in the future.

II.D.4.vii.b. Superstructure Shear Capacity

Similar to the bent caps, the superstructure shear capacity should be calculated as per AASHTO LRFD. As shear failure is a brittle failure, nominal rather than expected material properties are used to calculate the shear capacity of the superstructure using $\phi_{shear} = 0.90$. Table 1.9 compares the seismic shear demands with the available section shear capacity as determined from *CTBridge* for strength and service demands.

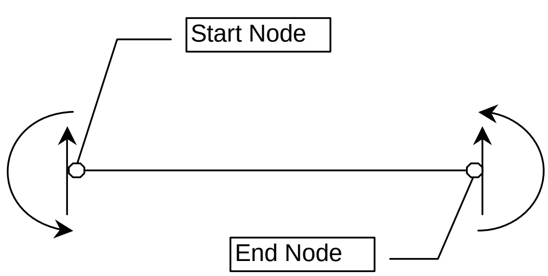
Table 1.1 Dead Load and Secondary Prestress Momnets

Moments (k-ft) from CTBridge Output

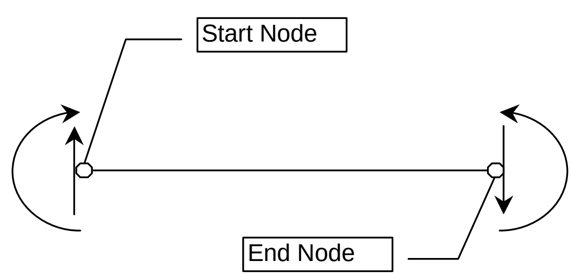
	Location		Whole Superstructure Width			Per Girder		
			M_{DL}	M_{ADL}	M_{PS}	M_{DL}	M_{ADL}	M_{PS}
Span 1	**Support**	1.5	619	114	647	124	23	129
	0.1	12.6	7110	1275	1462	1422	255	292
	0.2	25.2	12158	2178	2272	2432	436	454
	0.3	37.8	14741	2640	3096	2948	528	619
	0.4	50.4	14857	2661	3956	2971	532	791
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

Table 1.2 Earthquake Moments

Earthquake Moments (k-ft) from wFRAME Output						
Location		M_{EQ}			
		wFRAME Convention		Standard Convention	
		Case 1	Case 2	Case 1	Case 2
Span 1	**0.0**	0	0	0	0
	Support			-182	160
	0.1			-1531	1346
	0.2			-3062	2691
	0.3			-4593	4037
	0.4			-6124	5382
	0.5			-7656	6728
	0.6			-9187	8074
	0.7			-10718	9419
	0.8			-12249	10765
	0.9			-13780	12110
	Support			-14946	13136
	1.0	-15311	13456	-15311	13456
Span 2	**0.0**	-21896	21053	21896	-21053
	Support			21136	-20293
	0.1			17641	-16795
	0.2			13386	-12536
	0.3			9131	-8278
	0.4			4876	-4020
	0.5			621	239
	0.6			-3635	4497
	0.7			-7890	8755
	0.8			-12145	13013
	0.9			-16400	17272
	Support			-19895	20770
	1.0	-20655	21530	-20655	21530
Span 3	**0.0**	-13549	15544	13549	-15544
	Support			13205	-15149
	0.1			12194	-13990
	0.2			10839	-12435
	0.3			9484	-10881
	0.4			8129	-9326
	0.5			6775	-7772
	0.6			5420	-6218
	0.7			4065	-4663
	0.8			2710	-3109
	0.9			1355	-1554
	Support			172	-198
	1.0	0	0	0	0



wFRAME Positive Convention



Standard Positive Convention

Table 1.3 Moment Demand Envelope

Moment Demand (k-ft) Envelope

	Location		No. of Girders in Effective Section	M_{DL}	M_{ADL}	M_{PS}	Case 1 M_{EQ}	Case 2 M_{EQ}	Case 1 $M_{positive}$	Case 1 $M_{negative}$	Case 2 $M_{positive}$	Case 2 $M_{negative}$	Envelope $M_{positive}$	Envelope $M_{negative}$
Span 1	**Support**	1.5	4	496	91	517	-182	160	922	404	1264	747	1264	404
	0.1	12.6	5	7110	1275	1462	-1531	1346	8316	6854	11192	9731	11192	6854
	0.2	25.2	5	12158	2178	2272	-3062	2691	13546	11274	19299	17028	19299	11274
	0.3	37.8	5	14741	2640	3096	-4593	4037	15883	12788	24513	21418	24513	12788
	0.4	50.4	5	14857	2661	3956	-6124	5382	15349	11393	26856	22900	26856	11393
	0.5	63.0	5	12508	2240	4705	-7656	6728	11797	7092	26180	21476	26180	7092
	0.6	75.6	5	7693	1377	5617	-9187	8074	5501	-116	22761	17144	22761	-116
	0.7	88.2	5	412	74	6400	-10718	9419	-3832	-10232	16305	9905	16305	-10232
	0.8	100.8	5	-9334	-1671	7911	-12249	10765	-15343	-23254	7671	-240	7671	-23254
	0.9	113.4	5	-21553	-3857	8498	-13780	12110	-30692	-39190	-4802	-13300	-4802	-39190
	Support	123.0	4	-26079	-4656	6937	-14946	13136	-38744	-45681	-10662	-17599	-10662	-45681
Span 2	**Support**	129.0	4	-26923	-4807	6774	21136	-20293	-3819	-10594	-45248	-52023	-3819	-52023
	0.1	142.8	5	-17502	-3136	9516	17641	-16795	6519	-2998	-27917	-37433	6519	-37433
	0.2	159.6	5	-1955	-354	9005	13386	-12536	20083	11078	-5840	-14845	20083	-14845
	0.3	176.4	5	9208	1645	8318	9131	-8278	28302	19984	10893	2575	28302	2575
	0.4	193.2	5	15989	2859	8281	4876	-4020	32005	23724	23109	14828	32005	14828
	0.5	210.0	5	18388	3289	8027	621	239	30324	22297	29942	21915	30324	21915
	0.6	226.8	5	16406	2935	8072	-3635	4497	23778	15706	31910	23838	31910	15706
	0.7	243.6	5	10043	1795	7905	-7890	8755	11854	3949	28498	20594	28498	3949
	0.8	260.4	5	-699	-128	8355	-12145	13013	-4617	-12971	20542	12187	20542	-12971
	0.9	277.2	5	-15820	-2835	8645	-16400	17272	-26410	-35055	7262	-1384	7262	-35055
	Support	291.0	4	-25291	-4517	6043	-19895	20770	-43661	-49703	-2996	-9039	-2996	-49703
Span 3	**Support**	297.0	4	-24344	-4347	5986	13205	-15149	-9500	-15486	-37854	-43839	-9500	-43839
	0.1	305.8	5	-20789	-3723	7275	12194	-13990	-5043	-12318	-31227	-38502	-5043	-38502
	0.2	317.6	5	-9854	-1766	6861	10839	-12435	6081	-781	-17194	-24055	6081	-24055
	0.3	329.4	5	-1093	-197	5559	9484	-10881	13754	8194	-6611	-12171	13754	-12171
	0.4	341.2	5	5506	986	4870	8129	-9326	19490	14621	2034	-2835	19490	-2835
	0.5	353.0	5	9943	1781	4085	6775	-7772	22583	18498	8037	3952	22583	3952
	0.6	364.8	5	12219	2189	3417	5420	-6218	23244	19827	11607	8190	23244	8190
	0.7	376.6	5	12333	2210	2669	4065	-4663	21277	18608	12549	9880	21277	9880
	0.8	388.4	5	10286	1844	1945	2710	-3109	16784	14840	10966	9021	16784	9021
	0.9	400.2	5	6077	1091	1230	1355	-1554	9753	8523	6844	5614	9753	5614
	Support	410.5	4	509	94	423	172	-198	1198	775	828	405	1198	405

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

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

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

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

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

Table 1.4 Dead Load and Secondary Prestress Shear Forces

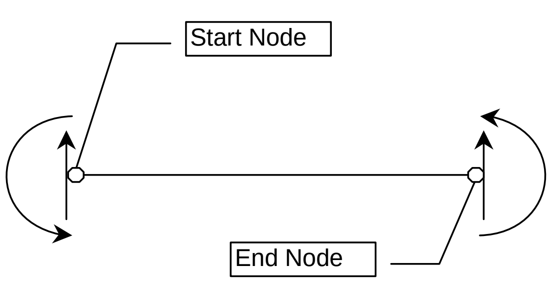
Shear (k) from CTBridge Output

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

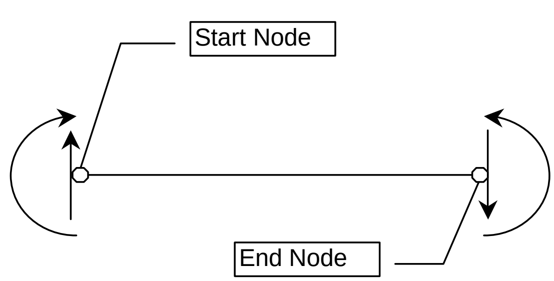
Table 1.5 Earthquake Shear Forces

Earthquake Shear (k) from wFRAME Output

	Location		V_{EQ} wFRAME Convention Case 1	wFRAME Convention Case 2	Standard Convention Case 1	Standard Convention Case 2
Span 1	**0**	0.0	-122	107	-122	107
	Support	1.5	0	0	-122	107
	0.1	12.6	0	0	-122	107
	0.2	25.2	0	0	-122	107
	0.3	37.8	0	0	-122	107
	0.4	50.4	0	0	-122	107
	0.5	63.0	0	0	-122	107
	0.6	75.6	0	0	-122	107
	0.7	88.2	0	0	-122	107
	0.8	100.8	0	0	-122	107
	0.9	113.4	0	0	-122	107
	Support	123.0	0	0	-122	107
	1	126.0	-122	107	-122	107
Span 2	**0**	126.0	-253	253	-253	253
	Support	129.0	0	0	-253	253
	0.1	142.8	0	0	-253	253
	0.2	159.6	0	0	-253	253
	0.3	176.4	0	0	-253	253
	0.4	193.2	0	0	-253	253
	0.5	210.0	0	0	-253	253
	0.6	226.8	0	0	-253	253
	0.7	243.6	0	0	-253	253
	0.8	260.4	0	0	-253	253
	0.9	277.2	0	0	-253	253
	Support	291.0	0	0	-253	253
	1	294.0	-253	253	-253	253
Span 3	**0**	294.0	-115	132	-115	132
	Support	297.0	0	0	-115	132
	0.1	305.8	0	0	-115	132
	0.2	317.6	0	0	-115	132
	0.3	329.4	0	0	-115	132
	0.4	341.2	0	0	-115	132
	0.5	353.0	0	0	-115	132
	0.6	364.8	0	0	-115	132
	0.7	376.6	0	0	-115	132
	0.8	388.4	0	0	-115	132
	0.9	400.2	0	0	-115	132
	Support	410.5	0	0	-115	132
	1	412.0	-115	132	-115	132



wFRAME Positive Convention



Standard Positive Convention

Table 1.6 Shear Demand Envelope

Shear Demand (k) Envelope

	Location		No. of Girders in Effective Section				Case 1	Case 2	Case 1		Case 2		Envelope		
				V_{DL}	V_{ADL}	V_{PS}	V_{EQ}	V_{EQ}	$V_{positive}$	$V_{negative}$	$V_{positive}$	$V_{negative}$	$V_{positive}$	$V_{negative}$	V_{max}
Span 1	**Support**	1.5	4	536	96	63	-122	107	574	510	803	739	803	510	803
	0.1	12.6	5	498	89	78	-122	107	543	465	772	694	772	465	772
	0.2	25.2	5	303	54	76	-122	107	311	235	540	464	540	235	540
	0.3	37.8	5	107	19	76	-122	107	80	4	309	233	309	4	309
	0.4	50.4	5	-89	-16	75	-122	107	-151	-227	78	3	78	-227	227
	0.5	63.0	5	-284	-51	75	-122	107	-382	-457	-153	-228	-153	-457	457
	0.6	75.6	5	-480	-86	75	-122	107	-613	-688	-384	-459	-384	-688	688
	0.7	88.2	5	-675	-121	75	-122	107	-843	-918	-614	-689	-614	-918	918
	0.8	100.8	5	-871	-156	75	-122	107	-1075	-1149	-846	-920	-846	-1149	1149
	0.9	113.4	5	-1070	-191	30	-122	107	-1354	-1383	-1125	-1154	-1125	-1383	1383
	Support	123.0	4	-986	-174	107	-122	107	-1175	-1282	-946	-1053	-946	-1282	1282
Span 2	**Support**	129.0	4	1029	182	-35	-253	253	958	923	1464	1429	1464	923	1464
	0.1	142.8	5	1056	189	-22	-253	253	992	971	1498	1477	1498	971	1498
	0.2	159.6	5	795	142	2	-253	253	686	684	1192	1190	1192	684	1192
	0.3	176.4	5	534	96	2	-253	253	378	377	884	883	884	377	884
	0.4	193.2	5	273	49	2	-253	253	71	69	577	575	577	69	577
	0.5	210.0	5	13	2	2	-253	253	-237	-238	269	268	269	-238	269
	0.6	226.8	5	-248	-45	1	-253	253	-544	-546	-38	-40	-38	-546	546
	0.7	243.6	5	-509	-91	1	-253	253	-852	-853	-346	-347	-346	-853	853
	0.8	260.4	5	-770	-138	1	-253	253	-1160	-1161	-654	-655	-654	-1161	1161
	0.9	277.2	5	-1031	-185	-28	-253	253	-1469	-1496	-963	-990	-963	-1496	1496
	Support	291.0	4	-1009	-178	30	-253	253	-1411	-1440	-905	-934	-905	-1440	1440
Span 3	**Support**	297.0	4	937	165	-94	-115	132	987	893	1234	1140	1234	893	1234
	0.1	305.8	5	1021	182	-69	-115	132	1088	1020	1335	1267	1335	1020	1335
	0.2	317.6	5	834	149	-48	-115	132	868	820	1115	1067	1115	820	1115
	0.3	329.4	5	651	117	-48	-115	132	652	604	899	851	899	604	899
	0.4	341.2	5	468	84	-48	-115	132	436	388	683	635	683	388	683
	0.5	353.0	5	284	51	-49	-115	132	220	172	467	419	467	172	467
	0.6	364.8	5	101	18	-48	-115	132	4	-44	252	203	252	-44	252
	0.7	376.6	5	-82	-15	-48	-115	132	-212	-259	36	-12	36	-259	259
	0.8	388.4	5	-265	-47	-48	-115	132	-428	-476	-181	-229	-181	-476	476
	0.9	400.2	5	-448	-80	-48	-115	132	-644	-692	-397	-445	-397	-692	692
	Support	410.5	4	-486	-87	-54	-115	132	-689	-743	-442	-496	-442	-743	743

$V_{positive} = V_{EQ, max} + V_{DL} + V_{ADL} + V_{PS}$*
$V_{negative} = V_{EQ, max} + V_{DL} + V_{ADL} + V_{PS}$**
V_{max} = Greater of Absolute($V_{positive}$) or Absolute($V_{negative}$)

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

Table 1.7 Section Flexural Capacity Calculation Data

P_{jack} = 9689.9 k

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

*** Area of mild steel based on minimum seismic requirement only (Remaining limit state requirements need to be satisfied; $A_{st,top}$ = 56.6 in^2 at right face of Bent 2)**

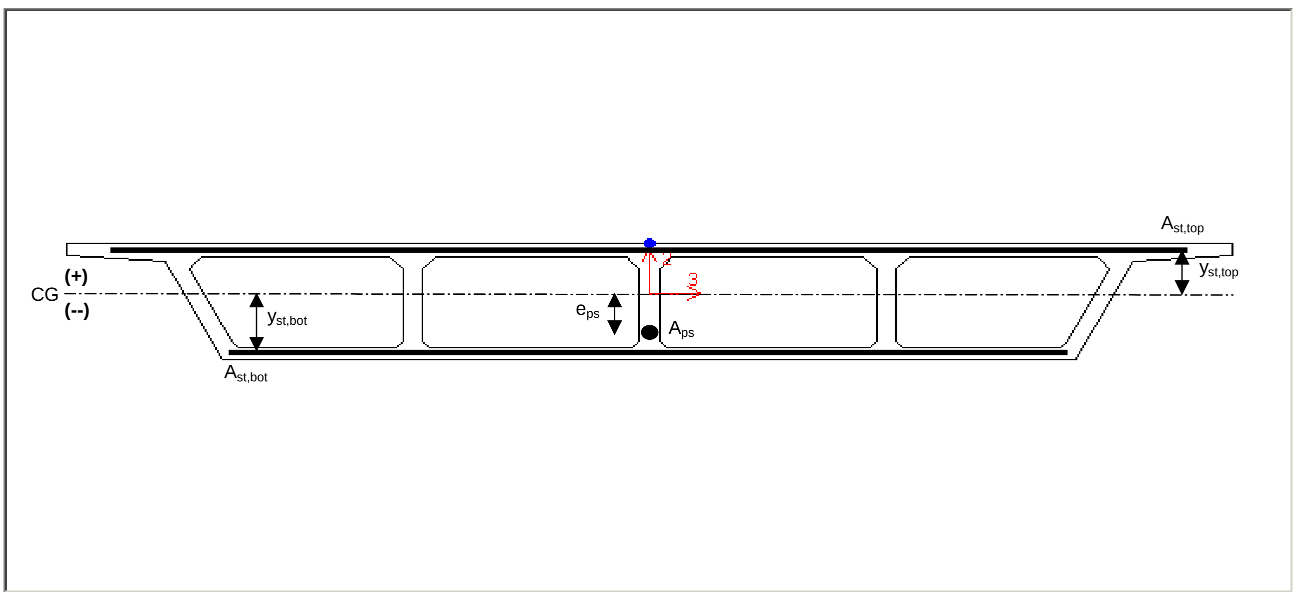


Table 1.8 Moment Demand Vs. Capacity

Moment Capacity Check

	Location		Moment Demand		Moment Capacity		D/C Ratio	
			$M_{positive}$	$M_{negative}$	$M_{positive}$	$M_{negative}$	$M_{positive}$	$M_{negative}$
Span 1	**Support**	1.5	1264	404	34596	-38743	0.04	0.00
	0.1	12.6	11192	6854	55180	-32488	0.20	0.00
	0.2	25.2	19299	11274	66080	-21498	0.29	0.00
	0.3	37.8	24513	12788	72579	-15148	0.34	0.00
	0.4	50.4	26856	11393	74776	-13090	0.36	0.00
	0.5	63.0	26180	7092	86980	-35177	0.30	0.00
	0.6	75.6	22761	-116	81403	-40842	0.28	0.00
	0.7	88.2	16305	-10232	72097	-50656	0.23	0.20
	0.8	100.8	7671	-23254	57859	-64846	0.13	0.36
	0.9	113.4	-4802	-39190	37336	-85324	0.00	0.46
	Support	123.0	-10662	-45681	25447	-83335	0.00	0.55
Span 2	**Support**	129.0	-3819	-52023	25308	-83481	0.00	0.62
	0.1	142.8	6519	-37433	40465	-82185	0.16	0.46
	0.2	159.6	20083	-14845	63415	-59711	0.32	0.25
	0.3	176.4	28302	2575	77800	-44699	0.36	0.00
	0.4	193.2	32005	14828	71765	-15896	0.45	0.00
	0.5	210.0	30324	21915	74777	-13077	0.41	0.00
	0.6	226.8	31910	15706	71764	-15897	0.44	0.00
	0.7	243.6	28498	3949	77795	-44689	0.37	0.00
	0.8	260.4	20542	-12971	63278	-59694	0.32	0.22
	0.9	277.2	7262	-35055	40444	-82178	0.18	0.43
	Support	291.0	-2996	-49703	25298	-83478	0.00	0.60
Span 3	**Support**	297.0	-9500	-43839	25495	-83265	0.00	0.53
	0.1	305.8	-5043	-38502	37078	-85517	0.00	0.45
	0.2	317.6	6081	-24055	57837	-64774	0.11	0.37
	0.3	329.4	13754	-12171	72076	-50587	0.19	0.24
	0.4	341.2	19490	-2835	81811	-40796	0.24	0.07
	0.5	353.0	22583	3952	87548	-35162	0.26	0.00
	0.6	364.8	23244	8190	74766	-13061	0.31	0.00
	0.7	376.6	21277	9880	72559	-15111	0.29	0.00
	0.8	388.4	16784	9021	66061	-21439	0.25	0.00
	0.9	400.2	9753	5614	55171	-32413	0.18	0.00
	Support	410.5	1198	405	34704	-38615	0.03	0.00

Table 1.9 Shear Demand Vs. Capacity

Shear Capacity Check

	Location		Shear Demand	Shear Capacity = Governing Shear Demand*	D/C Ratio
			V_{max}	$\phi V_n = V_{u,\ limit\ state}$	D/C
Span 1	**Support**	1.5	803	2851	0.28
	0.1	12.6	772	2317	0.33
	0.2	25.2	540	1687	0.32
	0.3	37.8	309	1101	0.28
	0.4	50.4	227	681	0.33
	0.5	63.0	457	1207	0.38
	0.6	75.6	688	1782	0.39
	0.7	88.2	918	2341	0.39
	0.8	100.8	1149	2901	0.40
	0.9	113.4	1383	3596	0.38
	Support	123.0	1282	3966	0.32
Span 2	**Support**	129.0	1464	4378	0.33
	0.1	142.8	1498	3759	0.40
	0.2	159.6	1192	2961	0.40
	0.3	176.4	884	2160	0.41
	0.4	193.2	577	1399	0.41
	0.5	210.0	269	686	0.39
	0.6	226.8	546	1375	0.40
	0.7	243.6	853	2139	0.40
	0.8	260.4	1161	2942	0.39
	0.9	277.2	1496	3792	0.39
	Support	291.0	1440	4367	0.33
Span 3	**Support**	297.0	1234	3760	0.33
	0.1	305.8	1335	3388	0.39
	0.2	317.6	1115	2817	0.40
	0.3	329.4	899	2312	0.39
	0.4	341.2	683	1774	0.39
	0.5	353.0	467	1238	0.38
	0.6	364.8	252	738	0.34
	0.7	376.6	259	1000	0.26
	0.8	388.4	476	1548	0.31
	0.9	400.2	692	2138	0.32
	Support	410.5	743	2653	0.28

***Shear demand base on governing limit state requirement as determined by CTBridge**

II.E. Final Displacement Demand Assessment

The LRFD Section 4.3 specifies that the total displacement demand be determined based on the combined responses to motions applied in two orthogonal directions. This is to account for the directional uncertainty of the earthquake. Given the skewed bents in this prototype bridge, strictly following these specifications would lead to a more complex analysis requiring the use of a three-dimensional pushover analysis to determine the displacement capacities. For regular bridges, such a sophisticated analysis exceeds Caltrans typical design practice.

As discussed previously, Caltrans SDC Section 2.1.2, which provides an alternative to the approach above, was followed to determine demands and capacities along the principle axes of individual components. Based on the pushover analysis performed for both the transverse and longitudinal directions, the columns adequately resist the displacement demands in the two principle directions and the bent and superstructure have sufficient capacity to handle the overstrength demands imparted by column plastic hinging.

II.F. Joint Shear Design

In a ductility based design approach for concrete structures, connections are key elements that must have adequate strength to maintain structure 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.

According to the LRFD Section 8.13.1, the 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_{po}^{col}. Additionally, the effects of overstrength shear V_{po}^{col} will be considered.

According to the LRFD Section 8.13.3, the following types of superstructure-to-substructure joints are considered T joints for the purpose of joint stress:

- Integral interior joints of a multi-column bents in the transverse direction
- All column/superstructure joints in the longitudinal direction.
- Exterior column joints for box girder superstructure if the cap beam extends beyond the joint far enough to develop the longitudinal cap reinforcement. All other exterior joints are considered knee joints in the transverse direction.

Other typical types of superstructure-to-substructure connections are knee and outrigger joints depending upon if the cap terminates within the box girder or it extends beyond it.

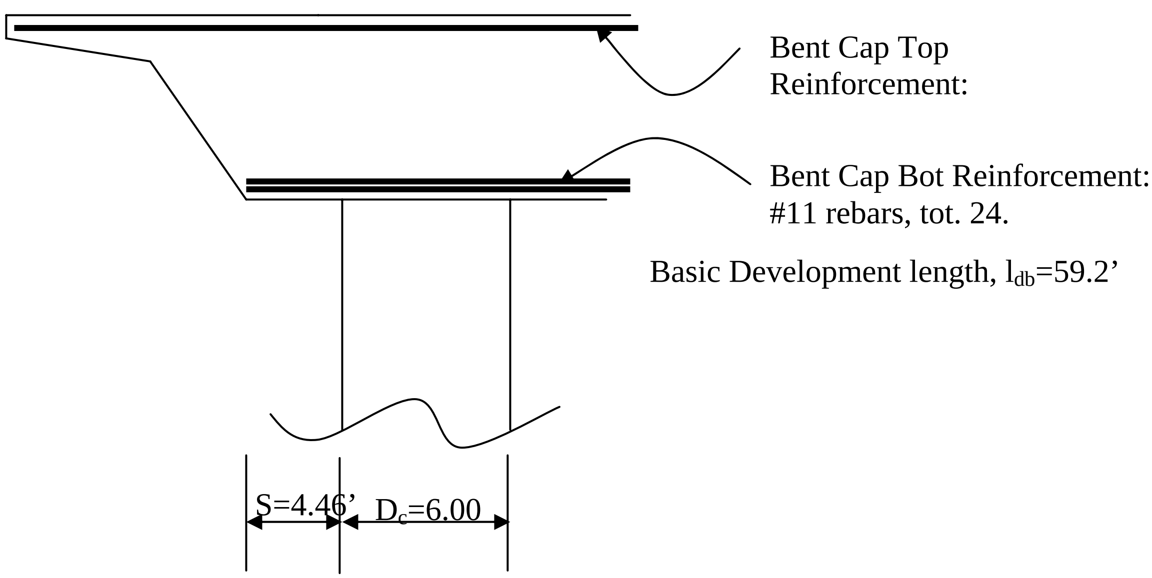


Figure 13 Bent Cap-to-Column Joint

Since the cap beam short stub length, S, is less than the development length of the main cap reinforcement, l_{db}, the column to cap joint cannot be characterized as a T-joint for transverse bending. Instead, it will be analyzed as knee-joint.

II.F.i. Transverse Direction (Knee Joint)

Knee joints require special analysis and detailing not currently provided in the LRFD. Therefore, the following procedure and guidelines used herein are based on SDC recommendations for T-joints along with additional recommendations from the SDC Joint Shear Work Team and the paper entitled, "Knee-Joint Shear Design Guidelines – DRAFT". A knee joint is defined as any exterior column joint where the cap beam short stub length, *S*, is less than the diameter of the column, D_c, or less than the development length of the main bent cap reinforcement, l_d.

$$S < D_c \qquad \text{or} \qquad S < l_d$$

In general, there are two cases that need to considered:

Case1: $S < \frac{D_c}{2}$

Case2: $\frac{D_c}{2} < S < D_c \ or \ \ l_d$

In our case $S = 4.42' > \frac{6.00'}{2}$, therefore it is classified as Case 2 knee joint.

Knee joints can fail in both opening and closing modes. Therefore, both loading conditions will be evaluated.

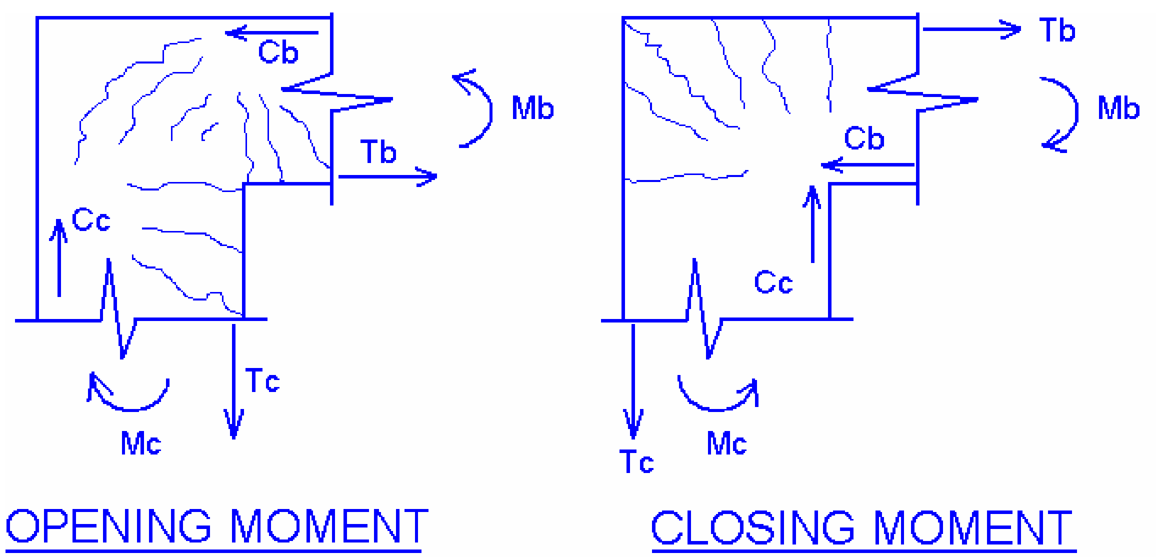


Figure 14 Knee Joint Failure Modes

In the opening moment, a series of arch-shaped cracks tends to form between the compression zones at the outside of the column and top of the beam. The intersection of the arch strut and the flexural compression zones at the top of the beam create outward-acting resultant forces. If the beam bottom reinforcement is anchored only by straight bar extension, there is nothing to resist the horizontal resultant tensile force. It will cause vertical splitting, reducing competence of the anchorage of the outer column rebars and beam top rebars.

Under the closing moment, a fan –shaped pattern of cracks develops, radiating from the outer surfaces of beam and column toward the inside. If there is no vertical reinforcement, clamping the beam top reinforcement into the joint, the entire beam tension, T_b, is transferred to the back of the joint as there is no mechanism to resist the moment at the base of the wedge shaped concrete elements caused by tension transfer to the concrete by bond.

Let us consider Bent 2 Knee Joint *Closing* Mode Failure.

Given:

Concrete compressive strength, $f_{ce}' = 5,200\ psi$

Superstructure depth, $D_s = 6.75\ ft$

Column diameter, $D_c = 6\ ft$

Column reinforcement:

- Main reinforcement: #14 bars, total 26 giving $A_{st} = 58.50\ in^2$.
- Transverse reinforcement: #8 hoops at 5” c/c.

Concrete cover= $2\ in$

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

Bent cap width, $B_{cap} = 96\ in$

Column plastic moment, $M_p = 14,906\ ft-kip$ *

Column axial force (including the effect of overturning), $P_c = 2,470\ k$ *

Cap Beam main reinforcement

- Top:#11 bars, total 22.
- Bottom: #11 bars, total 24.

*These values are obtained from the *xSECTION* and *wFRAME* pushover analysis of Bent 2 and are listed on page ?.

Calculate principal stresses, p_t *and* p_c

- Vertical shear stress, $v_{jv} = \dfrac{T_c}{A_{jv}}$

$A_{jv} = l_{ac} \times B_{cap}$ where l_{ac} = Anchorage of column rebars into the bent cap.
B_{cap} = Bent Cap Width.

The tensile stress resultant in the column, $T_{c,}$ corresponding to the column overstrength moment, M_0 is obtained to be *1.2 x 2872 kips = 3,446 kips* (CT 3434 k) using *xSECTION* results. See Appendix W.

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

$$\therefore v_{jv} = \frac{3{,}446}{6336} = 0.544\ ksi$$

- Nominal vertical stress,

$$f_v = \frac{P_c}{A_{jh}} = \frac{P_c}{\left(D_c + (2 \times \frac{D_s}{2})\right) \times B_{cap}} = \frac{2{,}470}{\left(6.00 + (2 \times \frac{6.75}{2})\right) \times 8.00 \times 144} = 0.168\ ksi$$

- Nominal horizontal stress

$$f_h = \frac{P_b}{B_{cap} \times D_s} = \frac{0}{8.00 \times 6.75} = 0.00\ ksi$$

Since no prestressing is specifically designed to provide horizontal joint compression, it is assumed that $P_b = 0$.

Now the principal stresses are calculated.

$$p_t = \left| \frac{(0.00 + 0.168)}{2} - \sqrt{\left(\frac{0.00 - 0.168}{2}\right)^2 + 0.544^2} \right| = \left| -0.466 \right| = 0.466\ ksi \qquad (CT \quad 0.464)$$

The negative sign indicates that the joint is under nominal principal tensile stresses.

$$p_c = \frac{(0.00+0.168)}{2} + \sqrt{\left(\frac{0.00-0.168}{2}\right)^2 + 0.544^2} = 0.634\ ksi \qquad (CT \quad 0.632)$$

<u>Check the Joint Size Adequacy</u>

According to LRFD Section 8.13.2, all superstructure to column moment resisting joints shall be proportioned so that the principal stresses satisfy the following requirements:

Principal compression, $p_c \le 0.25 \times f_{ce}' \quad (ksi)$ LRFD Eqn. (8.38)

Principal tension, $p_t \le 12 \times \sqrt{f_{ce}'} \quad (psi)$ LRFD Eqn. (8.39)

In our case,

Principal compression, $p_c = 0.634 ksi \le 0.25 \times 5.2 = 1.3\ ksi \qquad (CT \quad 1.0)$ OK
Principal tension, $p_t = 0.466\ ksi < 12 \times \sqrt{5200}/1000 = 0.865\ ksi \qquad (CT \quad 0.760)$ OK

Therefore, the bent cap-to-column joint satisfies the SDC joint proportioning requirements.

<u>Check the Need for Additional Joint Requirement</u>

According to the LRFD Section 8.13.4.2, if the principal tensile stress, $p_t \le 3.5 \times \sqrt{f_{ce}'}\ (psi)$, no additional joint reinforcement is required. If no additional joint reinforcement is needed, then the volumetric ratio of transverse column reinforcement ρ_s continued into the cap shall not be less that

$$\rho_{s,\min} = \frac{3.5 \times \sqrt{f_{ce}'}}{f_{yh}} \quad (psi)$$ LRFD Eqn. (8.47)

Since in our case $p_t = 0.466\ ksi > 3.5 \times \sqrt{5200}/1000 = 0.252\ ksi$ (CT 0.221), additional joint reinforcement will be necessary.

Similar calculations can be performed for Bent 3.

Let us now evaluate the same Bent 2 Knee Joint for the *Opening* Mode Failure.

Given:

From the *wFRAME* push-over analysis results, Column plastic moment, $M_p = 12,502\ ft-kip$ *

Column axial force (including the effect of overturning), $P_c = 911\, kip$ *

Cap Beam main reinforcement

- Top:#11 bars, total 22.
- Bottom: #11 bars, total 24.

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

- Vertical shear stress, $v_{jv} = \dfrac{T_c}{A_{jv}}$

The tensile stress resultant in the column, $T_{c,}$ corresponding to the column overstrength moment, M_0 is obtained to be *1.2 x 3,076 kips = 3,691 kips* (CT 3778 k) using *xSECTION* results.

$$A_{jv} = 66 \times 96 = 6336\, in^2$$

$$\therefore v_{jv} = \frac{3,691}{6,336} = 0.583\, ksi$$

- Nominal vertical stress,

$$f_v = \frac{P_c}{A_{jh}} = \frac{P_c}{\left(D_c + (2 \times \frac{D_s}{2})\right) \times B_{cap}} = \frac{911}{\left(6.00 + (2 \times \frac{6.75}{2})\right) \times 8.00 \times 144} = 0.062\, ksi$$

- Nominal horizontal stress

$$f_h = \frac{P_b}{B_{cap} \times D_s} = \frac{0}{8.00 \times 6.75} = 0.00$$

Since no prestressing is specifically designed to provide horizontal joint compression, we can assume that $P_b = 0$.

Now the principal stresses are calculated substituting these data.

$$p_t = \left| \frac{(0.00 + 0.062)}{2} - \sqrt{\left(\frac{0.00 - 0.062}{2}\right)^2 + 0.583^2} \right| = |-0.553| = 0.553\, ksi \qquad (CT \quad 0.566)$$

The negative sign indicates that the joint is under nominal principal tensile stresses.

$$p_c = \frac{(0.00+0.062)}{2} + \sqrt{\left(\frac{0.00-0.062}{2}\right)^2 + 0.583^2} = 0.614\, ksi \qquad (CT \quad 0.628)$$

Check the Joint Size Adequacy

Principal compression, $p_c = 0.614\, ksi \le 0.25 \times 5.2 = 1.3\, ksi$ OK

Principal tension, $p_t = 0.553\, ksi < 12 \times \sqrt{5200}/1000 = 0.865\, ksi$ OK

Therefore, the bent cap-to-column joint satisfies the SDC joint proportioning requirements.

Check the Need for Additional Joint Reinforcement

According to the LRFD Section 8.13.4.2, if the principal tensile stress, $p_t \le 3.5 \times \sqrt{f_{ce}'}\ (psi)$, no additional joint reinforcement is required. If no additional joint reinforcement is needed, then the volumetric ratio of transverse column reinforcement ρ_s continued into the cap shall not be less that

$$\rho_{s,\min} = \frac{3.5 \times \sqrt{f_{ce}'}}{f_{yh}} \quad (psi) \qquad \text{LRFD Eqn. (8.47)}$$

Since in our case $p_t = 0.553 ksi > 3.5 \times \sqrt{5200}/1000 = 0.252\, ksi$, additional joint reinforcement will be necessary.

Therefore, based upon joint stress condition evaluation for both closing and opening modes of failure, the joint needs additional joint reinforcement. Now refer to Figure 15.

Joint Shear Requirement

a.0) Continuous U-Bars (Refer to Figures 16 and 20)

The top and bottom main bent cap reinforcement shall be in the form of continuous U-bars. The minimum area of this type of reinforcement shall be 33% of the area of the main column reinforcement anchored into the bent cap. The splices in U-bars shall not be allowed within a distance l_d beyond the interior face of the column.

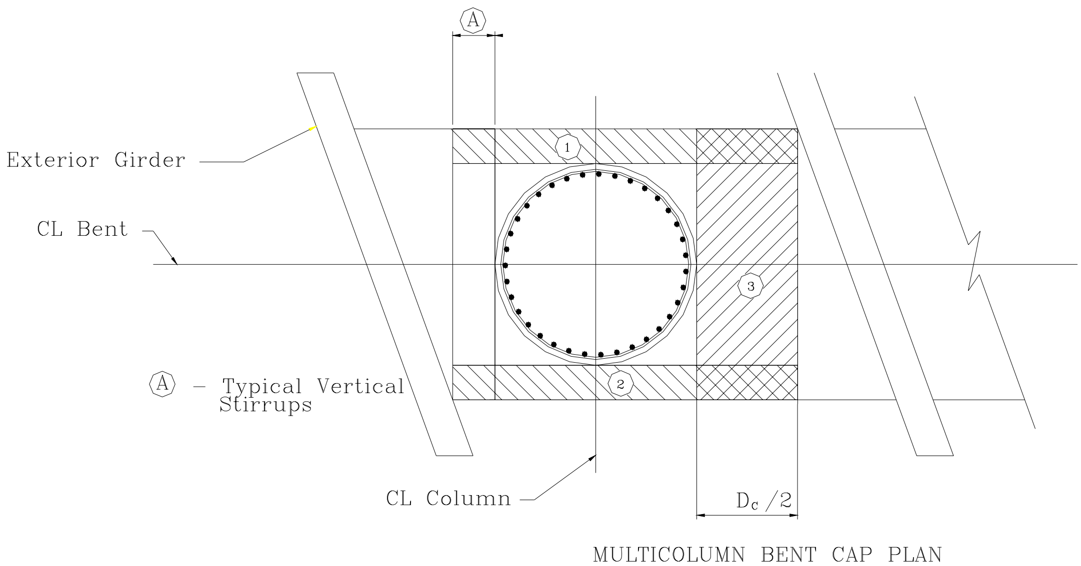


Figure 15 Regions of Additional Joint Shear Reinforcement

$$A_{s\ required}^{U-Bar} = 0.33 \times A_{st} = 0.33 \times 58.5 = 19.3\ in^2$$

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

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

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

$$A_{s\ provided}^{U-Bar} = 12 \times 1.56 = 18.72\ in^2 \approx 19.3\ in^2 \qquad \text{OK}$$

See Figure 16 for rebar layout.

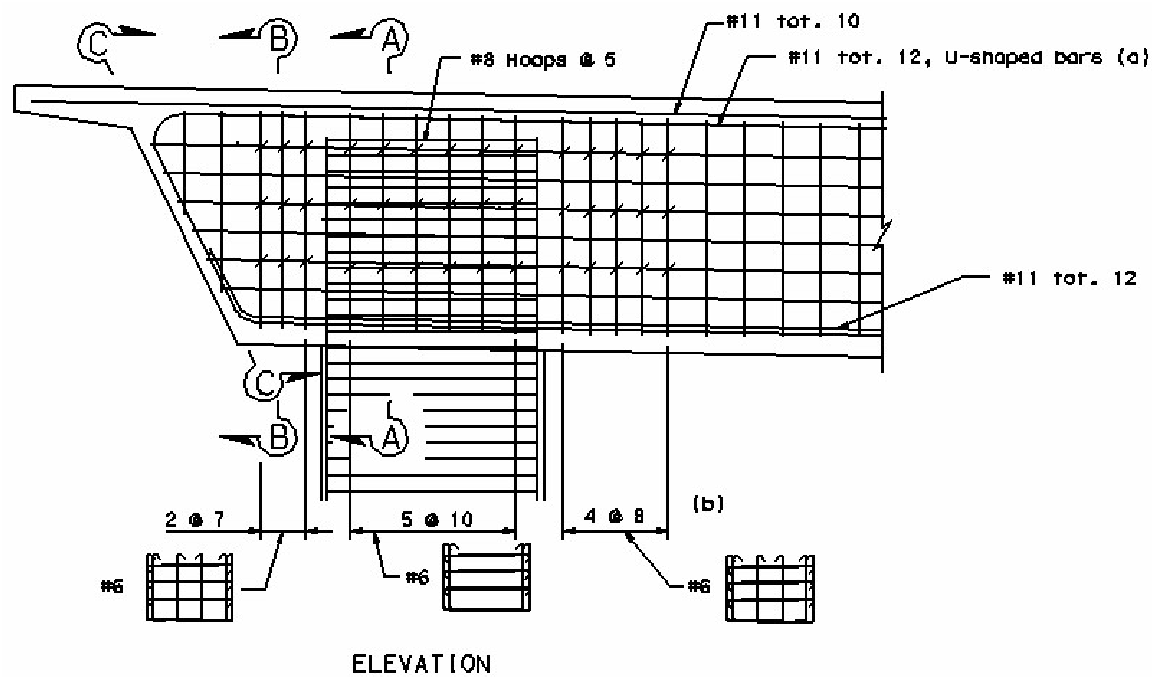


Figure 16 Location of Joint Shear Reinforcement (Elevation View)

a.1) Vertical Stirrups in Joint Region

Vertical stirrups or ties shall be placed transversally within region 3 as shown in Figure 15.

$$A_s^{jv} = 0.2 \times A_{st}$$ LRFD Eqn. (8.48)

where A_{st} = Total area of column main reinforcement anchored in the joint.

In our case, the whole column main reinforcement i.e. #14, total 26, is anchored into the joint.

$$A_{s\ required}^{jv} = 0.20 \times 58.5 = 11.7 in^2$$

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

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

These vertical stirrups and ties are placed transversely within a distance $\frac{D_c}{2}$ extending from the face of the column. The maximum stirrup spacing was previously calculated for the bent cap shear capacity and was determined to be 24.0".

As shown in Figures 16 and 17, place 5 sets at 8 in c/c in region 3. These vertical stirrups are shown in Figure 16 and also as dots in Figure 17.

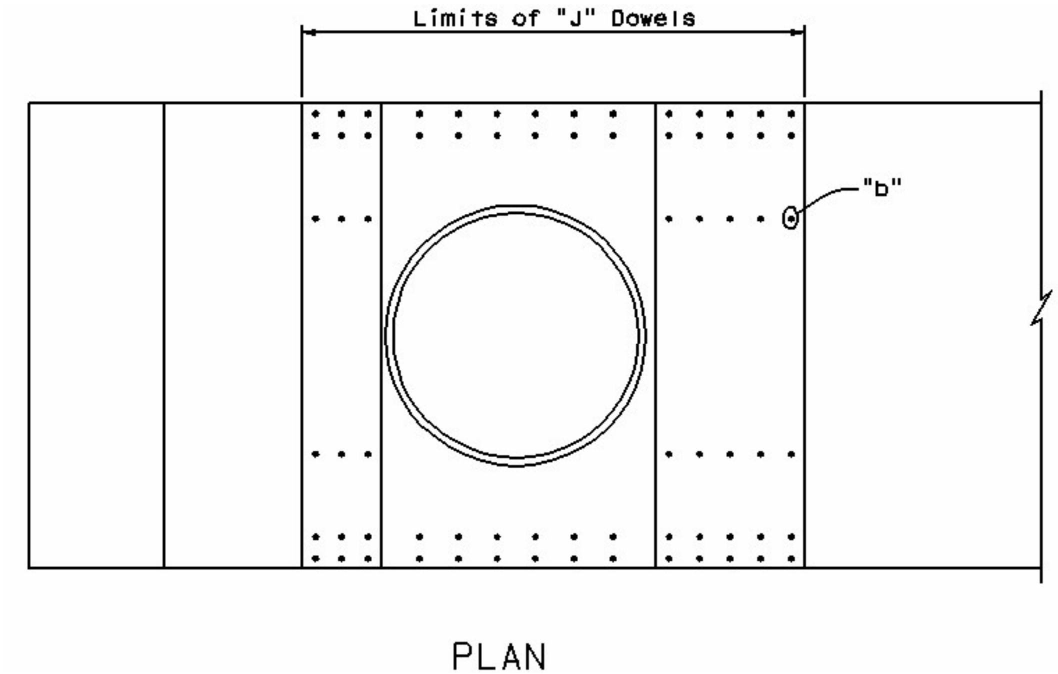


Figure 17 Location of Vertical Stirrups (Elevation View*)*

Note the required stirrup spacing determined from the overstrength shear demands on the bent cap is 5 in. and governs for this location. However, to illustrate the design procedure for joint shear, the remainder of the joint shear design will be carried out using the 8 in spacing as shown in Figure 16.

b) Horizontal Stirrups in Joint Region

Horizontal stirrups or ties shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers vertically at not more than 18 in.

$$A_{s\ required}^{jh} = 0.1 \times A_{st}$$ LRFD Eqn. (8.49)

$$A_{s\ required}^{jh} = 0.1 \times 58.5 = 5.85 in^2$$

As shown in Figure 18, provide 3 legged #6 total 14 sets so that

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

This horizontal reinforcement shall be placed within a distance D_c extending from either side of the column centerline as shown in Figure 16. These stirrups are in Figure 18.

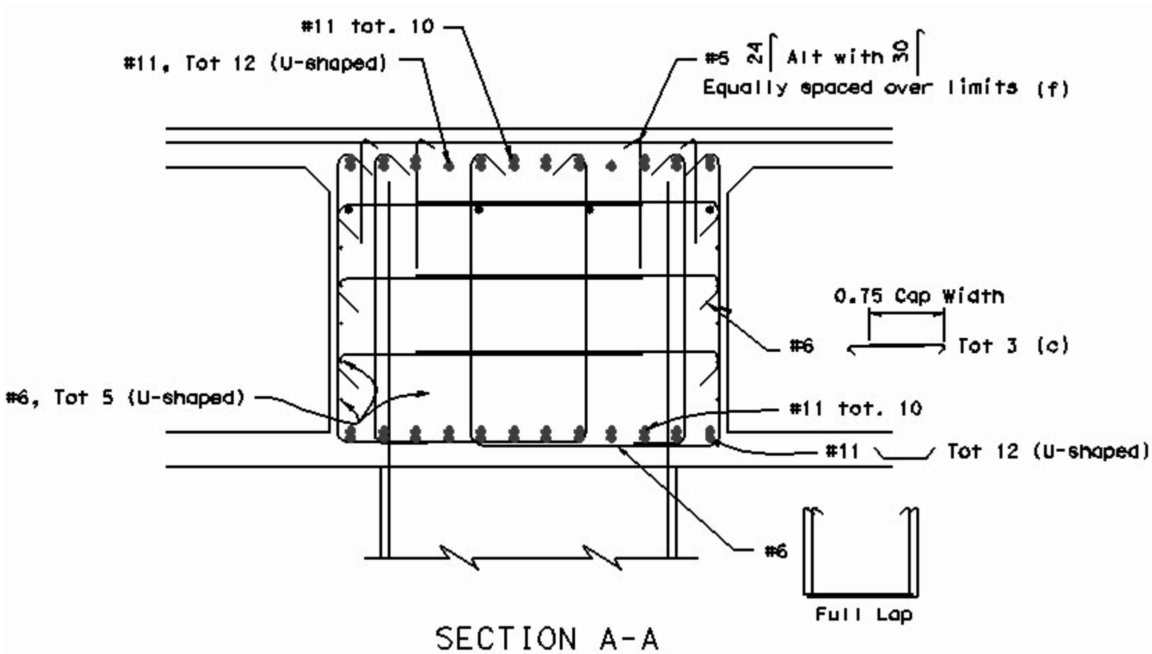


Figure 18 Joint Reinforcement Within the Column Region

c) Horizontal Side Reinforcement

According to the LRFD Section 8.13.4.3(C), the total longitudinal side face reinforcement in the bent cap shall be equal to the greater of the area specified by

$$A_s^{sf} \geq \begin{cases} 0.1 \times A_{cap}^{top} \\ \quad or \\ 0.1 \times A_{cap}^{bot} \end{cases}$$ LRFD Eqn. (8.50)

where $A_{cap} =$ Area of bent cap top or bottom flexural steel.

This side reinforcement shall be continuous around the joint end and placed near the side faces of the bent cap with a maximum spacing of 12 in. As shown in Figures 18 and 19, such horizontal reinforcement shall be in the form of continuous over the end face of the knee-joint. Splices in these continuous bars shall be located at least distance l_d beyond the interior face of the column.

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

$A_{cap}^{bot} = 37.44\ in^2$

$$A_s^{sf} \geq \begin{cases} 0.1 \times 34.32 = 3.43\ in^2 \\ or \\ 0.1 \times 37.44 = 3.74\ in^2 \end{cases}$$

As shown in Figures 18 and 19, provide #6, 5 continuous per side giving

$A_{s\ provided}^{sf} = (10\,bars) \times 0.44 = 4.4\ in^2 \quad > \quad 3.74\ in^2$.

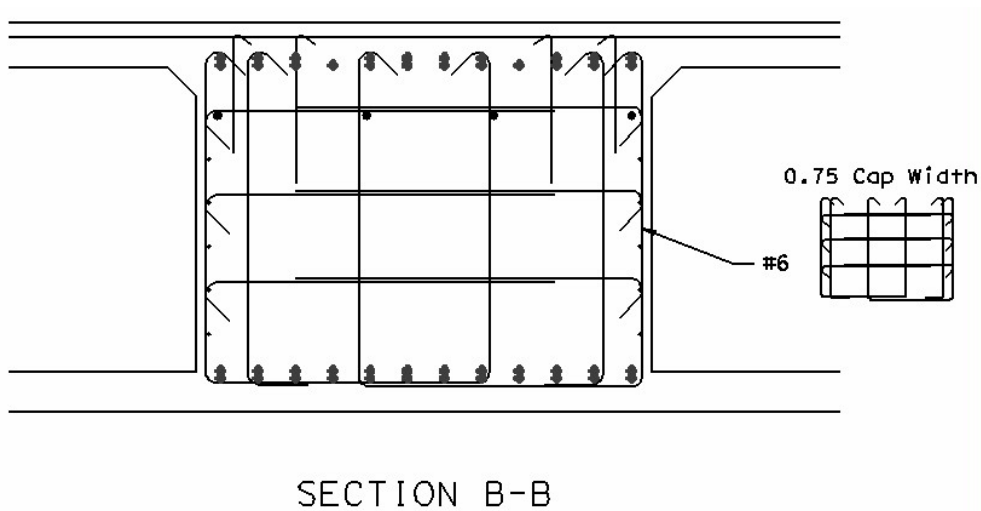


Figure 19 Joint Reinforcement Outside the Column Region

d) J-Dowels

According to the LRFD Section 8.13.4.3(D), for bents skewed greater than 20^0, J-dowels hooked around the longitudinal top deck steel extending alternatively 24 in and 30 in into the bent cap are required. This helps to prevent any potential delamination of concrete around deck top reinforcement. Although strictly following LRFD guidelines, there is no need for J-Dowels for this bridge, we will provide it anyway.

$A_s^{j-bar} = 0.08 \times A_{st} = 0.08 \times 58.5 = 4.68\ in^2$ LRFD Eqn. (8.51)

Use 16, #5 J-Dowels.

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

The J-Dowels will be uniformly 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. These dowels are shown in Figures 18 and 19.

e) Transverse Reinforcement

According to the recommendations made by the Work Team on Joint Shear, the transverse reinforcement in the joint region shall consist of hoops with a minimum reinforcement ratio specified as

$$\rho_s = 0.4 \times \frac{A_{st}}{l_{ac}^{\ 2}} \qquad \text{LRFD Eqn. (8.52)}$$

A_{st} = Area of longitudinal column reinforcement
$l_{ac=}$ = Anchorage length.

$$\rho_{s\,required} = 0.4 \times \frac{58.5}{66^2} = 0.0054$$

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

$$\rho_{s,\quad provided} = \frac{4 \times A_b}{D' \times s} = \frac{4 \times 0.79}{\left(72 - 2 \times 2 - 2 \times \frac{1.13}{2}\right) \times (5)} = 0.0094 \ > \ 0.0054$$

e) Anchorage for Main Column Reinforcement

According to the LRFD Section 8.13.4.3(F), the main column reinforcement shall extend into the cap as deeply as possible in order to fully develop the compression strut mechanism in the joint.

If the joint shear reinforcement prescribed in LRFD Section 8.13.4.2, and the minimum bar spacing requirements in AASHTO are met, then the anchorage for longitudinal column bars developed into the cap beam for seismic loads shall not be less than the length specified in LRFD Section 8.8.4:

$$l_{ac,\,required} = 24 d_{bl}$$

$$l_{ac,\,required} = 24 \times 1.69 = 40.6\ in$$

$$l_{ac,\,provided} = 66\ in > 40.6\ in \qquad \text{OK}$$

It is important to note that as per the LRFD requirements, the minimum anchorage length specified above cannot be reduced by adding hooks or mechanical anchorage devices. The reinforcement development requirements in LRFD Section 5.11 must also be satisfied for all cases other than seismic.

II.F.ii. Longitudinal (T-joint)

As determined earlier based upon LRFD guidelines, the connection between the column and the bent cap is analyzed as a T-joint for longitudinal bending.

For longitudinal bending, the overturning effects on the column axial force are insignificant, and hence the column plastic moments due to dead load can be used. Let us calculate joint stresses for the column that will provide higher value of principal tensile stress, generally more critical than principal compressive stress.

Column plastic moment, $M_p = 13,808\ ft-kip$ *

Column axial force (neglecting the effect of overturning), $P_c = 1,694\ kip$ *

Cap Beam main reinforcement

- Top Reinforcement: #11, total 22 bars
- Bottom Reinforcement #11, total 24 bars.

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

- Vertical shear stress, $v_{jv} = \dfrac{T_c}{A_{jv}}$

The tensile stress resultant in the column, $T_{c,}$ corresponding to the column overstrength moment, M_0 is obtained to be *1.2 x 3,040 kips = 3,648 kips* using *xSECTION* results.

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

$$\therefore v_{jv} = \frac{3,648}{6,336} = 0.576\ ksi$$

- Nominal vertical stress,

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

  - Nominal horizontal stress

Since no prestressing is specifically designed to provide horizontal joint compression, it is assumed that that $P_b = 0$.

$$f_h = \frac{P_b}{B_{cap} \times D_s} = \frac{0}{8.00 \times 6.75} = 0.00$$

Now the principal stresses are calculated substituting these data.

$$p_t = \left| \frac{(0.00 + 0.115)}{2} - \sqrt{\left(\frac{0.00 - 0.115}{2}\right)^2 + 0.576^2} \right| = |-0.521| = 0.521\, ksi \qquad (CT \quad 0.503)$$

The negative sign indicates that the joint is under nominal principal tensile stresses.

$$p_c = \frac{(0.00 + 0.115)}{2} + \sqrt{\left(\frac{0.00 - 0.115}{2}\right)^2 + 0.576^2} = 0.636\, ksi \qquad (CT \quad 0.676)$$

<u>Check the Joint Size Adequacy</u>

Principal compression, $p_c = 0.636\, ksi \le 0.25 \times 5.2 = 1.3\, ksi$ OK

Principal tension, $p_t = 0.521\, ksi < 12 \times \sqrt{5200} / 1000 = 0.865\, ksi$ OK

Therefore, the bent cap-to-column joint satisfies the LRFD joint proportioning requirements.

<u>Check the Need for Additional Joint Requirement</u>

According to the LRFD Section 8.13.4.2, if the principal tensile stress, $p_t \le 3.5 \times \sqrt{f_{ce}'}\ (psi)$, no additional joint reinforcement is required. If no additional joint reinforcement is needed, then the volumetric ratio of transverse column reinforcement ρ_s continued into the cap shall not be less that

$$\rho_{s,\min} = \frac{3.5 \times \sqrt{f_{ce}'}}{f_{yh}} \quad (psi) \qquad \text{LRFD Eqn. (8.47)}$$

Since in our case $p_t = 0.521\, ksi > 3.5 \times \sqrt{5200} / 1000 = 0.252\, ksi$, additional joint reinforcement will be necessary.

The horizontal stirrups, cap beam u-bar requirements, continuous cap side face reinforcement, j-dowels, and column reinforcement anchorage provided for the transverse bending will also satisfy the joint shear requirements for the longitudinal bending. The only additional joint reinforcement requirement that needs to be satisfied for the longitudinal bending is to provide vertical stirrups in Regions 1 and 2 of Figure 15.

a) Vertical Stirrups in Joint Region – Regions 1 and 2 of Figure 15

$A_s^{jv} = 0.2 \times A_{st}$ LRFD Eqn. (8.48)

where A_{st} = Total area of column main reinforcement anchored in the joint.

In our case, all the column main reinforcement i.e. #14, total 26 bars are anchored into the bent cap.

$\therefore A_s^{jv}{}_{provided} = 0.2 \times 58.5 = 11.7\ in^2$

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

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

As shown in Figures 16 and 17, these vertical stirrups and ties are placed transversely within a distance D_c extending from either side of the column centerline. The maximum stirrup spacing was previously calculated for the bent cap shear capacity and was determined to be 24.0”.

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 towards two legs of the vertical stirrups required for the longitudinal bending.

<u>Transverse Reinforcement</u>

According to the LRFD, the transverse reinforcement in a T-joint joint region shall consist of hoops with a minimum reinforcement ratio specified as:

$$\rho_{s\,required} = 0.4 \times \frac{A_{st}}{l_{ac}^2} \quad (in)$$ LRFD Eqn. (8.52)

Also, all vertical bars shall be extended to within 12 in from the deck top, so:

$$\rho_s = 0.4 \times \frac{58.5}{66^2} = 0.0054$$

As calculated for transverse bending:

$\rho_{s,\ provided} = 0.0094 > 0.0054$ OK

II.G. Torsional Capacity Check

The torsional effects in the bent cap beam under the longitudinal bending are well resisted by this integral bent cap that is clamped by the box girder superstructure on each side. If the superstructure remains elastic under the longitudinal bending (It will be assured that such is the case by making sure that the superstructure satisfies MTD 20-6 requirements – to be done in a subsequent section), it is difficult to expect torsional distress of cap beams, as the torsional rotations of the bent cap would require significant distortions and warping of the superstructure. Such rotations will be resisted by in-plane membrane forces in the deck and soffit slab. Additionally, there is no history of any damage to bent caps from previous earthquakes for integral bent caps. For these reasons, the torsional capacity of the cap beam is assumed to be adequate and not checked.

II.H. Abutment Seat Width Design

The bridge is supported on a seat type abutment. It is Caltrans design philosophy to provide adequate seat width so that the superstructure does not fall off during the anticipated seismic shaking. As per LRFD Section 4.12.2, sufficient seat width shall be available to accommodate the anticipated thermal movement, prestress shortening, creep, shrinkage, and the relative longitudinal earthquake displacement.

$$\left[N = (4 + \Delta_{ot} + 1.65\Delta_{eq}) \left(\frac{1 + S_k^2}{4000} \right) \right] \geq 12 \quad (in)$$ LRFD Eqn. (4.16)

Where

N = seat width normal to the face of an abutment, a pier or a hinge seat

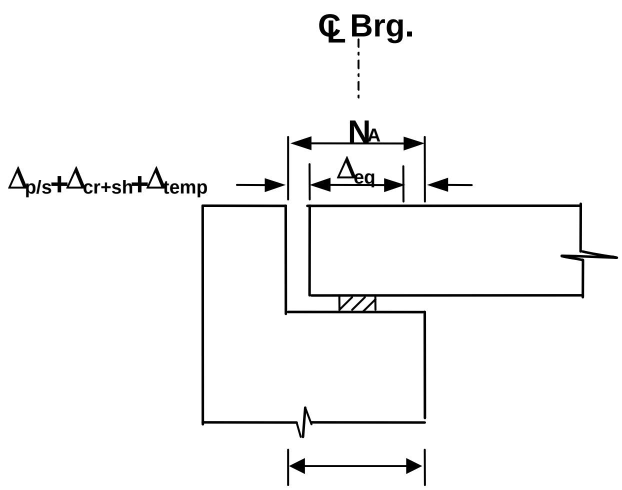
Δ_{ot} = movement attributed to prestress shortening, creep, shrinkage, and thermal expansion or contraction to be considered no less than one inch per 100 feet of bridge superstructure length between expansion joints

S_k = angle of skew of support in degrees, measured from a line normal to the span

Δ_{eq} = seismic displacement demand of the long period frame on one side of the expansion.

The minimum seat width calculated above is normal to the face of the abutment and in no case shall be less than 12 in (CT 30 in).

$N_{provided} = 36\,in \quad > \quad 12\,in$ OK



Minimum Seat Width, N =12 in

Figure 21 Minimum Abutment Seat Width

The combined effect of $\Delta_{ot} = \Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp}$ is calculated using the JOINT MOVEMENT CALCULATIONS form form in Appendix O to be 2.5 in. The skew angle of the support, $S_k =$, is 20 $^{\circ}$.

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) displacement occur simultaneously so that:

$$\Delta_{eq,long} = \Delta_{long} + \left(\Delta_{trans}\right)_{long\ component} = 23.58 + \left(23.69 \times \sin(20^0)\right) = 31.68\ in \qquad (CT \quad 21.06)$$

$$\Delta_{normal\ to\ bearing\ centerline} = 31.68 \times \cos(20^0) = 29.77\ in$$

$$\left[N_{req} = (4 + 2.5 + 1.65 \times 29.77)\left(\frac{1+20^2}{4000}\right) = 55.62 \times 0.10 = 5.56\ in \right] < \ 12\ in \qquad (CT \quad 26.14)$$

There appears to be an error in the equation as the skew factor can be equated to 1 if the displacement demands where obtained using full skew effects. Also if $S_k = 0$ (i.e. no skew), then the required seat width is essentially zero. The minimum should at least be the sum of the displacement demands. Using $\left(1 + \frac{S_k^2}{4000}\right)$ instead leads to $N_{req} = 61.18$ in and the abutment seat would need to be lengthened.

This value seems extremely conservative. Revising the maximum seismic demand using the 100% -30% rule results in $\Delta_{normal\ to\ bearing\ centerline} = 26.01 \times \cos(20^0) = 24.44\ in$ and a required seat width of 51.51 in, which is still much larger than 30 in required by Caltrans SDC.

II.I. No Splice Zone

Per LRFD Section 8.8.3, splicing of longitudinal column reinforcement shall be outside the plastic hinging region, L_{pr}, as defined in LRFD Section 4.11.7. This region is defined as the larger of:

- $1.5 \times D_c = 1.5 \times 6\ ft = 9\ ft$
- Region of column where moment exceeds 75% of the maximum plastic moment $\approx 0.25 \times L_{column} = 0.25 \times 47\ ft = 11.75\ ft$
- Analytical plastic hinge length = 61.88 in = 5.2 ft

Ultimate strength splicing of reinforcement shall be used by means of mechanical couplers as approved by Owner.

Caltrans Memo-to-Designers 20-9 deals with the issue of splices in bar reinforcing steel. In general any rebar longer than the standard 60 ft will need to be spliced. The type of splice depends upon whether the component is deemed as “seismic-critical” or not. As defined in MTD 20-9, seismic critical member elements are expected to undergo significant post-elastic deformations during a seismic event. For the prototype bridge, only columns have been designated as “seismic critical” elements.

In our case, maximum length of column rebar can be estimated as

L_{max}= 47.00 + 5.75 = 52.75 ft < 60.00 ft

Therefore, we will specify on the plans that no splices will be permitted for column main rebars. The superstructure rebars, however, will need be spliced. As per MTD20-6, “Service Splice” will be used to splice such rebars. It is good design practice, however, to also specify no splice zone up to the point of contraflexure, determined as per guidelines in MTD 20-6.

APPENDIX – A

(Selective Portions of CTBRIDGE Input)
Input Summary

Cross Section Shape Information

Box Girder 1 data		**Shape:**	**Box Girder**
Overall Width: 706.00 in	Top Slab Thickness:	9.13 in	
Overall Depth: 81.00 in	Bottom Slab Thickness:	8.25 in	

Left Overhang Width:	60.00 in	Right Overhang Width:	60.00 in
Left Overhang Inside Thickness:	12.00 in	Right Overhang Inside Thickness:	12.00 in
Left Overhang Outside Thickness:	8.00 in	Right Overhang Outside Thickness:	8.00 in
Left Exterior Web Offset:	34.50 in	Right Exterior Web Offset:	34.50 in
Left Exterior Web Thickness:	12.00 in	Right Exterior Web Thickness:	12.00 in
Top Fillet Width:	4.00 in	Top Fillet Depth:	4.00 in
Bottom Fillet Width:	0.00 in	Bottom Fillet Depth:	0.00 in
Web Spacing Type:	Symmetrical	Number of Interior Webs:	3
Interior Web Thickness:	12.00 in	Interior Web Distance:	144.00 in
Interior Web 1 Thickness:	12.00 in	Interior Web 1 Distance:	114.50 in
Interior Web 2 Thickness:	12.00 in	Interior Web 2 Distance:	144.00 in
Interior Web 3 Thickness:	12.00 in	Interior Web 3 Distance:	144.00 in

Box Girder 1 properties			Gross	Factor	Cracked
CG to Top	35.24 in	Area:	14902.38 in²	X 1.000	14902.38 in²
CG to Bottom	45.76 in	Ixx:	15160037.03 in4	X 1.000	15160037.03 in4
CG to Left	353.00 in	Iyy:	533747046.76 in4	X 1.000	533747046.76 in4
CG to Right	353.00 in	Torsion:	45022802.56 in4	X 1.000	45022802.56 in4

Box Girder 2 data		**Shape:**	**Box Girder**
Overall Width:	706.00 in	Top Slab Thickness:	9.13 in
Overall Depth:	81.00 in	Bottom Slab Thickness:	12.00 in
Left Overhang Width:	60.00 in	Right Overhang Width:	60.00 in
Left Overhang Inside Thickness:	12.00 in	Right Overhang Inside Thickness:	12.00 in
Left Overhang Outside Thickness:	8.00 in	Right Overhang Outside Thickness:	8.00 in
Left Exterior Web Offset:	34.50 in	Right Exterior Web Offset:	34.50 in
Left Exterior Web Thickness:	12.00 in	Right Exterior Web Thickness:	12.00 in
Top Fillet Width:	4.00 in	Top Fillet Depth:	4.00 in
Bottom Fillet Width:	0.00 in	Bottom Fillet Depth:	0.00 in
Web Spacing Type:	Symmetrical	Number of Interior Webs:	3
Interior Web Thickness:	12.00 in	Interior Web Distance:	144.00 in
Interior Web 1 Thickness:	12.00 in	Interior Web 1 Distance:	114.50 in
Interior Web 2 Thickness:	12.00 in	Interior Web 2 Distance:	144.00 in
Interior Web 3 Thickness:	12.00 in	Interior Web 3 Distance:	144.00 in

Box Girder 2 properties			**Gross**	**Factor**	**Cracked**
CG to Top	38.97 in	Area:	16646.17 in²	X 1.000	16646.17 in²
CG to Bottom	42.03 in	Ixx:	17143559.87 in4	X 1.000	17143559.87 in4
CG to Left	353.00 in	Iyy:	571091004.60 in4	X 1.000	571091004.60 in4
CG to Right	353.00 in	Torsion:	50040498.04 in4	X 1.000	50040498.04 in4

Circle 1 data	**Shape:**	**Circle**
	Diameter:	72.00 in

Circle 1 properties			**Gross**	**Factor**	**Cracked**
CG to Top	36.00 in	Area:	4071.50 in²	X 1.000	4071.50 in²
CG to Bottom	36.00 in	Ixx:	1319167.32 in4	X 1.000	1319167.32 in4
CG to Left	36.00 in	Iyy:	1319167.32 in4	X 1.000	1319167.32 in4
CG to Right	36.00 in	Torsion:	2638334.64 in4	X 1.000	2638334.64 in4

APPENDIX - A

(Selective Portions of CTBRIDGE Input) - Continues

Material Information

Concrete 1 data		**Material:**	**Concrete**
Unit Weight:	0.15 kip/ft	Poisson's Ratio:	0.200
		Elastic Modulus (Ec):	3834.25 ksi
Concrete Strength (f'c):	4.00 ksi	Shear Modulus:	1597.61 ksi
Initial Strength (fci):	3.50 ksi	Initial Modulus (Eci):	3586.62 ksi

Steel 1 data		**Material:**	**Steel**
Unit Weight:	0.49 kip/ft³	Poisson's Ratio:	0.300
		Elastic Modulus (Es):	29000.00 ksi
Yield Strength (fy):	60.00 ksi	Shear Modulus:	11153.85 ksi

Prestress 1 data		**Material:**	**Prestress Steel**
Unit Weight:	0.49 kip/ft³	Poisson's Ratio:	0.300
Ultimate Strength (fpu):	270.00 ksi	Elastic Modulus (Ep):	28500.00 ksi
Yield Strength (fpy):	243.00 ksi	Shear Modulus:	10961.54 ksi

Span Information

Span 1 data

Length: 126.00 ft

Effective Dimensions	
Begin:	1.50 ft
End:	3.00 ft
Model As Link:	Yes

Num	**Distance**	**Section**	**Num**	**Distance**	**Section**
1	Begin	Box Girder 1	3	121.78 ft	Box Girder 2
2	105.78 ft	Box Girder 1	4	End	Box Girder 2

Num	**Distance**	**Material**	**Num**	**Distance**	**Material**
1	Begin	Concrete 1	2	End	Concrete 1

Placement of Results

Evenly spaced: 10

Placement of Nodes

Evenly spaced: 4

Span 2 data

Length: 168.00 ft

Effective Dimensions	
Begin:	3.00 ft
End:	3.00 ft
Model As Link:	Yes

Num	**Distance**	**Section**	**Num**	**Distance**	**Section**
1	Begin	Box Girder 2	4	147.78 ft	Box Girder 1
2	4.22 ft	Box Girder 2	5	163.78 ft	Box Girder 2
3	20.22 ft	Box Girder 1	6	End	Box Girder 2

Num	**Distance**	**Material**	**Num**	**Distance**	**Material**
1	Begin	Concrete 1	2	End	Concrete 1

Placement of Results

Evenly spaced: 10

Placement of Nodes

Evenly spaced: 4

Span 3 data

Length: 118.00 ft

Effective Dimensions
Begin: 3.00 ft
End: 1.50 ft
Model As Link: Yes

Num	Distance	Section
1	Begin	Box Girder 2
2	4.26 ft	Box Girder 2
3	20.22 ft	Box Girder 1
4	End	Box Girder 1

Num	Distance	Material
1	Begin	Concrete 1
2	End	Concrete 1

Placement of Results
Evenly spaced: 10

Placement of Nodes
Evenly spaced: 4

Column Type Information

Column Type 1 data

Datum: Bottom

Num	Distance	Section
1	Bottom	Circle 1
2	Top	Circle 1

Num	Distance	Material
1	Bottom	Concrete 1
2	Top	Concrete 1

Placement of Results
Evenly spaced: 4

Placement of Nodes
Evenly spaced: 4

Bent Information

Bent 2 data

Skew Angle: 20.0000 °
Continuous Connection — Condition: Fix

Bent 2, Column 1 — **Dist in Bent -17.00 ft**
Rotation: 0.0000 °
Column Type: Column Type 1

Column top placed at bent bottom — Top Elev: 82.02 ft
Bot Elev: 38.02 ft
Height defined by column length — Length: 44.00 ft
Bottom Condition: Pin

Bent 2, Column 2 — **Dist in Bent 17.00 ft**
Rotation: 0.0000 °
Column Type: Column Type 1

Column top placed at bent bottom — Top Elev: 82.02 ft
Bot Elev: 38.02 ft
Height defined by column length — Length: 44.00 ft
Bottom Condition: Pin

APPENDIX - A
(Selective Portions of CTBRIDGE Input) - Continues

Bent 3 data

	Skew Angle:	20.0000 °
Continuous Connection	Condition:	Fix
Bent 3, Column 1	**Dist in Bent**	**-17.00 ft**
	Rotation:	0.0000 °
Column Type:	Column Type 1	
Column top placed at bent bottom	Top Elev:	84.70 ft
	Bot Elev:	37.70 ft
Height defined by column length	Length:	47.00 ft
Bottom Condition: Pin		
Bent 3, Column 2	**Dist in Bent**	**17.00 ft**
	Rotation:	0.0000 °
Column Type:	Column Type 1	
Column top placed at bent bottom	Top Elev:	84.70 ft
	Bot Elev:	37.70 ft
Height defined by column length	Length:	47.00 ft
Bottom Condition: Pin		

Support Information

Abut 1 data

Skew Type: Skew Angle: 20.0000 °
Connection to Span(s): Continuous Spans Connection Type: Roller

Abut 4 data

Skew Type: Skew Angle: 20.0000 °
Connection to Span(s): Continuous Spans Connection Type: Roller

Dead Load

Dead load is active **Self weight is applied**

Added Dead Load

Additional dead load is active **Wearing surface is applied**

Wearing Surface: 35.00 psf Deck Width: 56.00 ft

Load Name	**Start Magnitude**	**End Magnitude**	**Start Distance**	**End Distance**	**Load Type**	**Load Direction**	**Applied To**
Type 732 Barrier ...	0.82 kip/ft	0.82 kip/ft	0.000 ratio	1.000 ratio	Distributed Force	Gravity	Span 1 Span 2 Span 3

APPENDIX – B

(Selective Portions of CTBRIDGE Output)

Dead Load - Unfactored Forces - Columns

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
0.00	-1445.2	13.6	1.0	0.0	0.0	-0.0
11.00	-1398.6	13.6	1.0	0.0	11.2	-149.6
22.00	-1351.9	13.6	1.0	0.0	22.4	-299.1
33.00	-1305.3	13.6	1.0	0.0	33.6	-448.7
44.00	**-1258.6**	13.6	1.0	0.0	44.8	-598.3

Bent 2, Column 2

Location ft	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
0.00	-1448.8	13.4	1.0	0.0	0.0	-0.0
11.00	-1402.1	13.4	1.0	0.0	11.5	-147.7
22.00	-1355.5	13.4	1.0	0.0	23.0	-295.4
33.00	-1308.8	13.4	1.0	0.0	34.4	-443.1
44.00	**-1262.1**	13.4	1.0	0.0	45.9	-590.8

Bent 3, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
0.00	-1403.6	-13.9	-0.0	0.0	-0.0	0.0
11.75	-1353.8	-13.9	-0.0	0.0	-0.2	163.2
23.50	-1303.9	-13.9	-0.0	0.0	-0.4	326.4
35.25	-1254.1	-13.9	-0.0	0.0	-0.7	489.5
47.00	**-1204.3**	-13.9	-0.0	0.0	-0.9	652.7

Bent 3, Column 2

Location ft	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
0.00	-1427.3	-13.9	0.0	0.0	0.0	0.0
11.75	-1377.5	-13.9	0.0	0.0	0.0	163.1
23.50	-1327.6	-13.9	0.0	0.0	0.1	326.2
35.25	-1277.8	-13.9	0.0	0.0	0.1	489.3
47.00	**-1228.0**	-13.9	0.0	0.0	0.1	652.4

Dead Load - Unfactored Bent Reactions

Bent Location	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
Bent 2 Col Bots	-2894.0	27.0	2.1	0.0	0.0	-0.0
Bent 2 Col Tops	-2520.8	27.0	2.1	0.0	90.7	-1189.0
Bent 3 Col Bots	-2830.9	-27.8	-0.0	0.0	-0.0	0.0
Bent 3 Col Tops	-2432.2	-27.8	-0.0	0.0	-0.8	1305.1

Additional Dead Load - Unfactored Forces - Columns

Bent 2, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
0.00	-230.6	2.4	0.2	0.0	0.0	-0.0
11.00	-230.6	2.4	0.2	0.0	2.1	-26.8
22.00	-230.6	2.4	0.2	0.0	4.3	-53.7
33.00	-230.6	2.4	0.2	0.0	6.4	-80.5
44.00	**-230.6**	2.4	0.2	0.0	8.5	-107.3

Bent 2, Column 2

Location ft	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
0.00	-231.3	2.4	0.2	0.0	0.0	-0.0
11.00	-231.3	2.4	0.2	0.0	2.2	-26.5
22.00	-231.3	2.4	0.2	0.0	4.4	-53.0
33.00	-231.3	2.4	0.2	0.0	6.6	-79.5
44.00	**-231.3**	2.4	0.2	0.0	8.7	-106.0

Bent 3, Column 1

Location ft	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
0.00	-221.0	-2.5	-0.0	0.0	-0.0	0.0
11.75	-221.0	-2.5	-0.0	0.0	-0.2	29.3
23.50	-221.0	-2.5	-0.0	0.0	-0.3	58.6
35.25	-221.0	-2.5	-0.0	0.0	-0.5	87.9
47.00	**-221.0**	-2.5	-0.0	0.0	-0.6	117.1

Bent 3, Column 2

Location ft	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
0.00	-225.2	-2.5	-0.0	0.0	-0.0	0.0
11.75	-225.2	-2.5	-0.0	0.0	-0.1	29.3
23.50	-225.2	-2.5	-0.0	0.0	-0.2	58.5
35.25	-225.2	-2.5	-0.0	0.0	-0.3	87.8
47.00	**-225.2**	-2.5	-0.0	0.0	-0.5	117.1

Additional Dead Load - Unfactored Bent Reactions

Bent	Location	AX kip	VY kip	VZ kip	TX kip·ft	MY kip·ft	MZ kip·ft
Bent 2	Col Bots	-461.9	4.8	0.4	0.0	0.0	-0.0
Bent 2	Col Tops	-461.9	4.8	0.4	0.0	17.3	-213.4
Bent 3	Col Bots	-446.2	-5.0	-0.0	0.0	-0.0	0.0
Bent 3	Col Tops	-446.2	-5.0	-0.0	0.0	-1.1	234.2

APPENDIX – C

(Input file for xSECTION)

```
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
BRIDGE_NUMBER
99-9999
JOB_TITLE
PROTYPE BRIDGE BENT 2 DL ONLY - LRFD
***********************************************************
*                      6' Dia. Column                     *
*                     Bent 2 (DL Only)                    *
*                     Prototype Bridge                    *
*                                                         *
*                                     8/18/06             *
***********************************************************
Subsection definition is supported by coordinates
bending parallel to x-axis (horiz.)
local x- and y- axes parallel to global X- and YUnits
are Kips and inches
**************************************************************
* Welded hoops or seismic hooks are required for confinement *
* to be effective.                                            *
**************************************************************
CONC_TYPES_START
NUMBER_OF_TYPES 2
TYPE_NUMBER 1 MODEL mander
   CONFINED_SUBSECTION_SHAPE circular
       CONFINED_SUBSECTION_DIAM 68.00
       CONF_TYPE hoops
               CONF_STEEL_TYPE 1 CONF_BAR_AREA 0.79 CONF_BAR_DIAM 1.00
       CONF_BAR_SPACING 5.0
       MAIN_BAR_TOTAL 26 MAIN_BAR_AREA 2.25
    STRAIN_e0 0.002 STRAIN_eu 0.005 ULT_STRAIN_FACT 1.0
    STRESS_f0 5.20 STRESS_fu 2.60
    UNIT_WEIGHT_FACT 0.986
TYPE_NUMBER 2 MODEL unconfined_mander
    STRAIN_e0 0.002 STRAIN_eu 0.005 ULT_STRAIN_FACT 1.0
    STRESS_f0 5.20 STRESS_fu 2.60
    UNIT_WEIGHT_FACT 0.986
CONC_TYPES_END
***************************************************************
* A706 Steel type 2 is for #14 bars. Type 1 is for #8 bars.   *
***************************************************************
STEEL_TYPES_START
NUMBER_OF_TYPES 2
TYPE_NUMBER 1 MODEL park
YIELD_STRAIN 0.00228 HARDEN_STRAIN 0.0150 ULT_STRAIN 0.120
YIELD_STRESS 66.0 ULT_STRESS 92.0
MODULUS 29000.0
TYPE_NUMBER 2 MODEL park
YIELD_STRAIN 0.00228 HARDEN_STRAIN 0.0075 ULT_STRAIN 0.06
YIELD_STRESS 66.0 ULT_STRESS 92.0
MODULUS 29000.0
STEEL_TYPES_END
```

APPENDIX – C

(Input file for xSECTION) – Continues

```
*************************************************
* Comment area                                  *
* Arc_strip is used to model a full circle.     *
*************************************************
SUBSECTION_START
NUMBER_OF_SUBSECTIONS 2
SUBSECTION_NUMBER 1
    SHAPE arc_strip
    CENTER_GLOBAL_X_Y 0 0 START_ANGLE 0 DURATION_CCW 360
    RADIUS_OUTER 34.00 RADIUS_INNER 0
    NUMBER_OF_FIBERS_RADIAL 10 NUMBER_OF_FIBERS_ANGULAR 40
    CONC_TYPE 1
    MIRROR_4_WAYS no
SUBSECTION_NUMBER 2
    SHAPE arc_strip
    CENTER_GLOBAL_X_Y 0 0 START_ANGLE 0 DURATION_CCW 360
    RADIUS_OUTER 36.00 RADIUS_INNER 34.00
    NUMBER_OF_FIBERS_RADIAL 1 NUMBER_OF_FIBERS_ANGULAR 50
    CONC_TYPE 2
    MIRROR_4_WAYS no
SUBSECTION_END
*********************************************************
* Comment area                                          *
* Circular rebar distribution is a special case of arc  *
* distribution                                          *
*********************************************************
REBAR_LAYOUT_START
NUMBER_OF_REBAR_GROUPS 1
GROUP_NUMBER 1
    LAYOUT_SHAPE circular
    NUMBER_OF_REBARS 26 AREA_OF_EACH_BAR 2.25 STEEL_TYPE 2
    CENTER_GLOBAL_X_Y 0 0 START_ANGLE 0 DURATION_CCW 360
    RADIUS 31.930
    MIRROR_4_WAYS no
REBAR_LAYOUT_END
********************************************************
AXIAL_LOAD
    LOAD_VALUE 1694
    CENTER_OF_LOAD_APPLICATION_GLOBAL_X_Y 0 0
***************************************************************
* Comment area                                                *
* Let the cover concrete fail but stop at first longitudinal  *
* rebar failure.To control the initial guess of the Neutral   *
* Axis a factor is defined which varies from 0.01 to 0.99     *
* as shown below. This is used if there is instablity in      *
* the moment-curvature curve.                                 *
***************************************************************
ANALYSIS_CONTROL
    STOP_DUE_FIRST_CONC_FAILURE no
    STOP_DUE_FIRST_REBAR_FAILURE yes
    BENDING_AXIS_CCW_ROTATION_DEGREES 0
    NEUTRAL_AXIS_PROXIMITY_TO_COMPRESSION_EDGE 0.99
    CONVERGENCE_TOLERANCE 0.001
*********************************************************
RESULTS_REQUESTED
    MOMENT_AT_GLOBAL_X_Y 0 0
    CONC_FIBER_INFO_OUTPUT no
    REBAR_FIBER_INFO_OUTPUT yes
*********************************************************
```

APPENDIX – D

(Output from xSECTION)

```
08/18/2006, 13:00
************************************************************
*                                                          *
*                       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 BENT 2 DL ONLY - LRFD



Concrete Type Information:

     ----------strains--------  --------strength--------
Type   e0     e2     ecc     eu     f0    f2    fcc    fu    E    W
 1 0.0020 0.0040 0.0055 0.0185  5.20  6.86  7.02  5.56  4280   148
 2 0.0020 0.0040 0.0020 0.0050  5.20  3.58  5.20  2.60  4280   148


Steel Type Information:

     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0150 0.1200 66.00 92.00 29000
 2 0.0023 0.0075 0.0600 66.00 92.00 29000


Steel Fiber Information:

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

7	2	3.85	31.70	2.25
8	2	-3.85	31.70	2.25
9	2	-11.32	29.86	2.25
10	2	-18.14	26.28	2.25
11	2	-23.90	21.17	2.25
12	2	-28.27	14.84	2.25
13	2	-31.00	7.64	2.25
14	2	-31.93	0.00	2.25
15	2	-31.00	-7.64	2.25
16	2	-28.27	-14.84	2.25
17	2	-23.90	-21.17	2.25
18	2	-18.14	-26.28	2.25
19	2	-11.32	-29.86	2.25
20	2	-3.85	-31.70	2.25
21	2	3.85	-31.70	2.25
22	2	11.32	-29.85	2.25
23	2	18.14	-26.28	2.25
24	2	23.90	-21.17	2.25
25	2	28.27	-14.84	2.25
26	2	31.00	-7.64	2.25

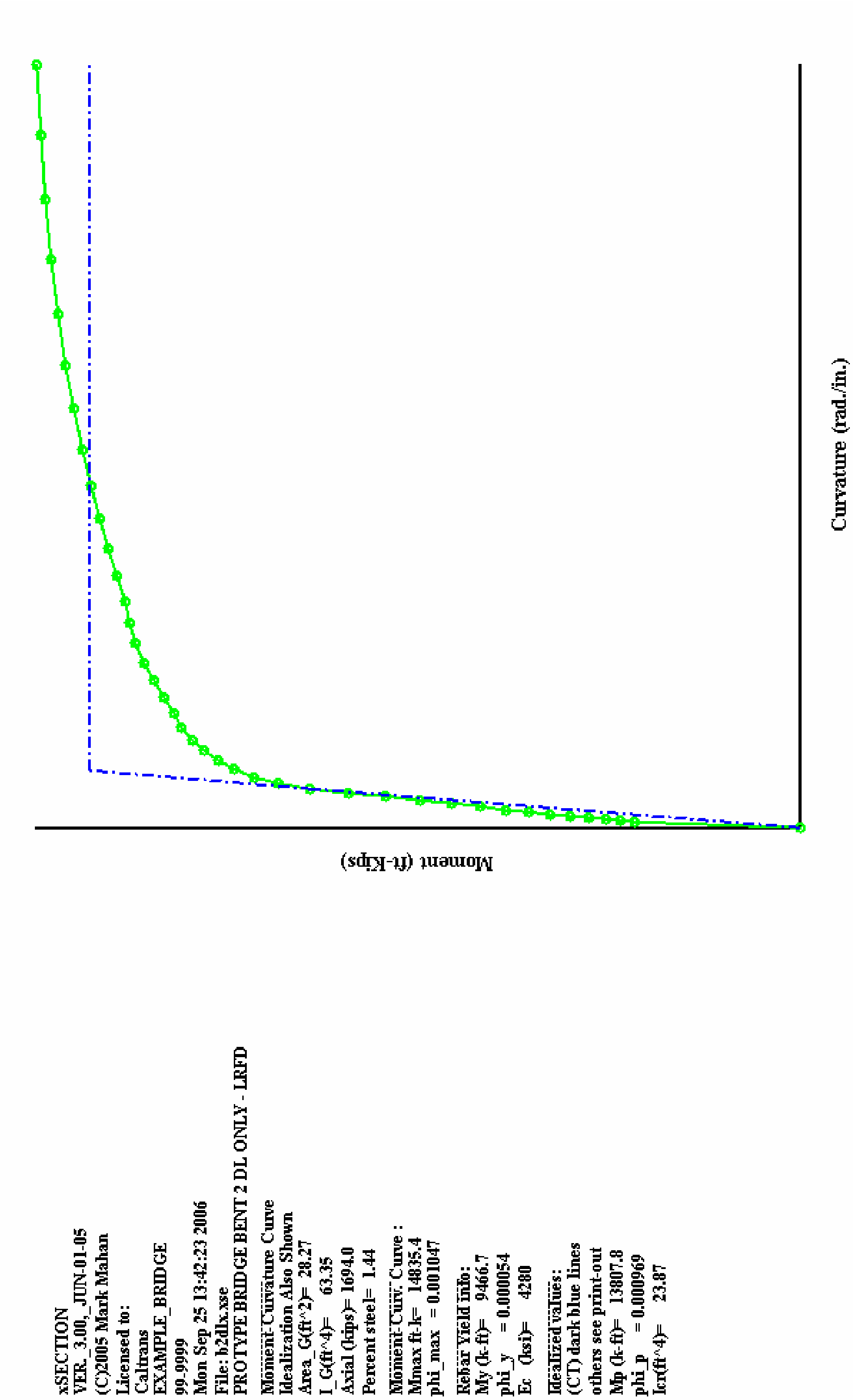
Force Equilibrium Condition of the x-section:

step	Max. Conc. Strain epscmax	Neutral Axis in.	Max. Steel Strain Tens.	Conc. Comp.	Steel force Comp.	Steel force Tens.	P/S force	Net force	Curvature rad/in	Moment (K-ft)
0	0.00000	0.00	0.0000	0	0	0	0	0.00	0.000000	0
1	0.00037	-5.35	-0.0002	1614	197	-116	0	0.96	0.000009	3204
2	0.00041	-2.98	-0.0003	1642	209	-156	0	0.77	0.000010	3487
3	0.00045	-0.80	-0.0004	1677	220	-203	0	-0.59	0.000012	3786
4	0.00050	1.07	-0.0005	1720	234	-260	0	-0.36	0.000014	4109
5	0.00055	2.68	-0.0006	1770	250	-325	0	0.18	0.000017	4462
6	0.00061	4.21	-0.0007	1830	267	-404	0	-1.09	0.000019	4845
7	0.00068	5.49	-0.0008	1900	285	-491	0	0.08	0.000022	5266
8	0.00075	6.61	-0.0010	1978	305	-590	0	-0.87	0.000025	5726
9	0.00083	7.57	-0.0011	2068	326	-700	0	0.59	0.000029	6231
10	0.00091	8.45	-0.0013	2169	353	-828	0	-0.31	0.000033	6786
11	0.00101	9.21	-0.0015	2284	383	-972	0	1.32	0.000038	7395
12	0.00112	9.86	-0.0018	2410	415	-1130	0	1.39	0.000043	8057
13	0.00123	10.42	-0.0020	2546	452	-1304	0	-0.73	0.000048	8775
14	0.00136	10.89	-0.0023	2692	492	-1490	0	-0.40	0.000054	9539
15	0.00151	11.54	-0.0027	2808	532	-1645	0	0.84	0.000062	10134
16	0.00167	12.32	-0.0031	2899	572	-1777	0	0.08	0.000070	10602
17	0.00184	13.09	-0.0036	2974	613	-1894	0	-1.16	0.000080	10996
18	0.00204	13.90	-0.0042	3027	654	-1987	0	-0.56	0.000092	11303
19	0.00225	14.66	-0.0049	3079	696	-2081	0	0.30	0.000106	11594
20	0.00249	15.44	-0.0057	3105	747	-2159	0	-1.41	0.000121	11805
21	0.00275	16.10	-0.0066	3138	805	-2248	0	0.51	0.000138	12026
22	0.00304	16.75	-0.0077	3147	855	-2308	0	0.24	0.000158	12172
23	0.00336	17.21	-0.0088	3175	897	-2377	0	0.64	0.000179	12361
24	0.00372	17.53	-0.0099	3229	920	-2455	0	-1.19	0.000202	12553
25	0.00411	17.78	-0.0112	3289	944	-2539	0	-0.04	0.000226	12762
26	0.00454	18.02	-0.0126	3321	971	-2597	0	1.25	0.000253	12927
27	0.00502	18.14	-0.0140	3342	1003	-2651	0	-0.03	0.000281	13037
28	0.00555	18.13	-0.0155	3381	1015	-2702	0	-0.05	0.000311	13124
29	0.00614	18.18	-0.0172	3431	1026	-2763	0	-0.37	0.000345	13269
30	0.00678	18.24	-0.0191	3484	1037	-2827	0	0.10	0.000383	13433
31	0.00750	18.29	-0.0212	3540	1051	-2895	0	1.24	0.000424	13606
32	0.00829	18.32	-0.0235	3596	1066	-2968	0	0.22	0.000470	13780
33	0.00917	18.33	-0.0260	3650	1084	-3040	0	0.43	0.000519	13947
34	0.01013	18.34	-0.0288	3702	1105	-3113	0	-0.42	0.000575	14111
35	0.01120	18.34	-0.0318	3748	1134	-3186	0	1.15	0.000635	14270
36	0.01239	18.38	-0.0352	3767	1166	-3239	0	-0.35	0.000704	14414
37	0.01369	18.41	-0.0391	3779	1203	-3289	0	-0.54	0.000779	14552
38	0.01514	18.41	-0.0432	3794	1233	-3334	0	-0.75	0.000862	14668

```
39   0.01673   18.36 -0.0475  3819  1247 -3373     0     -0.61 0.000950   14755
40   0.01850   18.32 -0.0524  3836  1268 -3408     0      1.49 0.001047   14835


First Yield of Rebar Information (not Idealized):

     Rebar Number 20
     Coordinates X and Y (global in.)  -3.85, -31.70
     Yield strain =  0.00228
     Curvature (rad/in)=  0.000054
     Moment (ft-k) =    9467


Cross Section Information:

     Axial Load on Section (kips) =  1694
     Percentage of Main steel in Cross Section =   1.44
     Concrete modulus used in Idealization (ksi) =  4280
     Cracked Moment of Inertia (ft^4) =    23.872


Idealization of Moment-Curvature Curve by Various Methods:


                  Points on Curve        Idealized Values
                  ===============   ============================
Method     Conc.                    Yield         symbol  Plastic
 ID      | Strain  Curv.   Moment | Curv.  Moment   for   Curv.
         | in/in   rad/in  (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000155   12151 0.000069   12151   Mn  0.000979
Strain @ 0.004   0.000219   12704 0.000072   12704   Mn  0.000976
Strain @ 0.005   0.000280   13032 0.000074   13032   Mn  0.000974
CALTRANS 0.00844 0.000478   13808 0.000078   13808   Mp  0.000969
UCSD@5phy0.00480 0.000268   12985 0.000074   12985   Mn  0.000974
```

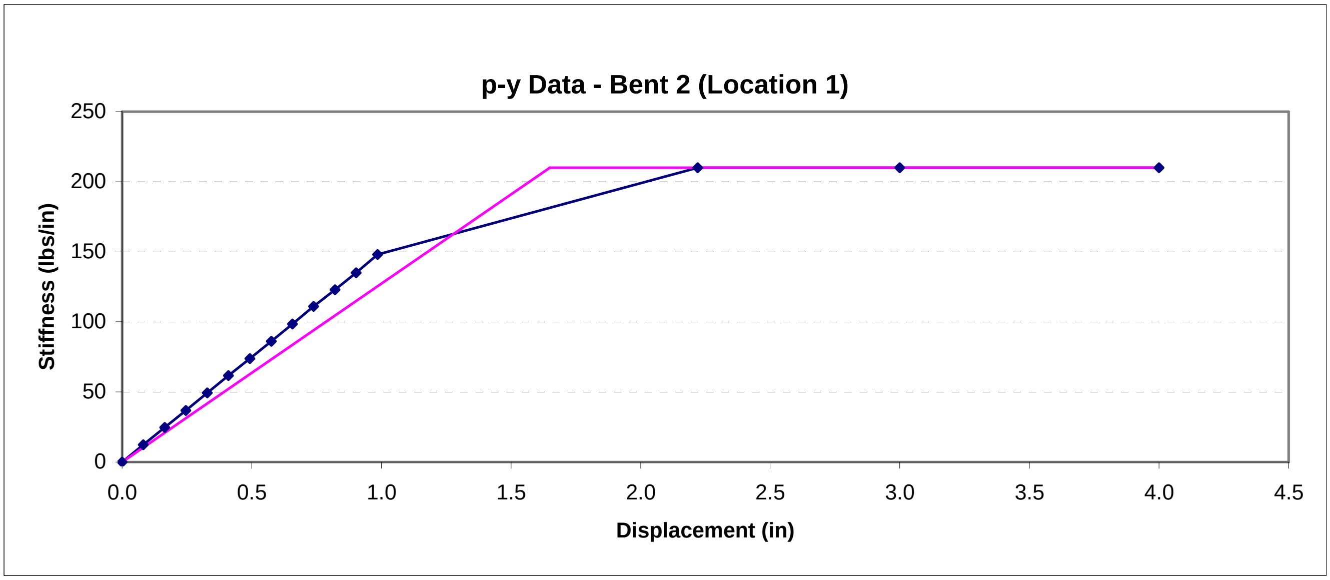
APPENDIX – E

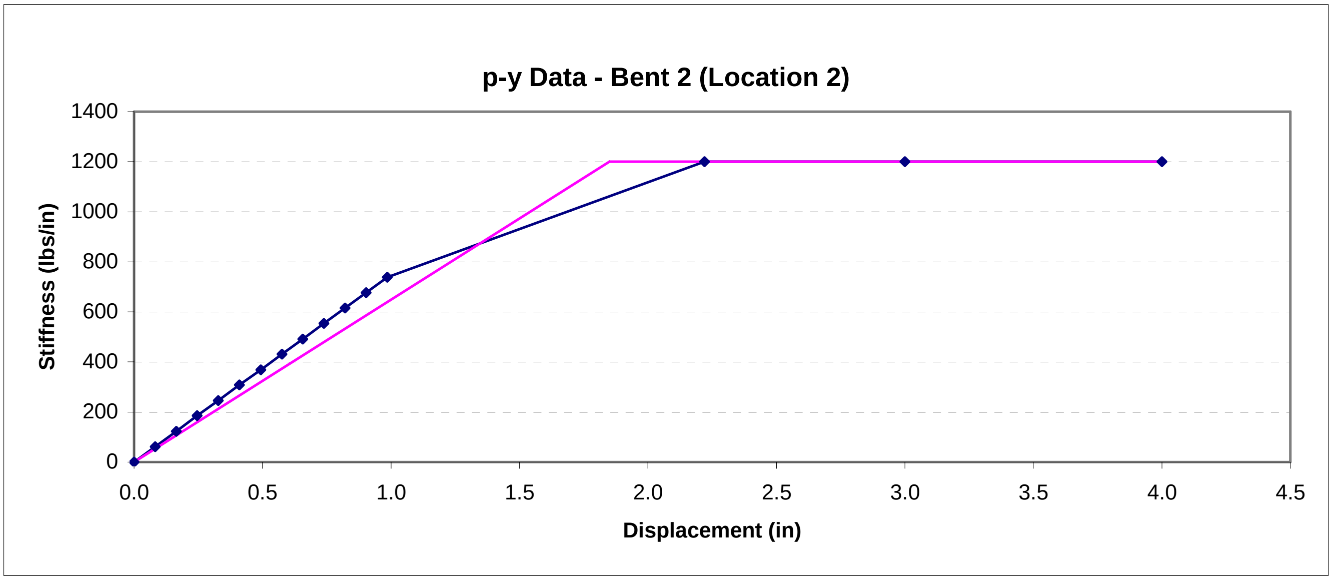
(Moment – Curvature Relationship)

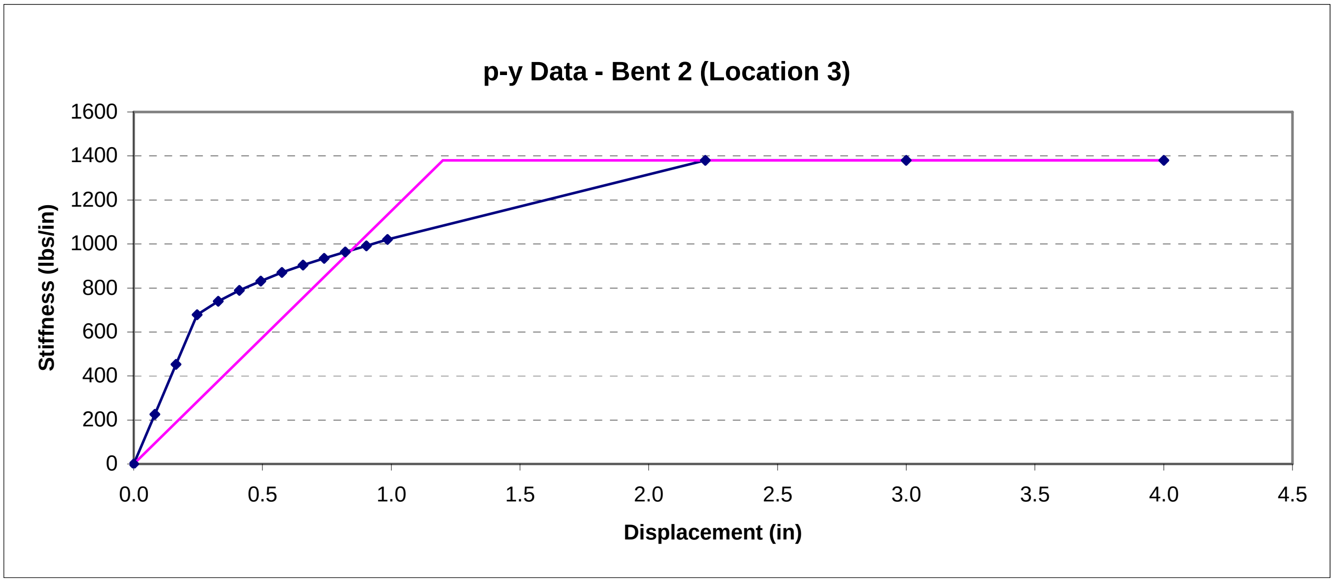


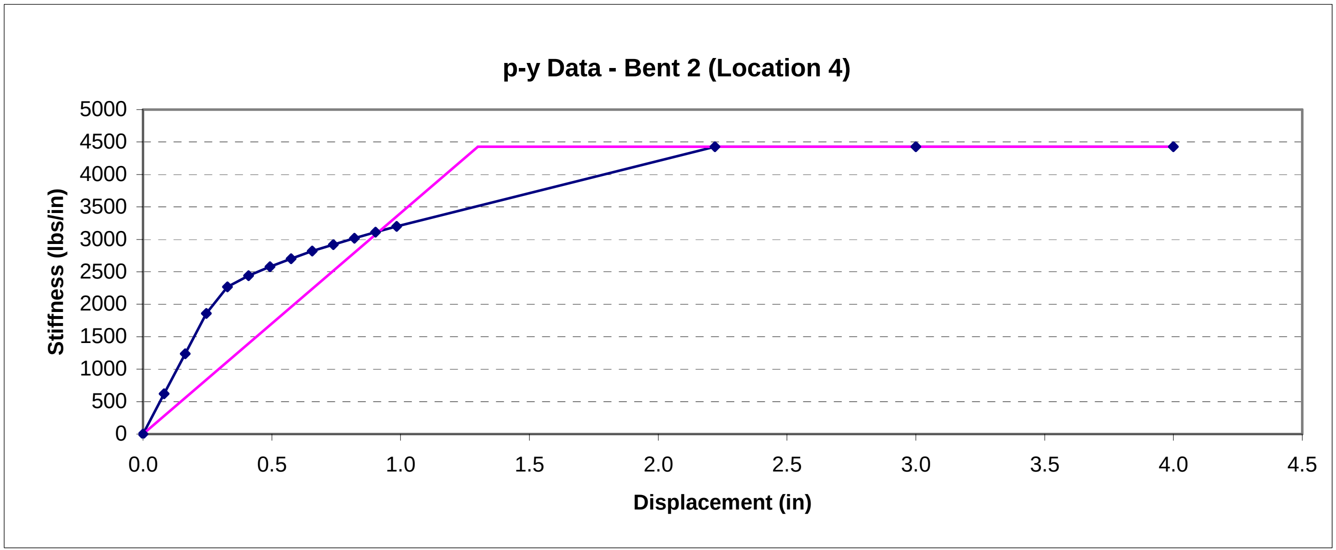
APPENDIX – F

(Soil Spring Data)

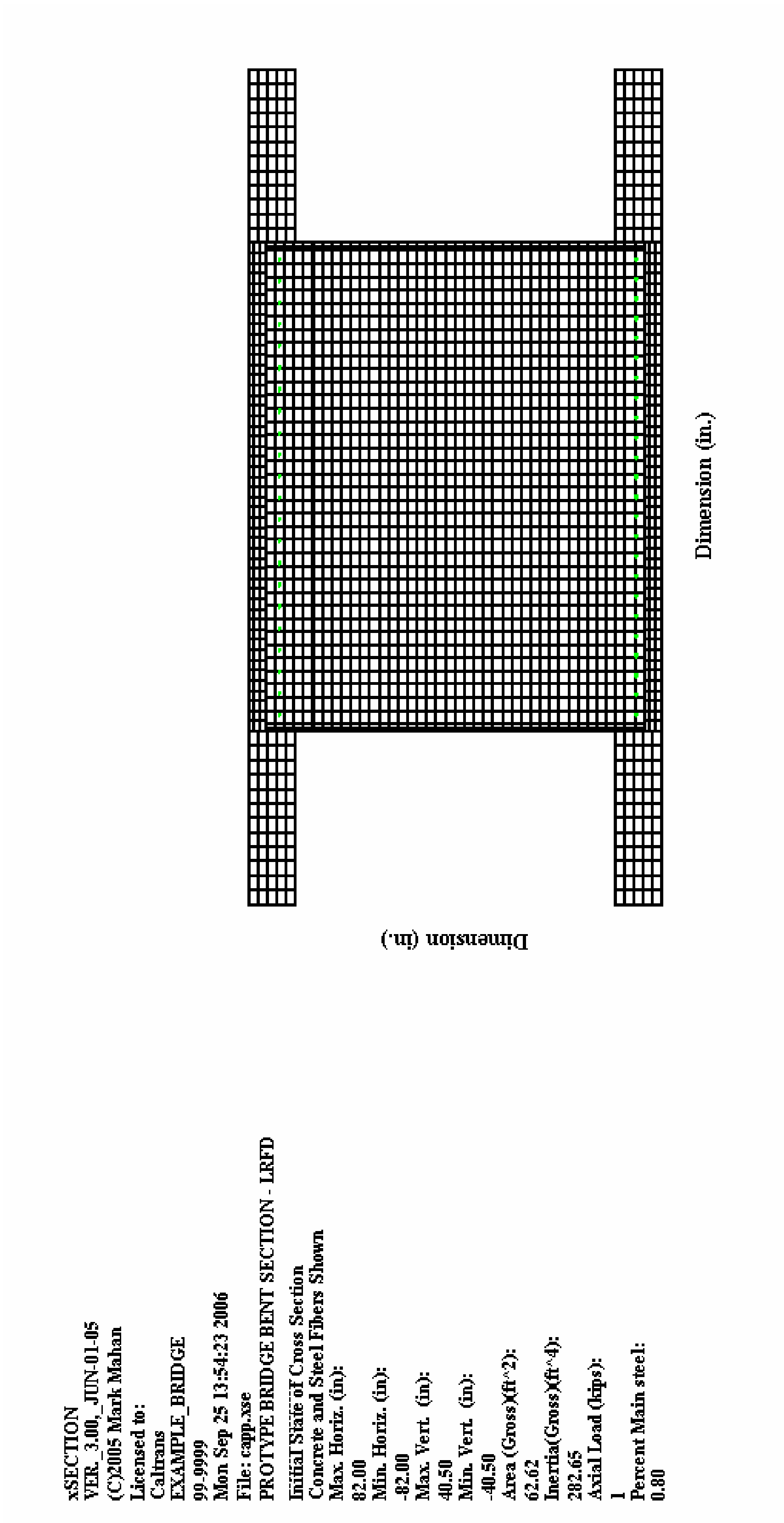





p-y Data - Bent 2 (Location 3)
Stiffness (lbs/in)
1600
1400
1200
1000
800
600
400
200
0
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
Displacement (in)


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

APPENDIX – G
(Bent Cap – xSECTION Model)



APPENDIX – H1

(Bent Cap – Positive Bending Section Capacities)

```
09/18/2006, 10:51
************************************************************
*                                                          *
*                      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.      *
*                                                          *
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* For GOVERNMENT work call 916-227-8404, otherwise leave a *
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* In no event shall the author be held liable for          *
* incidental or consequential damages arising out of the   *
* use of this program.                                     *
*                                                          *
************************************************************
This output was generated by running:
xSECTION
VER._2.40,_MAR-14-99
LICENSE    (choices: LIMITED/UNLIMITED)
UNLIMITED
ENTITY     (choices: GOVERNMENT/CONSULTANT)
GOVERNMENT
NAME_OF_FIRM
CALTRANS
BRIDGE_NAME
EXAMPLE
BRIDGE_NUMBER
99-9999
JOB_TITLE
PROTYPE BRIDGE BENT SECTION - LRFD



Concrete Type Information:

     ----------strains--------  --------strength--------
Type   e0    e2     ecc     eu     f0    f2    fcc    fu    E    W
 1 0.0020 0.0040 0.0027 0.0145  5.20  5.19  5.56  2.18  4283   148
 2 0.0020 0.0040 0.0020 0.0050  5.20  3.59  5.20  2.50  4283   148


Steel Type Information:

     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0150 0.1200 66.00 92.00 29000
 2 0.0023 0.0115 0.0900 66.00 92.00 29000


--------------------------------------------
--------------------------------------------
--------------------------------------------
--------------------------------------------
--------------------------------------------
--------------------------------------------
--------------------------------------------
```

Force Equilibrium Condition of the x-section:

step	Max. Conc. Strain epscmax	Neutral Axis in.	Max. Steel Strain Tens.	Conc. Comp.	Steel force Comp.	Steel force Tens.	P/S force	Net force	Curvature rad/in	Moment (K-ft)
0	0.00000	0.00	0.0000	0	0	0	0	0.00	0.000000	0
1	0.00029	27.04	-0.0013	1304	160	-1463	0	0.00	0.000022	8728
2	0.00032	27.04	-0.0015	1441	177	-1617	0	0.00	0.000024	9645
3	0.00035	27.03	-0.0016	1592	195	-1786	0	0.00	0.000026	10656
-----	-----	-----	-----	-----	-----	-----				
12	0.00087	32.77	-0.0077	2279	193	-2471	0	0.00	0.000113	15052
13	0.00097	33.36	-0.0093	2320	152	-2471	0	0.00	0.000135	15093
14	0.00107	33.81	-0.0111	2364	108	-2471	0	0.00	0.000160	15137
15	0.00118	34.06	-0.0128	2459	78	-2536	0	0.00	0.000183	15567
16	0.00131	34.26	-0.0146	2577	47	-2623	0	0.00	0.000209	16123
17	0.00144	34.45	-0.0167	2701	9	-2709	0	0.00	0.000238	16679
18	0.00160	34.63	-0.0191	2836	0	-2835	0	0.00	0.000272	17233
19	0.00176	34.78	-0.0217	2969	0	-2968	0	0.00	0.000308	17759
20	0.00195	34.89	-0.0245	3097	0	-3096	0	0.00	0.000348	18250
21	0.00216	34.97	-0.0275	3218	0	-3217	0	0.00	0.000390	18701
22	0.00238	35.03	-0.0307	3329	0	-3328	0	0.00	0.000436	19109
23	0.00264	35.06	-0.0342	3428	0	-3427	0	0.01	0.000484	19471
24	0.00291	35.06	-0.0378	3512	0	-3511	0	0.00	0.000536	19786
25	0.00322	35.04	-0.0416	3578	0	-3577	0	0.00	0.000590	20056
26	0.00356	35.00	-0.0456	3624	0	-3623	0	0.01	0.000647	20280
27	0.00394	34.94	-0.0498	3647	0	-3646	0	0.00	0.000708	20462
28	0.00435	34.86	-0.0543	3646	0	-3645	0	0.00	0.000772	20604
29	0.00481	34.67	-0.0579	3531	0	-3530	0	0.00	0.000825	20627
30	0.00532	34.32	-0.0601	3249	139	-3387	0	0.00	0.000861	20501
31	0.00588	34.08	-0.0637	3030	373	-3402	0	0.00	0.000915	20404
32	0.00650	33.99	-0.0693	2926	495	-3420	0	0.00	0.000998	20421

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)= 0.000036
Moment (ft-k) = 14292

Cross Section Information:

Axial Load on Section (kips) = 1
Percentage of Main steel in Cross Section = 0.80
Concrete modulus used in Idealization (ksi) = 4283
Cracked Moment of Inertia (ft^4) = 52.948

Idealization of Moment-Curvature Curve by Various Methods:

Method ID	Conc. Strain in/in	Points on Curve: Curv. rad/in	Points on Curve: Moment (K-ft)	Idealized Values: Yield Curv. rad/in	Idealized Values: Moment (K-ft)	Idealized Values: symbol for moment	Idealized Values: Plastic Curv. rad/in
Strain @	0.003	0.000551	19862	0.000051	19862	Mn	0.000947
Strain @	0.004	0.000718	20484	0.000052	20484	Mn	0.000946
Strain @	**0.005**	**0.000839**	**20580**	**0.000053**	**20580**	**Mn**	**0.000945**
CALTRANS	0.00220	0.000399	18780	0.000048	18780	Mp	0.000950
UCSD@5phy	0.00118	0.000182	15548	0.000040	15548	Mn	0.000958

APPENDIX – H2

(Bent Cap – Negative Bending Section Capacities)

```
09/18/2006, 10:52
************************************************************
*                                                          *
*                       xSECTION                           *
*                                                          *
*               DUCTILITY and STRENGTH of                  *
*     Circular, Semi-Circular, full and partial Rings,     *
*  Rectangular, T-, I-, Hammer head, Octagonal, Polygons   *
*      or any combination of above shapes forming          *
*         Concrete Sections using Fiber Models             *
*                                                          *
* VER._2.40,_MAR-14-99                                     *
*                                                          *
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*                                                          *
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* In no event shall the author be held liable for          *
* incidental or consequential damages arising out of the   *
* use of this program.                                     *
*                                                          *
************************************************************
This output was generated by running:
xSECTION
VER._2.40,_MAR-14-99
LICENSE    (choices: LIMITED/UNLIMITED)
UNLIMITED
ENTITY     (choices: GOVERNMENT/CONSULTANT)
GOVERNMENT
NAME_OF_FIRM
CALTRANS
BRIDGE_NAME
EXAMPLE
BRIDGE_NUMBER
99-9999
JOB_TITLE
PROTYPE BRIDGE - BRIDGE DESIGN ACADEMY
```

Concrete Type Information:

	----------strains--------				--------strength--------					
Type	e0	e2	ecc	eu	f0	f2	fcc	fu	E	W
1	0.0020	0.0040	0.0027	0.0145	5.20	5.19	5.56	2.18	4283	148
2	0.0020	0.0040	0.0020	0.0050	5.20	3.59	5.20	2.50	4283	148

Steel Type Information:

	-----strains------			--strength-		
Type	ey	eh	eu	fy	fu	E
1	0.0023	0.0150	0.1200	66.00	92.00	29000
2	0.0023	0.0115	0.0900	66.00	92.00	29000

```
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---------------------------------------------
---------------------------------------------
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```

APPENDIX – H2

Force Equilibrium Condition of the x-section:

step	Max. Conc. Strain epscmax	Neutral Axis in.	Max. Steel Strain Tens.	Conc. Comp.	Steel force Comp.	Steel force Tens.	P/S force	Net force	Curvature rad/in	Moment (K-ft)
0	0.00000	0.00	0.0000	0	0	0	0	0.00	0.000000	0
1	0.00029	27.89	-0.0014	1240	190	-1428	0	0.00	0.000023	8431
2	0.00032	27.89	-0.0016	1370	210	-1578	0	0.00	0.000025	9317
3	0.00035	27.88	-0.0018	1513	232	-1744	0	0.00	0.000028	10294
4	0.00039	27.88	-0.0019	1671	256	-1927	0	0.00	0.000031	11372
5	0.00043	27.87	-0.0021	1846	284	-2128	0	0.00	0.000034	12561
6	0.00048	28.41	-0.0025	1962	304	-2265	0	0.00	0.000040	13390
7	0.00053	29.74	-0.0032	1959	307	-2265	0	0.00	0.000049	13444
8	0.00059	30.78	-0.0039	1959	308	-2265	0	0.00	0.000060	13496
9	0.00065	31.70	-0.0049	1964	302	-2265	0	0.00	0.000074	13540
10	0.00072	32.40	-0.0059	1971	296	-2265	0	0.00	0.000088	13586
11	0.00079	33.12	-0.0072	1991	275	-2265	0	0.00	0.000107	13621
12	0.00087	33.69	-0.0087	2016	250	-2265	0	0.00	0.000128	13657
13	0.00097	34.12	-0.0104	2042	224	-2265	0	0.00	0.000152	13694
14	0.00107	34.48	-0.0122	2108	194	-2301	0	0.00	0.000177	13936
15	0.00118	34.73	-0.0142	2219	168	-2386	0	0.00	0.000205	14471
16	0.00131	34.94	-0.0163	2333	139	-2470	0	0.00	0.000235	14995
17	0.00144	35.12	-0.0187	2446	106	-2551	0	0.00	0.000268	15504
18	0.00160	35.27	-0.0213	2558	71	-2629	0	0.00	0.000305	15990
19	0.00176	35.39	-0.0241	2668	35	-2702	0	0.00	0.000345	16448
20	0.00195	35.49	-0.0272	2772	0	-2771	0	0.00	0.000389	16874
21	0.00216	35.56	-0.0306	2870	0	-2869	0	0.00	0.000437	17263
22	0.00238	35.61	-0.0342	2958	0	-2957	0	0.00	0.000488	17613
23	0.00264	35.64	-0.0381	3035	0	-3034	0	0.00	0.000543	17922
24	0.00291	35.66	-0.0422	3099	0	-3098	0	0.00	0.000602	18190
25	0.00322	35.65	-0.0466	3147	0	-3146	0	0.00	0.000665	18416
26	0.00356	35.64	-0.0513	3180	0	-3179	0	0.00	0.000732	18603
27	0.00394	35.61	-0.0564	3195	0	-3194	0	0.00	0.000804	18750
28	0.00435	35.57	-0.0618	3192	0	-3191	0	0.00	0.000882	18862
29	**0.00481**	35.48	-0.0671	3127	3	-3129	0	0.00	0.000959	**18899**

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) =   13221

Cross Section Information:

Axial Load on Section (kips) =   1
Percentage of Main steel in Cross Section =   0.80
Concrete modulus used in Idealization (ksi) = 4283
Cracked Moment of Inertia (ft^4) =   48.635

Idealization of Moment-Curvature Curve by Various Methods:

Method ID	Conc. Strain in/in	Points on Curve: Curv. rad/in	Points on Curve: Moment (K-ft)	Idealized Values: Yield Curv. rad/in	Idealized Values: Moment (K-ft)	symbol for moment	Plastic Curv. rad/in
Strain @	0.003	0.000619	18254	0.000051	18254	Mn	0.000908
Strain @	0.004	0.000816	18768	0.000052	18768	Mn	0.000906
Strain @	0.005	0.000000	0	0.000000	0	Mn	0.000959
CALTRANS	0.00200	0.000401	16968	0.000047	16968	Mp	0.000911
UCSD@5phy	0.00109	0.000184	14059	0.000039	14059	Mn	0.000919

APPENDIX – I

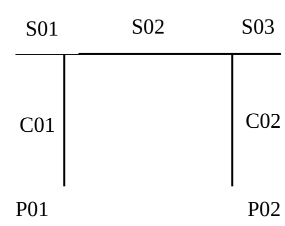
(wFRAME - Input File)

```
wFPREP
VER._1.12,_JAN-14-95
JOB_TITLE
LRFD Design Academy Example No: 1 (Bent 2) DL Only
*********************************************************************
* Columns are pinned at the base. Column longitudinal reinforcement *
* consists of 26, #14 bars. The lateral reinforcement consists of   *
* #8 Hoops at 5" spacing.                                           *
*                                                                   *
*                                         9/18/06                   *
*********************************************************************
All units in kips and feet
******************************************************************
            ***  Analysis Control Block Info ***

The following block of information is for analysis control.
Number of spans and number of link beams are specified.
Direction of push is specified (push to left is not checked yet).
2nd deck out-of-phase push is not checked yet.
******************************************************************
ANALYSIS_CONTROL
    NUMBER_OF_SPANS     3
    NUMBER_OF_LINK_BEAMS   0
    DIRECTION_OF_PUSH       right
    2ND_DECK_OUT_OF_PHASE   no
******************************************************************
            ***  Structural Data Block Info ***

The follwing block of information is for definition of spans, columns
and piles. A span/column/pile code and number (example S01) is specified;
followed by total number of elements in span/col/pile; followed by number
of different types of segments over which all elements are defined. The logic
of this version is such that info for S01, C01, P01, S02, C02 P02, etc...
is expected in the specified order. If a column is connected to a pile cap
and a pile group and the user does not wish to model the pile group, then
the portion of the column below ground (usually 2') must be modeled as a pile
and the tip of the 2' pile should be modeled as fixed in X and Y translation
and fixed, partially released (spring), or completely released for moment
for a column to footing connection of pin nature.51.84
For each segment input the following:
Number of elements per segment;
Fixity code (rn= no release, rs=release start, re=release end);
Length of each element (L);
Depth of element in direction of bending (not used in this version);
Area of cross section;
Modulus of elasticity (Ei);
Softened modulus (Ef, not used in this version);
Cracked moment of inertia(Icr);
Uniform dead load q (negative for superstructure elements, zero otherwise);
Positive plastic moment capacity (Mpp);
Negative plastic moment capacity (Mpn);
Tolerance for elasto-plastic transition (.02 recommended);
Element status = e for elastic, i for inactive.
#   F   L    D      A      Ei     Ef      I     q      Mp    Mn      T  status
*****************************************************************************
STRUCTURAL_DATA
S01   2  2
1  rn   4.72 6.75  62.62 629528  62953  50.80 -68.40  27616  27616    0.02 e
1  rn   3.00 6.75  62.62 629528  62953  50.80 -68.40  27616  27616    0.02 e
C01   4  2
1  rn   3.38 6.00  28.27 629528  62953  47.74     0   27616  27616    0.02 e
3  rn  11.93 6.00  28.27 629528  62953  23.87     0   13808  13808    0.02 e
P01   4  2
3  rn   2.05 6.00  28.27 629528  62953  23.87     0   13808  13808    0.02 e
1  rn   2.05 6.00  28.27 629528  62953  23.87     0   13808  13808    0.02 e
```

Bent cap average effective inertia



APPENDIX – I

(wFRAME - Input File) - Continues

```
S02   6  4
1  rn   3.00 6.75  62.62 629528  62953  50.80 -68.40  27616  27616    0.02 e
2  rn   7.00 6.75  62.62 629528  62953  50.80 -68.40  27616  27616    0.02 e
2  rn   7.00 6.75  62.62 629528  62953  50.80 -68.40  27616  27616    0.02 e
1  rn   3.00 6.75  62.62 629528  62953  50.80 -68.40  27616  27616    0.02 e
C02   4  2
1  rn   3.38 6.00  28.27 629528  62953  47.74      0  27616  27616    0.02 e
3  rn  11.93 6.00  28.27 629528  62953  23.87      0  13808  13808    0.02 e
P02   4  2
3  rn   2.05 6.00  28.27 629528  62953  23.87      0  13808  13808    0.02 e
1  rn   2.05 6.00  28.27 629528  62953  23.87      0  13808  13808    0.02 e
S03   2  2
1  rn   3.00 6.75  62.62 629528  62953  50.80 -68.40  27616  27616    0.02 e
1  rn   4.72 6.75  62.62 629528  62953  50.80 -68.40  27616  27616    0.02 e
*****************************************************************************
                ***  Link Beam or Second Deck Block Info ***

Link beam or second deck option may be placed at any span or any
elevation relative to the superstructure (down is negative).

For each link beam indicate beam number; total number of elements;
nmber of segments; left end elevation; right end elevation.

For each link beam segment input the following:

        see Structural Data Block Info.

Data Specific to this bridge: Link Beams are NOT being used.
#   F   L    D      A      Ei     Ef     I     q      Mp    Mn      T  status
******************************************************************
LINK_BEAM_DATA
******************************************************************

                ***  Soil p-y Block Info ***

This section contains the p-y information. First the number of p-y curves
is specified in the analysis (max 50). Then For each p-y curve enter the
curve number, number of segments (2 for this version with the plateaue
as the third segment generated by computer), p1, y1, p2, y2.
Data Specific to this bridge:

There are two layers of sand.
The top layer is loose sand with layer thickness of 3'.
The bottom layer is medimum dense sand with layer thickness of 5'
Two p-y curves are used per layer.
*****************************************************************
PYS
NUMBER_OF_PYS   4
PY_NO.  NO._OF_SEGMENTS     P1       Y1       P2        Y2
  1     2            2.520     0.142   2.520     1.000
  2     2           14.400     0.154  14.400     1.000
  3     2           15.840     0.104  15.840     1.000
  4     2           47.880     0.108  47.880     1.000
******************************************************************

                ***  Soil t-z Block Info ***

This section contains the t-z information. First the number of t-z curves
is specified in the analysis (max 50). Then For each t-z curve enter the
curve number, number of segments (2 for this version with the plateaue
as the third segment generated by computer), t1, z1, t2, z2.

t-z curves are usually specified for muti-pile situation.
```

Column effective inertia and plastic moment capacity.

Superstructure and bent cap weight uniformly distributed over the entire bent cap

```
Data Specific to this bridge:
*******************************************************************
TZS
NUMBER_OF_TZS   0
TZ_NO.  NO._OF_SEGMENTS     T1      Z1      T2      Z2
*******************************************************************

                ***  Foundation Block Info for p-y application  ***

These p-y values are used to attach horizontal springs to the pile
nodes for lateral response of the pile in the soil-structure interaction
study.

This section contains the foundation information for the p-y applications.
A foundation location is defined as pile locations defined in the structural
input. As discussed earlier the portion of a column below ground is called
a pile.

For each foundation location (i.e. pile or column 1, 2, etc.) indicate:
location number; and the number of p-y applications.

Each soil layer is considered one p-y application in this example.
A soil layer may be subdivided into several segments, each considered
one application. You need to input one new line per each count of
application. Provide as many new lines as the number of
p-y applications with the following info:
Start & end depth of soil layer or sub-layer (measured from top of pile).
Starting p-y number at top of layer. End p-y number at bottom of layer
where linear interpolation is used for the generation of the intermediate
springs.

A factor is also used for the case of many actual piles represented by one
"model pile" in the 2-D modeling of wFRAME. Also the group reduction factors
typically used in soil-structure interaction problems for pile-groups may
be applied through this factor.

Data Specific to this bridge:

*******************************************************************
FOUNDATIONS_PY
   |   NO. OF      |        |       |          |         | FACTOR
LOC| SOIL-LAYERS/ |START   |END    |START-PY  |END-PY  |FOR # OF
NO.| PY APPLIC.   |DEPTH   |DEPTH |  NO.     |   NO.  |  PILE
1       2
                     0.00     3.28    1          2        1
                     3.28     8.20    3          4        1
2       2
                     0.00     3.28    1          2        1
                     3.28     8.20    3          4        1
*******************************************************************

                ***  Foundation Block Info for t-z application  ***

This section contains the foundation information for the t-z applications.
The general logic followed in this section is similar to the p-y
applications. These values are used to attach vertical springs to the pile
nodes for axial response of the pile in the soil-structure interaction
study.

For each foundation location (i.e. column 1, 2, etc.) indicate:
location number, and the number of t-z applications.
Each soil layer may be considered one t-z application or
a soil layer may be subdivided into several segments, each considered
one application. Provide as many new lines as the number of
t-z applications with the following info:
start & end depth of soil (measured from top of pile). Starting t-z number
at top of layer. End t-z number at bottom of layer where linear interpolation
is used for the generation of the intermediate springs.
```

A factor is also used for the case of many actual piles represented by one "model pile" in the 2-D modeling of wFRAME.

Data Specific to this bridge: None

```
*******************************************************************
FOUNDATIONS_TZ
   |   NO. OF     |         |        |            |          | FACTOR
LOC| SOIL-LAYERS/ |START    |END     |START-TZ    |END-TZ    |FOR # OF
NO.| TZ APPLIC.   |DEPTH    |DEPTH   |  NO.       |   NO.    |  PILES
1       0
2       0
*******************************************************************
```

*** Boundary node Block Info for spring application ***

This section contains the boundary information where additional springs may be attached to the extreme boundaries of the structure. The locations are at the pile tips and at the abutments.

The boundary locations are identified according to the structural definition listed earlier in the input file. The following possibilities exist:

For transverse analysis of say a 2 column bent (pin at base of columns) on pile group the following assumptions may be made if the user does not wish to model the piles explicitly. The pile group at each footing location may be modeled as providing fixity or spring action in horizontal direction (the user must estimate the spring value, otherwise fixity must be used). Therefore, boundary locations 0 and 3 are the overhangs and they must be released in all components (rx, ry, rz). The locations 1 and 2 will be modeld at column to footing connection as fx, fy, rz. In general for the transverse analysis of bents with "n" columns, locationns 1 and n+1 indicate the ends of cap beam and it usually is free (rx, ry, rz).

For the transverse analysis of the above bent the user may decide to model the entire pile groups at the two foundations. The piles must be numbered as seen on the elevation view of the bent. This example will be presented later due to the complexity of the situation.

For the longitudinal analysis of a 2 span bridge one may input two ficticious column/pile combinations at the abutments with proper releases to model the roller action of the seat abutment support. In this case release the top of the ficticious column for moment (rs in the element) and model the bottom with fx, fy, rz. This column will not carry a shear in the longitudinal push and it will only carry the dead load at the abutment. Attach a spring at the right abutment to model the passive resistance of the soil (sx plus a new line with k1, del1, k2, del2).

For Location: enter 0 for left end of frame, 1 to xx for tips of piles,
and the last location is for right end of frame.
After boundary location number enter the following info on the next line:
Fixity code for each X, Y and Z directions on consecutive lines:
(rx=release x dir., fx=fix x dir., sx=spring code in x dir. etc.).
If a spring is defined, the next line must be included for the spring
with the following info.:
Number of segments, stiffness and displacements
at breakpoints of the multi-linear curve ((ki,deli) for i=1, 2...)
(Input only 2 segments for this version with the plateaue segment
generated by computer as the third segment).
End bearing at tip of compression piles may be modeled with these springs.

Data Specific to this bridge:

For this simple example only fixity in the Y-direction is provided because the t-z(s) were not explicitly modeled. With t-z modeling the structure will be floating is soil with releases at all boundary locations to represent the real condition.

```
*******************************************************************
BOUNDARIES
LOCATION    FIXITY_CODE   NO._OF_SEGMENTS       ki       del1     k2    del2
    0
               rx
               ry
               rz
    1
               fx
               fy
               rz
    2
               fx
               fy
               rz
    3
               rx
               ry
               rz
*******************************************************************
```

APPENDIX – J
(*wFRAME* Output File)

```
09/18/2006, 12:17
LRFD Design Academy Example No: 1 (Bent 2) DL Only

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


Node Point Information:

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

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


Spring Information at node points:

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

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


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


Element Information:
   element     nodes      depth
 # name fix    i  j      L   d   area    Ei      Ef      Icr      q      Mpp    Mpn  tol
status
  1 S01-01 rn   1   2   4.72 6.8 62.6 629528  62953  50.80 -68.40  27616  27616 0.02 e
  2 S01-02 rn   2   3   3.00 6.8 62.6 629528  62953  50.80 -68.40  27616  27616 0.02 e
  3 C01-01 rn   3   4   3.38 6.0 28.3 629528  62953  47.74   0.00  27616  27616 0.02 e
  4 C01-02 rn   4   5  11.93 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
  5 C01-03 rn   5   6  11.93 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
  6 C01-04 rn   6   7  11.93 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
  7 P01-01 rn   7   8   2.05 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
  8 P01-02 rn   8   9   2.05 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
  9 P01-03 rn   9  10   2.05 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
 10 P01-04 rn  10  11   2.05 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
 11 S02-01 rn   3  12   3.00 6.8 62.6 629528  62953  50.80 -68.40  27616  27616 0.02 e
 12 S02-02 rn  12  13   7.00 6.8 62.6 629528  62953  50.80 -68.40  27616  27616 0.02 e
 13 S02-03 rn  13  14   7.00 6.8 62.6 629528  62953  50.80 -68.40  27616  27616 0.02 e
 14 S02-04 rn  14  15   7.00 6.8 62.6 629528  62953  50.80 -68.40  27616  27616 0.02 e
 15 S02-05 rn  15  16   7.00 6.8 62.6 629528  62953  50.80 -68.40  27616  27616 0.02 e
 16 S02-06 rn  16  17   3.00 6.8 62.6 629528  62953  50.80 -68.40  27616  27616 0.02 e
 17 C02-01 rn  17  18   3.38 6.0 28.3 629528  62953  47.74   0.00  27616  27616 0.02 e
 18 C02-02 rn  18  19  11.93 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
 19 C02-03 rn  19  20  11.93 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
 20 C02-04 rn  20  21  11.93 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
 21 P02-01 rn  21  22   2.05 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
 22 P02-02 rn  22  23   2.05 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
 23 P02-03 rn  23  24   2.05 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
 24 P02-04 rn  24  25   2.05 6.0 28.3 629528  62953  23.87   0.00  13808  13808 0.02 e
 25 S03-01 rn  17  26   3.00 6.8 62.6 629528  62953  50.80 -68.40  27616  27616 0.02 e
 26 S03-02 rn  26  27   4.72 6.8 62.6 629528  62953  50.80 -68.40  27616  27616 0.02 e
bandwidth of the problem = 10
Number of rows and columns in strage = 81 x 30


Cumulative Results of analysis at end of stage   0

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

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00  0.00001  0.00650 -0.00138
    2 S01.01  0.00001 -0.00008 -0.00142
    3 S01.02  0.00001 -0.00450 -0.00155
    4 C01.01 -0.00491 -0.00417 -0.00137
    5 C01.02 -0.01469 -0.00304 -0.00032
    6 C01.03 -0.01398 -0.00191  0.00039
    7 C01.04 -0.00673 -0.00078  0.00077
    8 P01.01 -0.00511 -0.00058  0.00080
    9 P01.02 -0.00344 -0.00039  0.00083
   10 P01.03 -0.00173 -0.00019  0.00084
   11 P01.04  0.00000  0.00000  0.00084
   12 S02.01  0.00001 -0.00950 -0.00174
   13 S02.02  0.00000 -0.02055 -0.00124
```

```
   14 S02.03  0.00000 -0.02509  0.00000
   15 S02.04 -0.00001 -0.02055  0.00124
   16 S02.05 -0.00002 -0.00950  0.00174
   17 S02.06 -0.00002 -0.00450  0.00155
   18 C02.01  0.00490 -0.00417  0.00137
   19 C02.02  0.01468 -0.00304  0.00032
   20 C02.03  0.01397 -0.00191 -0.00039
   21 C02.04  0.00672 -0.00078 -0.00077
   22 P02.01  0.00511 -0.00058 -0.00080
   23 P02.02  0.00344 -0.00039 -0.00083
   24 P02.03  0.00173 -0.00019 -0.00084
   25 P02.04  0.00000  0.00000 -0.00084
   26 S03.01 -0.00002 -0.00008  0.00142
   27 S03.02 -0.00002  0.00650  0.00138

element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1  0.00001  0.00650 -0.00138      0.00      0.00      0.00
                2  0.00001 -0.00008 -0.00142      0.00    322.85   -761.93
  2 S01-02 rn   2  0.00001 -0.00008 -0.00142      0.00   -322.85    761.93
                3  0.00001 -0.00450 -0.00155      0.00    528.05  -2038.27
  3 C01-01 rn   3  0.00450  0.00001 -0.00155   1690.84    -34.91  -1641.29
                4  0.00417 -0.00491 -0.00137  -1690.84     34.91   1523.30
  4 C01-02 rn   4  0.00417 -0.00491 -0.00137   1690.84    -34.91  -1523.30
                5  0.00304 -0.01469 -0.00032  -1690.84     34.91   1106.82
  5 C01-03 rn   5  0.00304 -0.01469 -0.00032   1690.84    -34.91  -1106.81
                6  0.00191 -0.01398  0.00039  -1690.84     34.91    690.33
  6 C01-04 rn   6  0.00191 -0.01398  0.00039   1690.84    -34.91   -690.33
                7  0.00078 -0.00673  0.00077  -1690.84     34.91    273.84
  7 P01-01 rn   7  0.00078 -0.00673  0.00077   1690.84    -34.90   -273.84
                8  0.00058 -0.00511  0.00080  -1690.84     34.90    202.27
  8 P01-02 rn   8  0.00058 -0.00511  0.00080   1690.84    -34.23   -202.28
                9  0.00039 -0.00344  0.00083  -1690.84     34.23    132.10
  9 P01-03 rn   9  0.00039 -0.00344  0.00083   1690.84    -32.80   -132.10
               10  0.00019 -0.00173  0.00084  -1690.84     32.80     64.87
 10 P01-04 rn  10  0.00019 -0.00173  0.00084   1690.84    -31.64    -64.87
               11  0.00000  0.00000  0.00084  -1690.84     31.64      0.00
 11 S02-01 rn   3  0.00001 -0.00450 -0.00155     34.91   1162.81   3679.56
               12  0.00001 -0.00950 -0.00174    -34.91   -957.61   -498.94
 12 S02-02 rn  12  0.00001 -0.00950 -0.00174     34.91    957.60    498.94
               13  0.00000 -0.02055 -0.00124    -34.91   -478.80   4528.49
 13 S02-03 rn  13  0.00000 -0.02055 -0.00124     34.91    478.80  -4528.48
               14  0.00000 -0.02509  0.00000    -34.91      0.00   6204.30
 14 S02-04 rn  14  0.00000 -0.02509  0.00000     34.91      0.00  -6204.30
               15 -0.00001 -0.02055  0.00124    -34.91    478.80   4528.50
 15 S02-05 rn  15 -0.00001 -0.02055  0.00124     34.91   -478.80  -4528.50
               16 -0.00002 -0.00950  0.00174    -34.91    957.60   -498.88
 16 S02-06 rn  16 -0.00002 -0.00950  0.00174     34.91   -957.61    498.88
               17 -0.00002 -0.00450  0.00155    -34.91   1162.81  -3679.47
 17 C02-01 rn  17  0.00450 -0.00002  0.00155   1690.84     34.91   1641.17
               18  0.00417  0.00490  0.00137  -1690.84    -34.91  -1523.19
 18 C02-02 rn  18  0.00417  0.00490  0.00137   1690.84     34.91   1523.18
               19  0.00304  0.01468  0.00032  -1690.84    -34.91  -1106.70
 19 C02-03 rn  19  0.00304  0.01468  0.00032   1690.84     34.91   1106.70
               20  0.00191  0.01397 -0.00039  -1690.84    -34.91   -690.22
 20 C02-04 rn  20  0.00191  0.01397 -0.00039   1690.84     34.91    690.22
               21  0.00078  0.00672 -0.00077  -1690.84    -34.91   -273.73
 21 P02-01 rn  21  0.00078  0.00672 -0.00077   1690.84     34.92    273.75
               22  0.00058  0.00511 -0.00080  -1690.84    -34.92   -202.18
 22 P02-02 rn  22  0.00058  0.00511 -0.00080   1690.84     34.22    202.20
               23  0.00039  0.00344 -0.00083  -1690.84    -34.22   -132.05
 23 P02-03 rn  23  0.00039  0.00344 -0.00083   1690.84     32.78    132.05
               24  0.00019  0.00173 -0.00084  -1690.84    -32.78    -64.85
 24 P02-04 rn  24  0.00019  0.00173 -0.00084   1690.84     31.64     64.85
               25  0.00000  0.00000 -0.00084  -1690.84    -31.64      0.00
 25 S03-01 rn  17 -0.00002 -0.00450  0.00155      0.00    528.05   2038.28
               26 -0.00002 -0.00008  0.00142      0.00   -322.85   -761.94
 26 S03-02 rn  26 -0.00002 -0.00008  0.00142      0.00    322.85    761.93
```

```
                 27 -0.00002  0.00650  0.00138      0.00      0.00      0.00


Cumulative Results of analysis at end of stage   1

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL= 3381.7)  /    (in)
C02-02     1     rs       0.1693          8.4386

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00  0.70321  0.02764 -0.00385
    2 S01.01  0.70321  0.00941 -0.00389
    3 S01.02  0.70320 -0.00242 -0.00402
    4 C01.01  0.68747 -0.00225 -0.00527
    5 C01.02  0.57832 -0.00164 -0.01263
    6 C01.03  0.39568 -0.00103 -0.01760
    7 C01.04  0.16796 -0.00042 -0.02018
    8 P01.01  0.12637 -0.00031 -0.02039
    9 P01.02  0.08442 -0.00021 -0.02052
   10 P01.03  0.04226 -0.00010 -0.02060
   11 P01.04  0.00000  0.00000 -0.02062
   12 S02.01  0.70322 -0.01308 -0.00307
   13 S02.02  0.70323 -0.02652 -0.00078
   14 S02.03  0.70323 -0.02508  0.00105
   15 S02.04  0.70322 -0.01457  0.00169
   16 S02.05  0.70319 -0.00591  0.00040
   17 S02.06  0.70317 -0.00657 -0.00092
   18 C02.01  0.69729 -0.00610 -0.00254
   19 C02.02  0.60768 -0.00445 -0.01198
   20 C02.03  0.42361 -0.00279 -0.01837
   21 C02.04  0.18140 -0.00114 -0.02172
   22 P02.01  0.13658 -0.00085 -0.02199
   23 P02.02  0.09129 -0.00057 -0.02217
   24 P02.03  0.04571 -0.00028 -0.02228
   25 P02.04  0.00000  0.00000 -0.02231
   26 S03.01  0.70318 -0.00956 -0.00105
   27 S03.02  0.70318 -0.01465 -0.00109

element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1  0.70321  0.02764 -0.00385     27.75      0.00     -0.02
                2  0.70321  0.00941 -0.00389    -27.75    322.85   -761.93
  2 S01-02 rn   2  0.70321  0.00941 -0.00389     70.84   -322.82    761.95
                3  0.70320 -0.00242 -0.00402    -70.84    528.02  -2038.24
  3 C01-01 rn   3  0.00242  0.70320 -0.00402    911.27    252.11  11610.99
                4  0.00225  0.68747 -0.00527   -911.27   -252.11 -10760.33
  4 C01-02 rn   4  0.00225  0.68747 -0.00527    911.22    251.57  10761.03
                5  0.00164  0.57832 -0.01263   -911.22   -251.57  -7759.73
  5 C01-03 rn   5  0.00164  0.57832 -0.01263    911.22    251.57   7759.79
                6  0.00103  0.39568 -0.01760   -911.22   -251.57  -4758.53
  6 C01-04 rn   6  0.00103  0.39568 -0.01760    911.22    251.57   4758.53
                7  0.00042  0.16796 -0.02018   -911.22   -251.57  -1757.26
  7 P01-01 rn   7  0.00042  0.16796 -0.02018    911.22    251.46   1757.39
                8  0.00031  0.12637 -0.02039   -911.22   -251.46  -1241.60
  8 P01-02 rn   8  0.00031  0.12637 -0.02039    911.22    234.56   1241.25
                9  0.00021  0.08442 -0.02052   -911.22   -234.56   -760.43
  9 P01-03 rn   9  0.00021  0.08442 -0.02052    911.22    199.51    760.37
               10  0.00010  0.04226 -0.02060   -911.22   -199.51   -351.22
 10 P01-04 rn  10  0.00010  0.04226 -0.02060    911.22    171.37    351.35
               11  0.00000  0.00000 -0.02062   -911.22   -171.37     -0.04
 11 S02-01 rn   3  0.70320 -0.00242 -0.00402   -146.70    383.25  -9572.67
               12  0.70322 -0.01308 -0.00307    146.70   -178.05  10414.60
 12 S02-02 rn  12  0.70322 -0.01308 -0.00307    -88.16    178.03 -10414.58
               13  0.70323 -0.02652 -0.00078     88.16    300.77   9985.00
 13 S02-03 rn  13  0.70323 -0.02652 -0.00078     -7.03   -300.77  -9984.99
               14  0.70323 -0.02508  0.00105      7.03    779.57   6203.80
```

```
14 S02-04 rn  14  0.70323 -0.02508  0.00105     74.06   -779.57   -6203.79
              15  0.70322 -0.01457  0.00169    -74.06   1258.37    -929.02
15 S02-05 rn  15  0.70322 -0.01457  0.00169    155.28  -1258.37     929.02
              16  0.70319 -0.00591  0.00040   -155.28   1737.17 -11413.44
16 S02-06 rn  16  0.70319 -0.00591  0.00040    214.17  -1737.19  11413.43
              17  0.70317 -0.00657 -0.00092   -214.17   1942.39 -16932.76
17 C02-01 rn  17  0.00657  0.70317 -0.00092   2470.40    321.62  14895.23
              18  0.00610  0.69729 -0.00254  -2470.40   -321.62 -13808.39
18 C02-02 rs  18  0.00610  0.69729 -0.00254   2470.46    321.46  13808.00
              19  0.00445  0.60768 -0.01198  -2470.46   -321.46   -9973.00
19 C02-03 rn  19  0.00445  0.60768 -0.01198   2470.46    321.47    9972.98
              20  0.00279  0.42361 -0.01837  -2470.46   -321.47   -6137.84
20 C02-04 rn  20  0.00279  0.42361 -0.01837   2470.46    321.47    6137.81
              21  0.00114  0.18140 -0.02172  -2470.46   -321.47   -2302.70
21 P02-01 rn  21  0.00114  0.18140 -0.02172   2470.46    320.90    2302.26
              22  0.00085  0.13658 -0.02199  -2470.46   -320.90   -1644.77
22 P02-02 rn  22  0.00085  0.13658 -0.02199   2470.46    302.99    1644.68
              23  0.00057  0.09129 -0.02217  -2470.46   -302.99   -1023.28
23 P02-03 rn  23  0.00057  0.09129 -0.02217   2470.46    264.60    1023.32
              24  0.00028  0.04571 -0.02228  -2470.46   -264.60    -480.76
24 P02-04 rn  24  0.00028  0.04571 -0.02228   2470.46    234.55     480.77
              25  0.00000  0.00000 -0.02231  -2470.46   -234.55       0.19
25 S03-01 rn  17  0.70317 -0.00657 -0.00092    -73.42    528.03    2038.19
              26  0.70318 -0.00956 -0.00105     73.42   -322.83    -761.91
26 S03-02 rn  26  0.70318 -0.00956 -0.00105    -27.39    322.85     761.92
              27  0.70318 -0.01465 -0.00109     27.39      0.00      -0.01
```

APPENDIX – K

(Output from xSECTION)

```
09/18/2006, 12:42
************************************************************
*                                                          *
*                      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 BENT 2 COMP - LRFD



Concrete Type Information:

     ----------strains--------  --------strength--------
Type   e0     e2     ecc     eu     f0    f2    fcc    fu    E     W
 1 0.0020 0.0040 0.0055 0.0185  5.20  6.86  7.02  5.56  4280   148
 2 0.0020 0.0040 0.0020 0.0050  5.20  3.58  5.20  2.60  4280   148


Steel Type Information:

     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0150 0.1200 66.00 92.00 29000
 2 0.0023 0.0075 0.0600 66.00 92.00 29000


Steel Fiber Information:

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

7	2	3.85	31.70	2.25
8	2	-3.85	31.70	2.25
9	2	-11.32	29.86	2.25
10	2	-18.14	26.28	2.25
11	2	-23.90	21.17	2.25
12	2	-28.27	14.84	2.25
13	2	-31.00	7.64	2.25
14	2	-31.93	0.00	2.25
15	2	-31.00	-7.64	2.25
16	2	-28.27	-14.84	2.25
17	2	-23.90	-21.17	2.25
18	2	-18.14	-26.28	2.25
19	2	-11.32	-29.86	2.25
20	2	-3.85	-31.70	2.25
21	2	3.85	-31.70	2.25
22	2	11.32	-29.85	2.25
23	2	18.14	-26.28	2.25
24	2	23.90	-21.17	2.25
25	2	28.27	-14.84	2.25
26	2	31.00	-7.64	2.25

Force Equilibrium Condition of the x-section:

step	Max. Conc. Strain epscmax	Neutral Axis in.	Max. Steel Strain Tens.	Conc. Comp.	Steel force Comp.	Steel force Tens.	P/S force	Net force	Curvature rad/in	Moment (K-ft)
0	0.00000	0.00	0.0000	0	0	0	0	0.00	0.000000	0
1	0.00037	-17.80	-0.0001	2261	242	-34	0	-0.80	0.000007	3225
2	0.00041	-14.06	-0.0001	2277	252	-57	0	2.24	0.000008	3586
3	0.00045	-10.70	-0.0002	2296	265	-89	0	1.34	0.000010	3950
4	0.00050	-7.72	-0.0003	2318	277	-127	0	-2.40	0.000011	4323
5	0.00055	-5.12	-0.0004	2355	294	-177	0	1.72	0.000013	4708
6	0.00061	-2.82	-0.0005	2394	311	-235	0	-0.26	0.000016	5116
7	0.00068	-0.72	-0.0006	2446	328	-305	0	-1.53	0.000018	5547
8	0.00075	1.07	-0.0007	2508	349	-388	0	-1.24	0.000021	6011
9	0.00083	2.60	-0.0008	2579	374	-483	0	-0.39	0.000025	6516
10	0.00091	4.05	-0.0010	2665	400	-596	0	-1.67	0.000029	7060
11	0.00101	5.26	-0.0012	2764	428	-721	0	0.61	0.000033	7653
12	0.00112	6.31	-0.0014	2874	459	-862	0	1.42	0.000038	8298
13	0.00123	7.21	-0.0017	2992	493	-1017	0	-1.48	0.000043	8995
14	0.00136	7.97	-0.0019	3129	533	-1192	0	0.77	0.000049	9753
15	0.00151	8.66	-0.0022	3280	580	-1390	0	-0.35	0.000055	10573
16	0.00167	9.42	-0.0026	3410	628	-1567	0	1.52	0.000063	11275
17	0.00184	10.30	-0.0030	3502	677	-1710	0	-1.54	0.000072	11805
18	0.00204	11.18	-0.0035	3570	728	-1826	0	1.68	0.000082	12227
19	0.00225	12.08	-0.0041	3619	780	-1930	0	-1.33	0.000094	12579
20	0.00249	12.88	-0.0048	3663	836	-2028	0	0.56	0.000108	12901
21	0.00275	13.69	-0.0056	3678	892	-2100	0	-0.08	0.000123	13145
22	0.00304	14.40	-0.0065	3710	932	-2171	0	0.72	0.000141	13346
23	0.00336	14.97	-0.0075	3745	960	-2235	0	-0.52	0.000160	13501
24	0.00372	15.49	-0.0086	3781	984	-2295	0	0.15	0.000181	13649
25	0.00411	15.90	-0.0097	3825	1012	-2368	0	-0.80	0.000205	13832
26	0.00454	16.16	-0.0110	3873	1041	-2444	0	-0.61	0.000229	13998
27	0.00502	16.23	-0.0122	3925	1055	-2512	0	-1.85	0.000254	14092
28	0.00555	16.28	-0.0135	3966	1071	-2567	0	-0.88	0.000282	14176
29	0.00614	16.44	-0.0151	4005	1085	-2619	0	0.92	0.000314	14312
30	0.00678	16.58	-0.0169	4048	1101	-2680	0	-0.98	0.000350	14472
31	0.00750	16.69	-0.0188	4095	1118	-2741	0	2.08	0.000389	14644
32	0.00829	16.80	-0.0210	4137	1138	-2804	0	0.98	0.000432	14810
33	0.00917	16.89	-0.0233	4180	1160	-2872	0	-1.99	0.000480	14978
34	0.01013	16.93	-0.0259	4222	1188	-2939	0	1.52	0.000532	15148
35	0.01120	16.94	-0.0286	4274	1198	-3003	0	-0.77	0.000589	15296
36	0.01239	16.93	-0.0316	4325	1211	-3065	0	1.11	0.000650	15442
37	0.01369	16.91	-0.0349	4370	1224	-3124	0	-0.15	0.000718	15567
38	0.01514	16.88	-0.0385	4414	1237	-3182	0	-0.48	0.000792	15678
39	0.01673	16.83	-0.0424	4453	1257	-3240	0	0.27	0.000874	15783

```
40   0.01850   16.77 -0.0467  4486  1279 -3294     0      0.00 0.000963   15865


First Yield of Rebar Information (not Idealized):

     Rebar Number 20
     Coordinates X and Y (global in.)  -3.85, -31.70
     Yield strain =  0.00228
     Curvature (rad/in)=  0.000056
     Moment (ft-k) =   10722


Cross Section Information:

     Axial Load on Section (kips) =  2470
     Percentage of Main steel in Cross Section =   1.44
     Concrete modulus used in Idealization (ksi) =  4280
     Cracked Moment of Inertia (ft^4) =    25.728


Idealization of Moment-Curvature Curve by Various Methods:


                  Points on Curve        Idealized Values
                  ===============   =============================
Method     Conc.                    Yield         symbol  Plastic
 ID      | Strain  Curv.   Moment | Curv.  Moment   for   Curv.
         | in/in   rad/in  (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000138   13318 0.000070   13318   Mn  0.000893
Strain @ 0.004   0.000198   13782 0.000072   13782   Mn  0.000890
Strain @ 0.005   0.000253   14088 0.000074   14088   Mn  0.000889
CALTRANS 0.00879 0.000460   14906 0.000078   14906   Mp  0.000885
UCSD@5phy0.00555 0.000282   14175 0.000074   14175   Mn  0.000888
```

APPENDIX – L

(*wFRAME* Output File)

```
09/18/2006, 13:54
LRFD Design Academy Example No: 1 (Bent 2 w/ OT)

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


Node Point Information:

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

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


Spring Information at node points:

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

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


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


Element Information:
   element     nodes      depth
 # name fix    i  j     L   d   area    Ei      Ef      Icr      q      Mpp    Mpn  tol
status
  1 S01-01 rn   1   2   4.72 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
  2 S01-02 rn   2   3   3.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
  3 C01-01 rn   3   4   3.38 6.0 28.3 629528  62953  51.46   0.00  29812  29812 0.02 e
  4 C01-02 rn   4   5  11.93 6.0 28.3 629528  62953  21.65   0.00  12502  12502 0.02 e
  5 C01-03 rn   5   6  11.93 6.0 28.3 629528  62953  21.65   0.00  12502  12502 0.02 e
  6 C01-04 rn   6   7  11.93 6.0 28.3 629528  62953  21.65   0.00  12502  12502 0.02 e
  7 P01-01 rn   7   8   2.05 6.0 28.3 629528  62953  21.65   0.00  12502  12502 0.02 e
  8 P01-02 rn   8   9   2.05 6.0 28.3 629528  62953  21.65   0.00  12502  12502 0.02 e
  9 P01-03 rn   9  10   2.05 6.0 28.3 629528  62953  21.65   0.00  12502  12502 0.02 e
 10 P01-04 rn  10  11   2.05 6.0 28.3 629528  62953  21.65   0.00  12502  12502 0.02 e
 11 S02-01 rn   3  12   3.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 12 S02-02 rn  12  13   7.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 13 S02-03 rn  13  14   7.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 14 S02-04 rn  14  15   7.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 15 S02-05 rn  15  16   7.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 16 S02-06 rn  16  17   3.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 17 C02-01 rn  17  18   3.38 6.0 28.3 629528  62953  51.46   0.00  29812  29812 0.02 e
 18 C02-02 rn  18  19  11.93 6.0 28.3 629528  62953  25.73   0.00  14906  14906 0.02 e
 19 C02-03 rn  19  20  11.93 6.0 28.3 629528  62953  25.73   0.00  14906  14906 0.02 e
 20 C02-04 rn  20  21  11.93 6.0 28.3 629528  62953  25.73   0.00  14906  14906 0.02 e
 21 P02-01 rn  21  22   2.05 6.0 28.3 629528  62953  25.73   0.00  14906  14906 0.02 e
 22 P02-02 rn  22  23   2.05 6.0 28.3 629528  62953  25.73   0.00  14906  14906 0.02 e
 23 P02-03 rn  23  24   2.05 6.0 28.3 629528  62953  25.73   0.00  14906  14906 0.02 e
 24 P02-04 rn  24  25   2.05 6.0 28.3 629528  62953  25.73   0.00  14906  14906 0.02 e
 25 S03-01 rn  17  26   3.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 26 S03-02 rn  26  27   4.72 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
bandwidth of the problem = 10
Number of rows and columns in strage = 81 x 30


Cumulative Results of analysis at end of stage   0

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

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00 -0.00544  0.00652 -0.00139
    2 S01.01 -0.00544 -0.00006 -0.00142
    3 S01.02 -0.00544 -0.00450 -0.00155
    4 C01.01 -0.01040 -0.00417 -0.00139
    5 C01.02 -0.01974 -0.00304 -0.00024
    6 C01.03 -0.01760 -0.00191  0.00054
    7 C01.04 -0.00830 -0.00078  0.00096
    8 P01.01 -0.00629 -0.00058  0.00099
    9 P01.02 -0.00423 -0.00039  0.00102
   10 P01.03 -0.00212 -0.00019  0.00103
   11 P01.04  0.00000  0.00000  0.00104
   12 S02.01 -0.00544 -0.00951 -0.00174
```

APPENDIX – L

(*wFRAME* Output File) - Continues

```
   13 S02.02 -0.00545 -0.02057 -0.00124
   14 S02.03 -0.00546 -0.02511  0.00000
   15 S02.04 -0.00546 -0.02057  0.00124
   16 S02.05 -0.00547 -0.00950  0.00174
   17 S02.06 -0.00547 -0.00450  0.00155
   18 C02.01 -0.00052 -0.00418  0.00138
   19 C02.02  0.00992 -0.00304  0.00042
   20 C02.03  0.01065 -0.00191 -0.00024
   21 C02.04  0.00530 -0.00078 -0.00060
   22 P02.01  0.00404 -0.00058 -0.00063
   23 P02.02  0.00272 -0.00039 -0.00065
   24 P02.03  0.00137 -0.00019 -0.00066
   25 P02.04  0.00000  0.00000 -0.00067
   26 S03.01 -0.00547 -0.00007  0.00142
   27 S03.02 -0.00547  0.00651  0.00139

element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1 -0.00544  0.00652 -0.00139      0.00      0.00      0.00
                2 -0.00544 -0.00006 -0.00142      0.00    322.85   -761.92
  2 S01-02 rn   2 -0.00544 -0.00006 -0.00142      0.01   -322.85    761.92
                3 -0.00544 -0.00450 -0.00155     -0.01    528.05  -2038.27
  3 C01-01 rn   3  0.00450 -0.00544 -0.00155   1690.67    -34.80  -1633.04
                4  0.00417 -0.01040 -0.00139  -1690.67     34.80   1515.42
  4 C01-02 rn   4  0.00417 -0.01040 -0.00139   1690.67    -34.80  -1515.44
                5  0.00304 -0.01974 -0.00024  -1690.67     34.80   1100.33
  5 C01-03 rn   5  0.00304 -0.01974 -0.00024   1690.67    -34.80  -1100.33
                6  0.00191 -0.01760  0.00054  -1690.67     34.80    685.22
  6 C01-04 rn   6  0.00191 -0.01760  0.00054   1690.67    -34.80   -685.22
                7  0.00078 -0.00830  0.00096  -1690.67     34.80    270.12
  7 P01-01 rn   7  0.00078 -0.00830  0.00096   1690.67    -34.81   -270.13
                8  0.00058 -0.00629  0.00099  -1690.67     34.81    198.74
  8 P01-02 rn   8  0.00058 -0.00629  0.00099   1690.67    -33.97   -198.76
                9  0.00039 -0.00423  0.00102  -1690.67     33.97    129.12
  9 P01-03 rn   9  0.00039 -0.00423  0.00102   1690.67    -32.18   -129.11
               10  0.00019 -0.00212  0.00103  -1690.67     32.18     63.13
 10 P01-04 rn  10  0.00019 -0.00212  0.00103   1690.67    -30.79    -63.12
               11  0.00000  0.00000  0.00104  -1690.67     30.79      0.00
 11 S02-01 rn   3 -0.00544 -0.00450 -0.00155     34.80   1162.63   3671.30
               12 -0.00544 -0.00951 -0.00174    -34.80   -957.43   -491.20
 12 S02-02 rn  12 -0.00544 -0.00951 -0.00174     34.81    957.44    491.20
               13 -0.00545 -0.02057 -0.00124    -34.81   -478.64   4535.06
 13 S02-03 rn  13 -0.00545 -0.02057 -0.00124     34.80    478.64  -4535.05
               14 -0.00546 -0.02511  0.00000    -34.80      0.16   6209.71
 14 S02-04 rn  14 -0.00546 -0.02511  0.00000     34.80     -0.16  -6209.70
               15 -0.00546 -0.02057  0.00124    -34.80    478.96   4532.77
 15 S02-05 rn  15 -0.00546 -0.02057  0.00124     34.80   -478.97  -4532.77
               16 -0.00547 -0.00950  0.00174    -34.80    957.77   -495.78
 16 S02-06 rn  16 -0.00547 -0.00950  0.00174     34.79   -957.78    495.77
               17 -0.00547 -0.00450  0.00155    -34.79   1162.98  -3676.89
 17 C02-01 rn  17  0.00450 -0.00547  0.00155   1691.00     34.80   1638.60
               18  0.00418 -0.00052  0.00138  -1691.00    -34.80  -1520.98
 18 C02-02 rn  18  0.00418 -0.00052  0.00138   1691.00     34.80   1520.97
               19  0.00304  0.00992  0.00042  -1691.00    -34.80  -1105.82
 19 C02-03 rn  19  0.00304  0.00992  0.00042   1691.00     34.80   1105.83
               20  0.00191  0.01065 -0.00024  -1691.00    -34.80   -690.68
 20 C02-04 rn  20  0.00191  0.01065 -0.00024   1691.00     34.80    690.68
               21  0.00078  0.00530 -0.00060  -1691.00    -34.80   -275.53
 21 P02-01 rn  21  0.00078  0.00530 -0.00060   1691.00     34.79    275.51
               22  0.00058  0.00404 -0.00063  -1691.00    -34.79   -204.18
 22 P02-02 rn  22  0.00058  0.00404 -0.00063   1691.00     34.26    204.18
               23  0.00039  0.00272 -0.00065  -1691.00    -34.26   -133.94
 23 P02-03 rn  23  0.00039  0.00272 -0.00065   1691.00     33.13    133.95
               24  0.00019  0.00137 -0.00066  -1691.00    -33.13    -66.04
 24 P02-04 rn  24  0.00019  0.00137 -0.00066   1691.00     32.22     66.05
               25  0.00000  0.00000 -0.00067  -1691.00    -32.22      0.00
 25 S03-01 rn  17 -0.00547 -0.00450  0.00155      0.00    528.05   2038.28
               26 -0.00547 -0.00007  0.00142      0.00   -322.85   -761.93
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```
 26 S03-02 rn  26 -0.00547 -0.00007  0.00142      0.00    322.85    761.93
               27 -0.00547  0.00651  0.00139      0.00      0.00      0.00


Cumulative Results of analysis at end of stage   1

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL= 3381.7)  /    (in)
C02-02     1     rs       0.1763          8.7119

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00  0.72599  0.02573 -0.00359
    2 S01.01  0.72598  0.00872 -0.00363
    3 S01.02  0.72598 -0.00234 -0.00376
    4 C01.01  0.71130 -0.00217 -0.00491
    5 C01.02  0.60242 -0.00158 -0.01291
    6 C01.03  0.41359 -0.00100 -0.01831
    7 C01.04  0.17581 -0.00041 -0.02112
    8 P01.01  0.13228 -0.00030 -0.02134
    9 P01.02  0.08838 -0.00020 -0.02148
   10 P01.03  0.04424 -0.00010 -0.02157
   11 P01.04  0.00000  0.00000 -0.02159
   12 S02.01  0.72599 -0.01225 -0.00283
   13 S02.02  0.72600 -0.02425 -0.00062
   14 S02.03  0.72600 -0.02207  0.00109
   15 S02.04  0.72599 -0.01184  0.00156
   16 S02.05  0.72596 -0.00480  0.00006
   17 S02.06  0.72594 -0.00665 -0.00137
   18 C02.01  0.71852 -0.00618 -0.00299
   19 C02.02  0.62341 -0.00450 -0.01245
   20 C02.03  0.43364 -0.00283 -0.01886
   21 C02.04  0.18555 -0.00115 -0.02223
   22 P02.01  0.13970 -0.00086 -0.02250
   23 P02.02  0.09337 -0.00058 -0.02268
   24 P02.03  0.04676 -0.00029 -0.02279
   25 P02.04  0.00000  0.00000 -0.02282
   26 S03.01  0.72594 -0.01099 -0.00150
   27 S03.02  0.72595 -0.01821 -0.00154

element   node  -------- local -----------  ------ element ----------
#    name  fix     displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1  0.72599  0.02573 -0.00359     28.61      0.00      0.00
                2  0.72598  0.00872 -0.00363    -28.61    322.85   -761.93
  2 S01-02 rn   2  0.72598  0.00872 -0.00363     74.13   -322.83    761.94
                3  0.72598 -0.00234 -0.00376    -74.13    528.03  -2038.23
  3 C01-01 rn   3  0.00234  0.72598 -0.00376    880.79    248.33  11462.52
                4  0.00217  0.71130 -0.00491   -880.79   -248.33 -10622.71
  4 C01-02 rn   4  0.00217  0.71130 -0.00491    880.75    248.70  10622.05
                5  0.00158  0.60242 -0.01291   -880.75   -248.70  -7654.91
  5 C01-03 rn   5  0.00158  0.60242 -0.01291    880.75    248.69   7655.00
                6  0.00100  0.41359 -0.01831   -880.75   -248.69  -4688.09
  6 C01-04 rn   6  0.00100  0.41359 -0.01831    880.75    248.70   4688.10
                7  0.00041  0.17581 -0.02112   -880.75   -248.70  -1721.13
  7 P01-01 rn   7  0.00041  0.17581 -0.02112    880.75    248.73   1720.98
                8  0.00030  0.13228 -0.02134   -880.75   -248.73  -1210.97
  8 P01-02 rn   8  0.00030  0.13228 -0.02134    880.75    231.24   1210.65
                9  0.00020  0.08838 -0.02148   -880.75   -231.24   -736.63
  9 P01-03 rn   9  0.00020  0.08838 -0.02148    880.75    194.17    736.58
               10  0.00010  0.04424 -0.02157   -880.75   -194.17   -338.06
 10 P01-04 rn  10  0.00010  0.04424 -0.02157    880.75    164.76    338.11
               11  0.00000  0.00000 -0.02159   -880.75   -164.76     -0.02
 11 S02-01 rn   3  0.72598 -0.00234 -0.00376   -138.46    352.76  -9424.44
               12  0.72599 -0.01225 -0.00283    138.46   -147.56  10174.91
 12 S02-02 rn  12  0.72599 -0.01225 -0.00283    -78.15    147.54 -10174.90
               13  0.72600 -0.02425 -0.00062     78.15    331.26   9531.87
 13 S02-03 rn  13  0.72600 -0.02425 -0.00062      6.30   -331.26  -9531.86
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                14  0.72600 -0.02207  0.00109     -6.30    810.06   5537.22
 14 S02-04 rn  14  0.72600 -0.02207  0.00109     90.71   -810.06  -5537.21
                15  0.72599 -0.01184  0.00156    -90.71   1288.86  -1809.02
 15 S02-05 rn  15  0.72599 -0.01184  0.00156    175.73  -1288.86   1809.02
                16  0.72596 -0.00480  0.00006   -175.73   1767.66 -12506.87
 16 S02-06 rn  16  0.72596 -0.00480  0.00006    237.22  -1767.68  12506.87
                17  0.72594 -0.00665 -0.00137   -237.22   1972.88 -18117.67
 17 C02-01 rn  17  0.00665  0.72594 -0.00137   2500.74    347.08  16078.33
                18  0.00618  0.71852 -0.00299  -2500.74   -347.08 -14905.02
 18 C02-02 rs  18  0.00618  0.71852 -0.00299   2500.90    346.53  14906.00
                19  0.00450  0.62341 -0.01245  -2500.90   -346.53 -10771.90
 19 C02-03 rn  19  0.00450  0.62341 -0.01245   2500.91    346.53  10771.88
                20  0.00283  0.43364 -0.01886  -2500.91   -346.53  -6637.81
 20 C02-04 rn  20  0.00283  0.43364 -0.01886   2500.91    346.52   6637.80
                21  0.00115  0.18555 -0.02223  -2500.91   -346.52  -2503.74
 21 P02-01 rn  21  0.00115  0.18555 -0.02223   2500.91    346.86   2503.61
                22  0.00086  0.13970 -0.02250  -2500.91   -346.86  -1792.75
 22 P02-02 rn  22  0.00086  0.13970 -0.02250   2500.91    328.04   1793.24
                23  0.00058  0.09337 -0.02268  -2500.91   -328.04  -1120.53
 23 P02-03 rn  23  0.00058  0.09337 -0.02268   2500.91    289.12   1121.02
                24  0.00029  0.04676 -0.02279  -2500.91   -289.12   -528.45
 24 P02-04 rn  24  0.00029  0.04676 -0.02279   2500.91    257.84    528.65
                25  0.00000  0.00000 -0.02282  -2500.91   -257.84     -0.09
 25 S03-01 rn  17  0.72594 -0.00665 -0.00137    -75.08    527.89   2037.76
                26  0.72594 -0.01099 -0.00150     75.08   -322.69   -761.89
 26 S03-02 rn  26  0.72594 -0.01099 -0.00150    -28.61    322.85    761.90
                27  0.72595 -0.01821 -0.00154     28.61      0.00      0.00

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Cumulative Results of analysis at end of stage   5

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/   *g (DL= 3381.7)  /    (in)
C02-02     1     rs       0.1763          8.7119
P02X01     2      2       0.1831          9.5115
P01X01     3      2       0.1857          9.8201
P02X02     4      2       0.1889         10.2091
C01-02     5     rs       0.1896         10.2905

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
    1 S01.00  0.85755  0.03150 -0.00432
    2 S01.01  0.85754  0.01106 -0.00436
    3 S01.02  0.85754 -0.00218 -0.00449
    4 C01.01  0.84005 -0.00203 -0.00584
    5 C01.02  0.71115 -0.00148 -0.01526
    6 C01.03  0.48814 -0.00093 -0.02162
    7 C01.04  0.20749 -0.00038 -0.02492
    8 P01.01  0.15612 -0.00028 -0.02518
    9 P01.02  0.10430 -0.00019 -0.02536
   10 P01.03  0.05222 -0.00009 -0.02545
   11 P01.04  0.00000  0.00000 -0.02548
   12 S02.01  0.85755 -0.01400 -0.00338
   13 S02.02  0.85757 -0.02851 -0.00080
   14 S02.03  0.85757 -0.02665  0.00117
   15 S02.04  0.85756 -0.01518  0.00182
   16 S02.05  0.85753 -0.00600  0.00040
   17 S02.06  0.85751 -0.00681 -0.00103
   18 C02.01  0.85128 -0.00632 -0.00264
   19 C02.02  0.71998 -0.00461 -0.01548
```

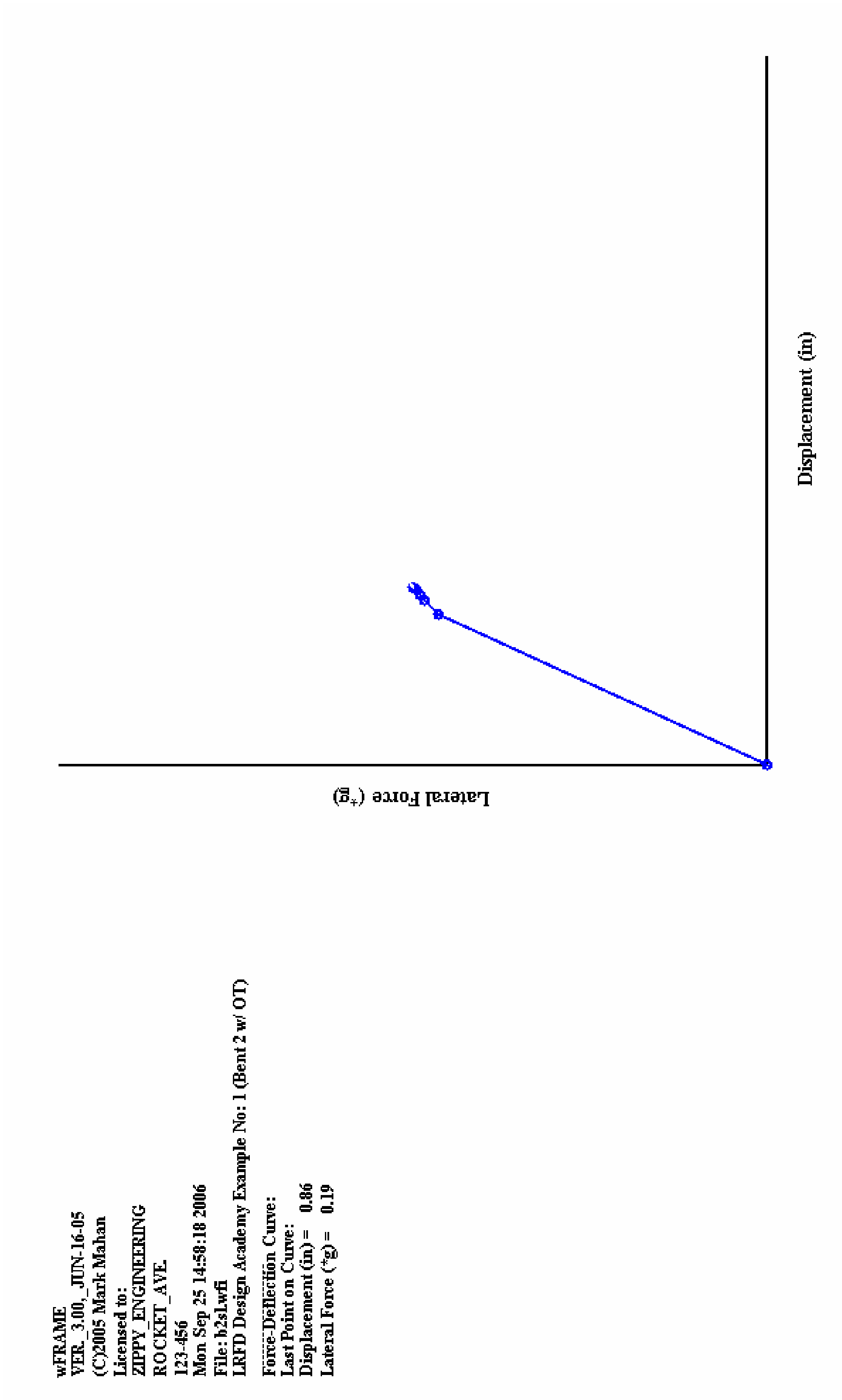
APPENDIX – L

(*wFRAME* Output File) - Continues

20	C02.03	0.49412	-0.00289	-0.02188
21	C02.04	0.21011	-0.00118	-0.02523
22	P02.01	0.15810	-0.00088	-0.02549
23	P02.02	0.10564	-0.00059	-0.02568
24	P02.03	0.05289	-0.00029	-0.02578
25	P02.04	0.00000	0.00000	-0.02581
26	S03.01	0.85752	-0.01011	-0.00115
27	S03.02	0.85752	-0.01568	-0.00119

element #	name	fix	node	local displ.x	local displ.y	local rotation	element axial	element shear	element moment
1	S01-01	rn	1	0.85755	0.03150	-0.00432	30.69	0.00	0.00
			2	0.85754	0.01106	-0.00436	-30.69	322.85	-761.93
2	S01-02	rn	2	0.85754	0.01106	-0.00436	79.61	-322.82	761.94
			3	0.85754	-0.00218	-0.00449	-79.61	528.02	-2038.22
3	C01-01	rn	3	0.00218	0.85754	-0.00449	821.03	292.28	13490.93
			4	0.00203	0.84005	-0.00584	-821.03	-292.28	-12502.53
4	C01-02	rs	4	0.00203	0.84005	-0.00584	821.00	292.61	12502.00
			5	0.00148	0.71115	-0.01526	-821.00	-292.61	-9011.08
5	C01-03	rn	5	0.00148	0.71115	-0.01526	821.00	292.60	9011.18
			6	0.00093	0.48814	-0.02162	-821.00	-292.60	-5520.51
6	C01-04	rn	6	0.00093	0.48814	-0.02162	821.00	292.60	5520.52
			7	0.00038	0.20749	-0.02492	-821.00	-292.60	-2029.78
7	P01-01	rn	7	0.00038	0.20749	-0.02492	821.00	292.70	2029.61
			8	0.00028	0.15612	-0.02518	-821.00	-292.70	-1429.53
8	P01-02	rn	8	0.00028	0.15612	-0.02518	821.00	272.95	1429.19
			9	0.00019	0.10430	-0.02536	-821.00	-272.95	-869.60
9	P01-03	rn	9	0.00019	0.10430	-0.02536	821.00	229.23	869.56
			10	0.00009	0.05222	-0.02545	-821.00	-229.23	-399.12
10	P01-04	rn	10	0.00009	0.05222	-0.02545	821.00	194.52	399.16
			11	0.00000	0.00000	-0.02548	-821.00	-194.52	-0.03
11	S02-01	rn	3	0.85754	-0.00218	-0.00449	-174.16	293.01	-11452.84
			12	0.85755	-0.01400	-0.00338	174.16	-87.81	12024.06
12	S02-02	rn	12	0.85755	-0.01400	-0.00338	-109.39	87.79	-12024.05
			13	0.85757	-0.02851	-0.00080	109.39	391.01	10962.80
13	S02-03	rn	13	0.85757	-0.02851	-0.00080	-18.56	-391.01	-10962.79
			14	0.85757	-0.02665	0.00117	18.56	869.81	6549.94
14	S02-04	rn	14	0.85757	-0.02665	0.00117	72.24	-869.81	-6549.93
			15	0.85756	-0.01518	0.00182	-72.24	1348.61	-1214.53
15	S02-05	rn	15	0.85756	-0.01518	0.00182	163.66	-1348.61	1214.52
			16	0.85753	-0.00600	0.00040	-163.66	1827.41	-12330.59
16	S02-06	rn	16	0.85753	-0.00600	0.00040	229.75	-1827.42	12330.59
			17	0.85751	-0.00681	-0.00103	-229.75	2032.62	-18120.62
17	C02-01	rn	17	0.00681	0.85751	-0.00103	2560.48	347.95	16081.14
			18	0.00632	0.85128	-0.00264	-2560.48	-347.95	-14905.03
18	C02-02	rs	18	0.00632	0.85128	-0.00602	2560.64	347.34	14906.00
			19	0.00461	0.71998	-0.01548	-2560.64	-347.34	-10762.20
19	C02-03	rn	19	0.00461	0.71998	-0.01548	2560.64	347.34	10762.18
			20	0.00289	0.49412	-0.02188	-2560.64	-347.34	-6618.41
20	C02-04	rn	20	0.00289	0.49412	-0.02188	2560.64	347.34	6618.40
			21	0.00118	0.21011	-0.02523	-2560.64	-347.34	-2474.65
21	P02-01	rn	21	0.00118	0.21011	-0.02523	2560.64	**347.71**	2474.55
			22	0.00088	0.15810	-0.02549	-2560.64	-347.71	-1761.95
22	P02-02	rn	22	0.00088	0.15810	-0.02549	2560.64	327.66	1762.47
			23	0.00059	0.10564	-0.02568	-2560.64	-327.66	-1090.57
23	P02-03	rn	23	0.00059	0.10564	-0.02568	2560.64	283.86	1091.11
			24	0.00029	0.05289	-0.02578	-2560.64	-283.86	-509.31
24	P02-04	rn	24	0.00029	0.05289	-0.02578	2560.64	248.49	509.51
			25	0.00000	0.00000	-0.02581	-2560.64	-248.49	-0.09
25	S03-01	rn	17	0.85751	-0.00681	-0.00103	-80.77	527.89	2037.76
			26	0.85752	-0.01011	-0.00115	80.77	-322.69	-761.90
26	S03-02	rn	26	0.85752	-0.01011	-0.00115	-30.72	322.85	761.91
			27	0.85752	-0.01568	-0.00119	30.72	0.00	0.00

APPENDIX – M
(Force – Displacement Relationship)



APPENDIX – N

(Cap Beam – Seismic Moment and Shear Demands due to Overstrength)

```
09/18/2006, 16:47
LRFD Design Academy Example No: 1 (Bent 2 w/ OT and Overstrength)

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


Node Point Information:

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

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


Spring Information at node points:

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

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


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


Element Information:
   element     nodes      depth
 # name fix    i  j      L   d   area    Ei      Ef      Icr     q      Mpp    Mpn  tol
status
  1 S01-01 rn   1   2   4.72 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
  2 S01-02 rn   2   3   3.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
  3 C01-01 rn   3   4   3.38 6.0 28.3 629528  62953  51.46   0.00  29812  29812 0.02 e
  4 C01-02 rn   4   5  11.93 6.0 28.3 629528  62953  21.65   0.00  15002  15002 0.02 e
  5 C01-03 rn   5   6  11.93 6.0 28.3 629528  62953  21.65   0.00  15002  15002 0.02 e
  6 C01-04 rn   6   7  11.93 6.0 28.3 629528  62953  21.65   0.00  15002  15002 0.02 e
  7 P01-01 rn   7   8   2.05 6.0 28.3 629528  62953  21.65   0.00  15002  15002 0.02 e
  8 P01-02 rn   8   9   2.05 6.0 28.3 629528  62953  21.65   0.00  15002  15002 0.02 e
  9 P01-03 rn   9  10   2.05 6.0 28.3 629528  62953  21.65   0.00  15002  15002 0.02 e
 10 P01-04 rn  10  11   2.05 6.0 28.3 629528  62953  21.65   0.00  15002  15002 0.02 e
 11 S02-01 rn   3  12   3.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 12 S02-02 rn  12  13   7.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 13 S02-03 rn  13  14   7.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 14 S02-04 rn  14  15   7.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 15 S02-05 rn  15  16   7.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 16 S02-06 rn  16  17   3.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 17 C02-01 rn  17  18   3.38 6.0 28.3 629528  62953  51.46   0.00  29812  29812 0.02 e
 18 C02-02 rn  18  19  11.93 6.0 28.3 629528  62953  25.73   0.00  17887  17887 0.02 e
 19 C02-03 rn  19  20  11.93 6.0 28.3 629528  62953  25.73   0.00  17887  17887 0.02 e
 20 C02-04 rn  20  21  11.93 6.0 28.3 629528  62953  25.73   0.00  17887  17887 0.02 e
 21 P02-01 rn  21  22   2.05 6.0 28.3 629528  62953  25.73   0.00  17887  17887 0.02 e
 22 P02-02 rn  22  23   2.05 6.0 28.3 629528  62953  25.73   0.00  17887  17887 0.02 e
 23 P02-03 rn  23  24   2.05 6.0 28.3 629528  62953  25.73   0.00  17887  17887 0.02 e
 24 P02-04 rn  24  25   2.05 6.0 28.3 629528  62953  25.73   0.00  17887  17887 0.02 e
 25 S03-01 rn  17  26   3.00 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
 26 S03-02 rn  26  27   4.72 6.8 62.6 629528  62953  50.80 -68.40  29812  29812 0.02 e
bandwidth of the problem = 10
Number of rows and columns in strage = 81 x 30

--------------------------------------------------------------------------
--------------------------------------------------------------------------
--------------------------------------------------------------------------
--------------------------------------------------------------------------


Cumulative Results of analysis at end of stage   6

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL= 3381.7)  /    (in)
P02X01     1      2       0.1884          9.3139
P01X01     2      2       0.1975          9.7707
P02X02     3      2       0.1988          9.8361
P01X02     4      2       0.2077         10.2856
C02-02     5     rs       0.2153         10.6735
C01-02     6     rs       0.2267         12.0897

node# name   ---------- GLOBAL ---------
            Displ.x  Displ.y  Rotation
    1 S01.00  1.00747  0.03515 -0.00474
    2 S01.01  1.00747  0.01276 -0.00477
```

3	S01.02	1.00746	-0.00172	-0.00490
4	C01.01	0.98812	-0.00160	-0.00653
5	C01.02	0.83919	-0.00116	-0.01783
6	C01.03	0.57718	-0.00073	-0.02548
7	C01.04	0.24565	-0.00030	-0.02949
8	P01.01	0.18486	-0.00022	-0.02981
9	P01.02	0.12352	-0.00015	-0.03002
10	P01.03	0.06184	-0.00007	-0.03014
11	P01.04	0.00000	0.00000	-0.03018
12	S02.01	1.00748	-0.01442	-0.00357
13	S02.02	1.00750	-0.02887	-0.00065
14	S02.03	1.00750	-0.02548	0.00141
15	S02.04	1.00749	-0.01288	0.00186
16	S02.05	1.00745	-0.00483	-0.00001
17	S02.06	1.00743	-0.00727	-0.00171
18	C02.01	0.99832	-0.00675	-0.00365
19	C02.02	0.84737	-0.00492	-0.01802
20	C02.03	0.58279	-0.00309	-0.02572
21	C02.04	0.24813	-0.00126	-0.02977
22	P02.01	0.18674	-0.00094	-0.03010
23	P02.02	0.12479	-0.00063	-0.03032
24	P02.03	0.06248	-0.00031	-0.03045
25	P02.04	0.00000	0.00000	-0.03049
26	S03.01	1.00744	-0.01263	-0.00184
27	S03.02	1.00744	-0.02145	-0.00188

element			node	-------- local -----------			------ element ----------		
#	name	fix		displ.x	displ.y	rotation	axial	shear	moment
1	S01-01	rn	1	1.00747	0.03515	-0.00474	36.77	0.00	0.00
			2	1.00747	0.01276	-0.00477	-36.77	322.85	-761.93
2	S01-02	rn	2	1.00747	0.01276	-0.00477	95.15	-322.82	761.94
			3	1.00746	-0.00172	-0.00490	-95.15	528.02	-2038.22
3	C01-01	rn	3	0.00172	1.00746	-0.00490	647.37	349.55	16184.04
			4	0.00160	0.98812	-0.00653	-647.37	-349.55	-15002.42
4	C01-02	rs	4	0.00160	0.98812	-0.00653	647.34	349.75	15002.00
			5	0.00116	0.83919	-0.01783	-647.34	-349.75	-10829.43
5	C01-03	rn	5	0.00116	0.83919	-0.01783	647.34	349.73	10829.55
			6	0.00073	0.57718	-0.02548	-647.34	-349.73	-6657.23
6	C01-04	rn	6	0.00073	0.57718	-0.02548	647.34	349.74	6657.25
			7	0.00030	0.24565	-0.02949	-647.34	-349.74	-2484.87
7	P01-01	rn	7	0.00030	0.24565	-0.02949	647.34	349.87	2484.73
			8	0.00022	0.18486	-0.02981	-647.34	-349.87	-1767.39
8	P01-02	rn	8	0.00022	0.18486	-0.02981	647.34	330.23	1767.04
			9	0.00015	0.12352	-0.03002	-647.34	-330.23	-1090.06
9	P01-03	rn	9	0.00015	0.12352	-0.03002	647.34	286.19	1090.03
			10	0.00007	0.06184	-0.03014	-647.34	-286.19	-502.80
10	P01-04	rn	10	0.00007	0.06184	-0.03014	647.34	245.06	502.83
			11	0.00000	0.00000	-0.03018	-647.34	-245.06	-0.04
11	S02-01	rn	3	1.00746	-0.00172	-0.00490	-208.17	119.35	-14145.99
			12	1.00748	-0.01442	-0.00357	208.17	85.85	14196.24
12	S02-02	rn	12	1.00748	-0.01442	-0.00357	-130.59	-85.87	**-14196.23**
			13	1.00750	-0.02887	-0.00065	130.59	564.67	11919.36
13	S02-03	rn	13	1.00750	-0.02887	-0.00065	-21.82	-564.67	-11919.35
			14	1.00750	-0.02548	0.00141	21.82	1043.47	6290.87
14	S02-04	rn	14	1.00750	-0.02548	0.00141	86.68	-1043.47	-6290.87
			15	1.00749	-0.01288	0.00186	-86.68	1522.27	-2689.21
15	S02-05	rn	15	1.00749	-0.01288	0.00186	196.04	-1522.27	2689.20
			16	1.00745	-0.00483	-0.00001	-196.04	2001.07	-15020.88
16	S02-06	rn	16	1.00745	-0.00483	-0.00001	274.91	-2001.08	**15020.89**
			17	1.00743	-0.00727	-0.00171	-274.91	2206.28	-21331.90
17	C02-01	rn	17	0.00727	1.00743	-0.00171	2734.14	415.96	19292.38
			18	0.00675	0.99832	-0.00365	-2734.14	-415.96	-17886.16
18	C02-02	rs	18	0.00675	0.99832	-0.00667	2734.33	415.39	17887.00
			19	0.00492	0.84737	-0.01802	-2734.33	-415.39	-12931.32
19	C02-03	rn	19	0.00492	0.84737	-0.01802	2734.34	415.39	12931.28
			20	0.00309	0.58279	-0.02572	-2734.34	-415.39	-7975.62
20	C02-04	rn	20	0.00309	0.58279	-0.02572	2734.34	415.39	7975.62
			21	0.00126	0.24813	-0.02977	-2734.34	-415.39	-3019.96

APPENDIX – N

(Cap Beam – Seismic Moment and Shear Demands due to Overstrength) - Continues

21	P02-01	rn	21	0.00126	0.24813	-0.02977	2734.34	415.87	3019.91
			22	0.00094	0.18674	-0.03010	-2734.34	-415.87	-2167.56
22	P02-02	rn	22	0.00094	0.18674	-0.03010	2734.34	395.79	2168.21
			23	0.00063	0.12479	-0.03032	-2734.34	-395.79	-1356.63
23	P02-03	rn	23	0.00063	0.12479	-0.03032	2734.34	351.95	1357.20
			24	0.00031	0.06248	-0.03045	-2734.34	-351.95	-635.81
24	P02-04	rn	24	0.00031	0.06248	-0.03045	2734.34	310.21	636.02
			25	0.00000	0.00000	-0.03049	-2734.34	-310.21	-0.09
25	S03-01	rn	17	1.00743	-0.00727	-0.00171	-96.75	527.88	2037.74
			26	1.00744	-0.01263	-0.00184	96.75	-322.68	-761.89
26	S03-02	rn	26	1.00744	-0.01263	-0.00184	-36.81	322.85	761.90
			27	1.00744	-0.02145	-0.00188	36.81	0.00	0.00

APPENDIX – O

(Joint Movement Calculations)

STATE OF CALIFORNIA.DEPARTMENT OF TRANSPORTATION

JOINT MOVEMENTS CALCULATIONS [a]

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

Note: Specific instructions are included as footnotes.

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

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

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

ITEM(1) DESIGNER	DATE	ITEM(2)CHECKED BY	DATE
DESIGNER		CHECKER	5/2/2006

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

$$\text{Anticipated Shortening} = \frac{0.63}{100} \times \left(\frac{202 + 210}{2} \right) = 1.30\ in$$

APPENDIX – P

(wFRAME Longitudinal Push Over – Input file)

```
wFPREP
VER._1.12,_JAN-14-95
JOB_TITLE
Design Academy Example No: LRFD Superstructure (Right Push)
*********************************************************************
* Columns are pinned at the base. Column longitudinal reinforcement *
* consists of 26, #14 bars. The lateral reinforcement consists of   *
* #8 Hoops at 5" spacing. The superstructure properties used are as *
* calculated in CTBridge.                                           *
*                                                                   *
* Used to assess abutment contribution.                              *
*                                                     09/26/06      *
*********************************************************************
All units in kips and feet
******************************************************************
            ***  Analysis Control Block Info ***

The following block of information is for analysis control.
Number of spans and number of link beams are specified.
Direction of push is specified (push to left is not checked yet).
2nd deck out-of-phase push is not checked yet.
******************************************************************
ANALYSIS_CONTROL
    NUMBER_OF_SPANS     5
    NUMBER_OF_LINK_BEAMS   0
    DIRECTION_OF_PUSH       right
    2ND_DECK_OUT_OF_PHASE   no
******************************************************************
            ***  Structural Data Block Info ***

The follwing block of information is for definition of spans, columns
and piles.  A span/column/pile code and number (example S01) is specified;
followed by total number of elements in span/col/pile; followed by number
of different types of segments over which all elements are defined. The logic
of this version is such that info for S01, C01, P01, S02, C02 P02, etc...
is expected in the specified order. If a column is connected to a pile cap
and a pile group and the user does not wish to model the pile group, then
the portion of the column below ground (usually 2') must be modeled as a pile
and the tip of the 2' pile should be modeled as fixed in X and Y translation
and fixed, partially released (spring), or completely released for moment
for a column to footing connection of pin nature.51.84

For each segment input the following:
Number of elements per segment;
Fixity code (rn= no release, rs=release start, re=release end);
Length of each element (L);
Depth of element in direction of bending (not used in this version);
Area of cross section;
Modulus of elasticity (Ei);
Softened modulus (Ef, not used in this version);
Cracked moment of inertia(Icr);
Uniform dead load q (negative for superstructure elements, zero otherwise);
Positive plastic moment capacity (Mpp);
Negative plastic moment capacity (Mpn);
Tolerance for elasto-plastic transition (.02 recommended);
Element status = e for elastic, i for inactive.

#   F   L    D      A      Ei     Ef     I     q      Mp    Mn      T  status
*****************************************************************************
STRUCTURAL_DATA
S01 1 1
 1  rn  2.0  6.75 103.49 629528  62953 826.75  -0.01   99999  99999    0.02 e
C01   1  1
 1  rs  1.0  6.00  56.55 629528  62953  95.48     0    99999  99999    0.02 e
P01   1  1
 1  rn  1.0  6.00  56.55 629528  62953  47.74     0    27616  27616    0.02 e
S02   12  4
 9  rn 12.60 6.75 103.49 629528  62953 731.10 -18.25   99999  99999    0.02 e
```

```
 1  rn  4.17 6.75 109.55 629528  62953 778.93 -19.12   99999  99999    0.02 e
 1  rn  4.17 6.75 109.55 629528  62953 778.93 -19.59   99999  99999    0.02 e
 1  rn  4.26 6.75 115.60 629528  62953 826.75 -70.07   99999  99999    0.02 e
C02   4  2
 1  rn  3.38 6.00  56.55 629528  62953  95.48     0    99999  99999    0.02 e
 3  rn 11.93 6.00  56.55 629528  62953  47.74     0    27616  27616    0.02 e
P02   4  2
 3  rn  2.05 6.00  56.55 629528  62953  47.74     0    27616  27616    0.02 e
 1  rn  2.05 6.00  56.55 629528  62953  47.74     0    27616  27616    0.02 e
S03  14  5
 1  rn  4.26 6.75 115.60 629528  62953 826.75 -70.07   99999  99999    0.02 e
 2  rn  6.27 6.75 109.55 629528  62953 778.93 -18.64   99999  99999    0.02 e
 8  rn 16.80 6.75 103.49 629528  62953 731.10 -18.25   99999  99999    0.02 e
 2  rn  6.27 6.75 109.55 629528  62953 778.93 -18.64   99999  99999    0.02 e
 1  rn  4.26 6.75 115.60 629528  62953 826.75 -70.07   99999  99999    0.02 e
C03   4  2
 1  rn  3.38 6.00  56.55 629528  62953  95.48     0    99999  99999    0.02 e
 3  rn 11.95 6.00  56.55 629528  62953  47.54     0    27494  27494    0.02 e
P03   5  2
 4  rn  2.23 6.00  56.55 629528  62953  47.54     0    27494  27494    0.02 e
 1  rn  2.23 6.00  56.55 629528  62953  47.54     0    27494  27494    0.02 e
S04  12   4
 1  rn  4.26 6.75 115.60 629528  62953 826.75 -70.07   99999  99999    0.02 e
 1  rn  3.77 6.75 109.55 629528  62953 778.93 -19.64   99999  99999    0.02 e
 1  rn  3.77 6.75 109.55 629528  62953 778.93 -19.21   99999  99999    0.02 e
 9  rn 11.80 6.75 103.49 629528  62953 731.10 -18.25   99999  99999    0.02 e
C04   1  1
 1  rs  1.0  6.00  56.55 629528  62953  95.48     0    99999  99999    0.02 e
P04   1  1
 1  rn  1.0  6.00  56.55 629528  62953  47.54     0    27494  27494    0.02 e
S05 1 1
 1  rn  2.0  6.75 103.49 629528  62953 826.75  -0.01   99999  99999    0.02 e
******************************************************************************

                ***  Link Beam or Second Deck Block Info ***

Link beam or second deck option may be placed at any span or any
elevation relative to the superstructure (down is negative).

For each link beam indicate beam number; total number of elements;
nmber of segments; left end elevation; right end elevation.

For each link beam segment input the following:

        see Structural Data Block Info.

Data Specific to this bridge: Link Beams are NOT being used.
#   F   L    D      A      Ei     Ef     I      q      Mp    Mn      T  status
******************************************************************
LINK_BEAM_DATA
******************************************************************

                ***  Soil p-y Block Info ***

This section contains the p-y information. First the number of p-y curves
is specified in the analysis (max 50). Then For each p-y curve enter the
curve number, number of segments (2 for this version with the plateaue
as the third segment generated by computer), p1, y1, p2, y2.
Data Specific to this bridge:

There are two layers of sand.
The top layer is loose sand with layer thickness of 3'.
The bottom layer is medimum dense sand with layer thickness of 5'
Two p-y curves are used per layer.
*****************************************************************
PYS
NUMBER_OF_PYS   8
PY_NO.  NO._OF_SEGMENTS     P1       Y1       P2        Y2
```

```
  1         2            5.040     0.142   5.040     1.000
  2         2           28.800     0.154  28.800     1.000
  3         2           31.680     0.104  31.680     1.000
  4         2           95.360     0.108  95.360     1.000
  5         2            5.040     0.138   5.040     1.000
  6         2           28.800     0.154  28.800     1.000
  7         2           30.960     0.110  30.960     1.000
  8         2          126.240     0.108 126.240     1.000
******************************************************************

                ***  Soil t-z Block Info ***

This section contains the t-z information. First the number of t-z curves
is specified in the analysis (max 50). Then For each t-z curve enter the
curve number, number of segments (2 for this version with the plateaue
as the third segment generated by computer), t1, z1, t2, z2.

t-z curves are usually specified for muti-pile situation.

Data Specific to this bridge:
Curve 1 is applicable at 6" below Ground Level
Curve 2 is applicable at 2'-6" below Ground Level.
Curve 3 is applicable at 3'-6" below Ground Level.
Curve 4 is applicable at 7'-6" below Ground Level.
******************************************************************
TZS
NUMBER_OF_TZS   0
TZ_NO.  NO._OF_SEGMENTS     T1      Z1      T2      Z2
******************************************************************

                ***  Foundation Block Info for p-y application  ***

These p-y values are used to attach horizontal springs to the pile
nodes for lateral response of the pile in the soil-structure interaction
study.

This section contains the foundation information for the p-y applications.
A foundation location is defined as pile locations defined in the structural
input. As discussed earlier the portion of a column below ground is called
a pile.

For each foundation location (i.e. pile or column 1, 2, etc.) indicate:
location number; and the number of p-y applications.

Each soil layer is considered one p-y application in this example.
A soil layer may be subdivided into several segments, each considered
one application. You need to input one new line per each count of
application. Provide as many new lines as the number of
p-y applications with the following info:
Start & end depth of soil layer or sub-layer (measured from top of pile).
Starting p-y number at top of layer. End p-y number at bottom of layer
where linear interpolation is used for the generation of the intermediate
springs.

A factor is also used for the case of many actual piles represented by one
"model pile" in the 2-D modeling of wFRAME. Also the group reduction factors
typically used in soil-structure interaction problems for pile-groups may
be applied through this factor.

Data Specific to this bridge:

******************************************************************
FOUNDATIONS_PY
   |   NO. OF     |         |        |           |         | FACTOR
LOC| SOIL-LAYERS/ |START    |END     |START-PY   |END-PY   |FOR # OF
NO.| PY APPLIC.   |DEPTH    |DEPTH |  NO.      |   NO.  |  PILE
1       0
2        2
```

```
                     0.00     3.28    1          2        1
                     3.28     8.20    3          4        1
3       2
                     0.00    3.28     5          6        1
                     3.28   11.15     7          8        1
4      0

******************************************************************

                ***  Foundation Block Info for t-z application  ***

This section contains the foundation information for the t-z applications.
The general logic followed in this section is similar to the p-y
applications. These values are used to attach vertical springs to the pile
nodes for axial response of the pile in the soil-structure interaction
study.

For each foundation location (i.e. column 1, 2, etc.) indicate:
location number, and the number of t-z applications.
Each soil layer may be considered one t-z application or
a soil layer may be subdivided into several segments, each considered
one application. Provide as many new lines as the number of
t-z applications with the following info:
start & end depth of soil (measured from top of pile). Starting t-z number
at top of layer. End t-z number at bottom of layer where linear interpolation
is used for the generation of the intermediate springs.
A factor is also used for the case of many actual piles represented by one
"model pile" in the 2-D modeling of wFRAME.

Data Specific to this bridge: None

******************************************************************
FOUNDATIONS_TZ
   |   NO. OF     |         |       |           |         | FACTOR
LOC| SOIL-LAYERS/ |START    |END    |START-TZ  |END-TZ  |FOR # OF
NO.| TZ APPLIC.   |DEPTH    |DEPTH |  NO.      |   NO.  |  PILES
1       0
2       0
3      0
4      0
******************************************************************

        ***  Boundary node Block Info for spring application  ***

This section contains the boundary information where additional springs
may be attached to the extreme boundaries of the structure. The locations
are at the pile tips and at the abutments.

The boundary locations are identified according to the structural definition
listed earlier in the input file. The following possibilities exist:

For transverse analysis of say a 2 column bent (pin at base of columns) on
pile group the following assumptions may be made if the user does not wish
to model the piles explicitly. The pile group at each footing location may
be modeled as providing fixity or spring action in horizontal direction (the
user must estimate the spring value, otherwise fixity must be used).
Therefore, boundary locations 0 and 3 are the overhangs and they must be
released in all components (rx, ry, rz). The locations 1 and 2 will be modeld
at column to footing connection as fx, fy, rz. In general for the transverse
analysis of bents with "n" columns, locationns 1 and n+1 indicate the ends
of cap beam and it usually is free (rx, ry, rz).


For the transverse analysis of the above bent the user may decide to model
the entire pile groups at the two foundations. The piles must be numbered as
seen on the elevation view of the bent. This example will be presented later
due to the complexity of the situation.
```

APPENDIX – P

(wFRAME Longitudinal Push Over – Input file) - Continues

For the longitudinal analysis of a 2 span bridge one may input two ficticious column/pile combinations at the abutments with proper releases to model the roller action of the seat abutment support. In this case release the top of the ficticious column for moment (rs in the element) and model the bottom with fx, fy, rz. This column will not carry a shear in the longitudinal push and it will only carry the dead load at the abutment. Attach a spring at the right abutment to model the passive resistance of the soil (sx plus a new line with k1, del1, k2, del2).

For Location: enter 0 for left end of frame, 1 to xx for tips of piles, and the last location is for right end of frame.
After boundary location number enter the following info on the next line:
Fixity code for each X, Y and Z directions on consecutive lines:
(rx=release x dir., fx=fix x dir., sx=spring code in x dir. etc.).
If a spring is defined, the next line must be included for the spring with the following info.:
Number of segments, stiffness and displacements at breakpoints of the multi-linear curve ((ki,deli) for i=1, 2...)
(Input only 2 segments for this version with the plateaue segment generated by computer as the third segment).
End bearing at tip of compression piles may be modeled with these springs.

Data Specific to this bridge:

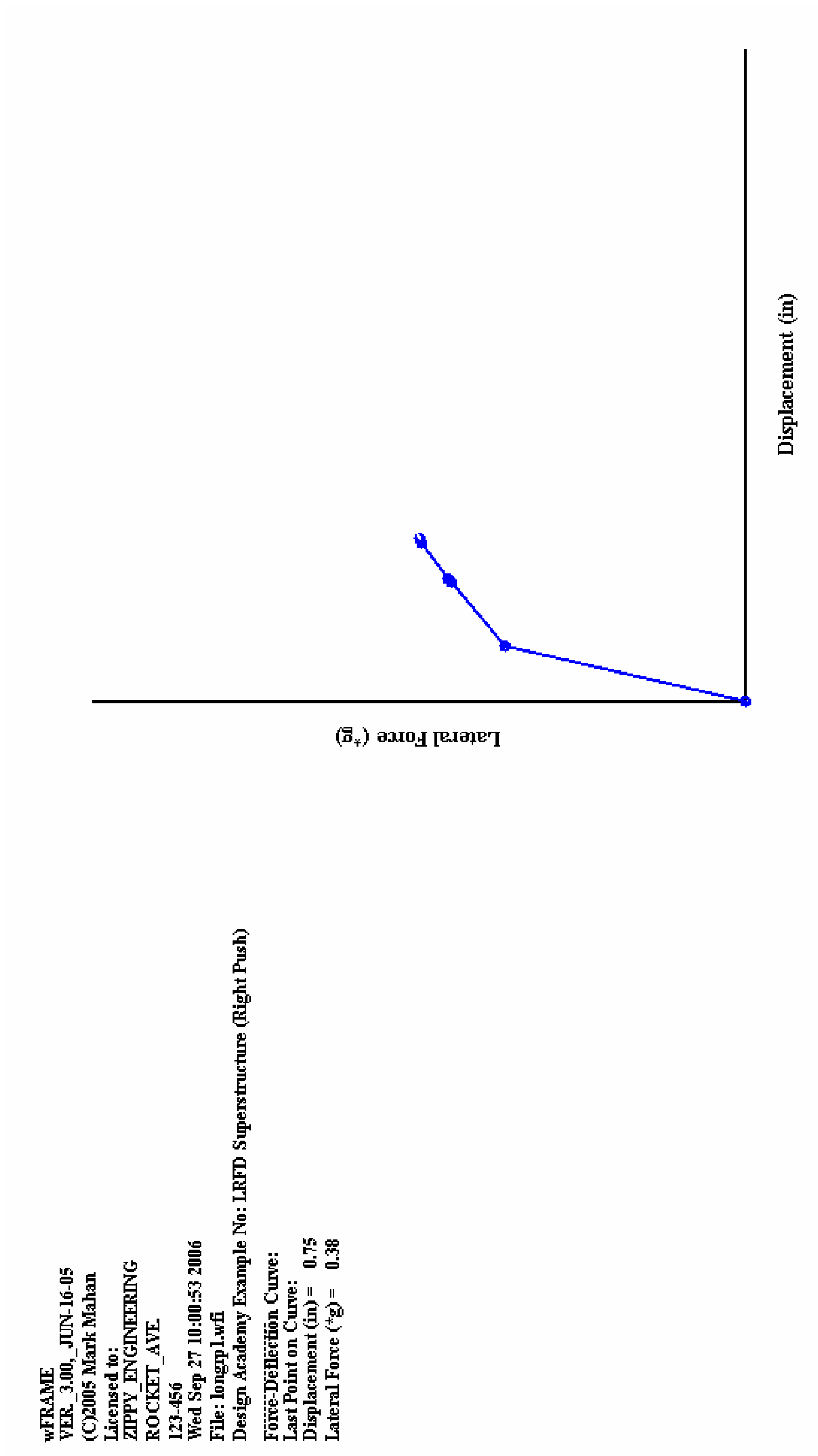
For this simple example only fixity in the Y-direction is provided because the t-z(s) were not explicitly modeled. With t-z modeling the structure will be floating is soil with releases at all boundary locations to represent the real condition.

```
******************************************************************
BOUNDARIES
LOCATION    FIXITY_CODE   NO._OF_SEGMENTS       ki       del1     k2    del2
    0
                rx
                ry
                rz
    1
                fx
                fy
                rz
    2
                fx
                fy
                rz
    3
                fx
                fy
                rz
    4
                fx
                fy
                rz
    5
                sx
                   2   7716  0.249  0.00  1
                ry
                rz
******************************************************************
```

Initial Abutment Stiffness
Units: kip,ft.

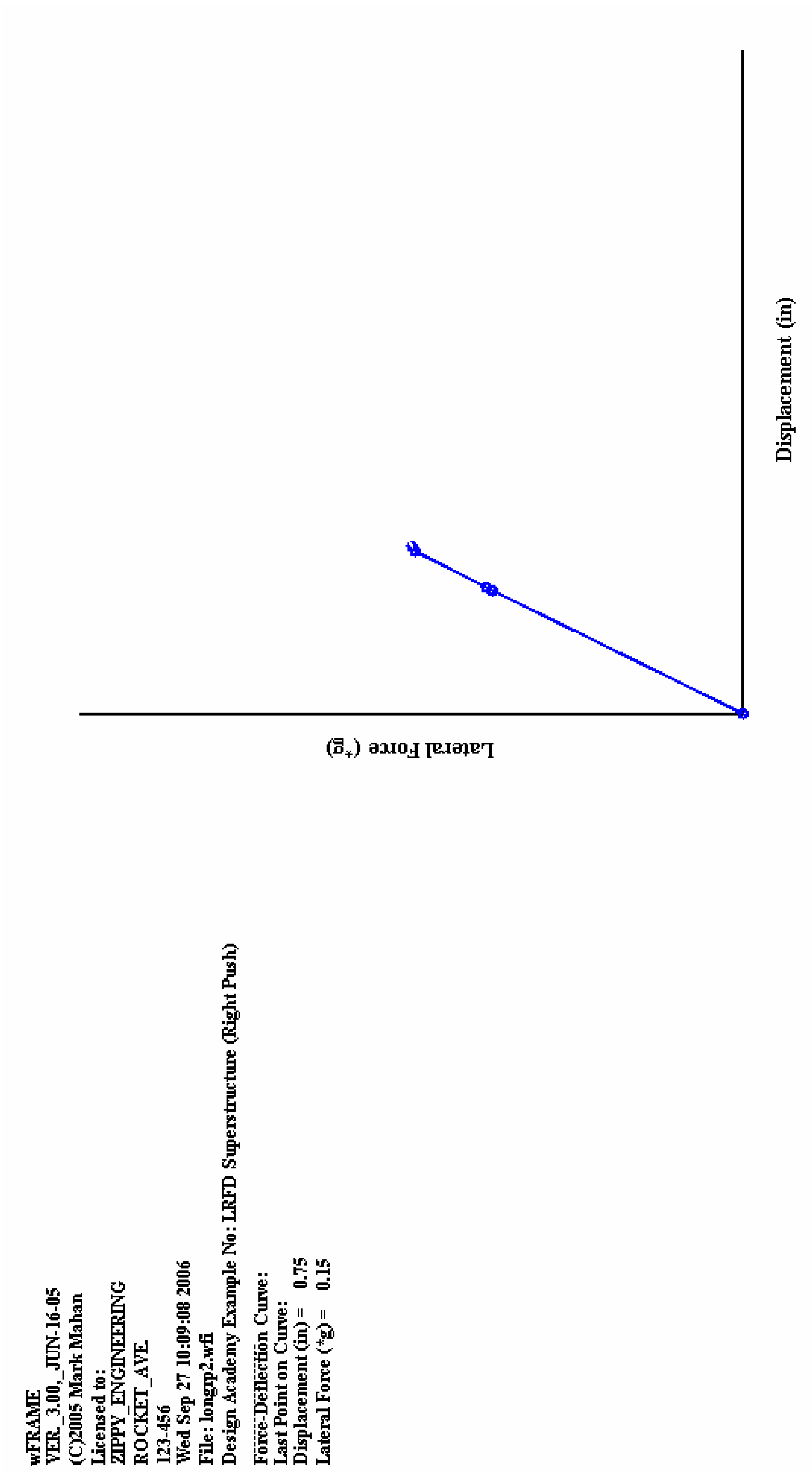
APPENDIX – Q1

(wFRAME Longitudinal Push Over – Force/Displacement Relationship)



APPENDIX – Q2

(wFRAME Longitudinal Push Over – Force/Displacement Relationship)



APPENDIX – R

(*wFRAME* Input File – To Determine Superstructure Force due to Column Hinging, Case 1)

```
wFPREP
VER._1.12,_JAN-14-95
JOB_TITLE
Design Academy Example No: LRFD Superstructure (Right Push)
*********************************************************************
* Columns are pinned at the base. Column longitudinal reinforcement *
* consists of 26, #14 bars. The lateral reinforcement consists of   *
* #8 Hoops at 5" spacing. The superstructure properties used are as *
* calculated in CTBridge.                                           *
*                                                                   *
* Used to obtain Case 1 EQ Demand Moments.                          *
*                                                    09/26/06       *
*********************************************************************
All units in kips and feet
******************************************************************
            ***  Analysis Control Block Info ***

The following block of information is for analysis control.
Number of spans and number of link beams are specified.
Direction of push is specified (push to left is not checked yet).
2nd deck out-of-phase push is not checked yet.
******************************************************************
ANALYSIS_CONTROL
    NUMBER_OF_SPANS     5
    NUMBER_OF_LINK_BEAMS   0
    DIRECTION_OF_PUSH       right
    2ND_DECK_OUT_OF_PHASE   no
******************************************************************
            ***  Structural Data Block Info ***

The follwing block of information is for definition of spans, columns
and piles.  A span/column/pile code and number (example S01) is specified;
followed by total number of elements in span/col/pile; followed by number
of different types of segments over which all elements are defined. The logic
of this version is such that info for S01, C01, P01, S02, C02 P02, etc...
is expected in the specified order. If a column is connected to a pile cap
and a pile group and the user does not wish to model the pile group, then
the portion of the column below ground (usually 2') must be modeled as a pile
and the tip of the 2' pile should be modeled as fixed in X and Y translation
and fixed, partially released (spring), or completely released for moment
for a column to footing connection of pin nature.51.84

For each segment input the following:
Number of elements per segment;
Fixity code (rn= no release, rs=release start, re=release end);
Length of each element (L);
Depth of element in direction of bending (not used in this version);
Area of cross section;
Modulus of elasticity (Ei);
Softened modulus (Ef, not used in this version);
Cracked moment of inertia(Icr);
Uniform dead load q (negative for superstructure elements, zero otherwise);
Positive plastic moment capacity (Mpp);
Negative plastic moment capacity (Mpn);
Tolerance for elasto-plastic transition (.02 recommended);
Element status = e for elastic, i for inactive.

#   F   L    D      A      Ei     Ef     I     q      Mp    Mn      T  status
******************************************************************************
STRUCTURAL_DATA
S01 1 1
 1  rn  2.0  6.75 103.49 629528  62953 826.75  -0.01   99999  99999    0.02 e
C01   1  1
 1  rs  1.0  6.00  56.55 629528  62953  95.48     0    99999  99999    0.02 e
P01   1  1
 1  rn  1.0  6.00  56.55 629528  62953  47.74     0    99999  99999    0.02 e
S02   12  4
 9  rn 12.60 6.75 103.49 629528  62953 731.10  -0.01   99999  99999    0.02 e
```

```
 1  rn  4.17 6.75 109.55 629528  62953 778.93  -0.01   99999  99999    0.02 e
 1  rn  4.17 6.75 109.55 629528  62953 778.93  -0.01   99999  99999    0.02 e
 1  rn  4.26 6.75 115.60 629528  62953 826.75  -0.01   99999  99999    0.02 e
C02   4  2
 1  rn  3.38 6.00  56.55 629528  62953  95.48     0    99999  99999    0.02 e
 3  rn 11.93 6.00  56.55 629528  62953  47.74     0    31988  34494    0.02 e
P02   4  2
 3  rn  2.05 6.00  56.55 629528  62953  47.74     0    31988  34494    0.02 e
 1  rn  2.05 6.00  56.55 629528  62953  47.74     0    31988  34494    0.02 e
S03  14  5
 1  rn  4.26 6.75 115.60 629528  62953 826.75  -0.01   99999  99999    0.02 e
 2  rn  6.27 6.75 109.55 629528  62953 778.93  -0.01   99999  99999    0.02 e
 8  rn 16.80 6.75 103.49 629528  62953 731.10  -0.01   99999  99999    0.02 e
 2  rn  6.27 6.75 109.55 629528  62953 778.93  -0.01   99999  99999    0.02 e
 1  rn  4.26 6.75 115.60 629528  62953 826.75  -0.01   99999  99999    0.02 e
C03   4  2
 1  rn  3.38 6.00  56.55 629528  62953  95.48     0    99999  99999    0.02 e
 3  rn 11.95 6.00  56.55 629528  62953  47.54     0    34440  31764    0.02 e
P03   5  2
 4  rn  2.23 6.00  56.55 629528  62953  47.54     0    34440  31764    0.02 e
 1  rn  2.23 6.00  56.55 629528  62953  47.54     0    34440  31764    0.02 e
S04  12   4
 1  rn  4.26 6.75 115.60 629528  62953 826.75  -0.01   99999  99999    0.02 e
 1  rn  3.77 6.75 109.55 629528  62953 778.93  -0.01   99999  99999    0.02 e
 1  rn  3.77 6.75 109.55 629528  62953 778.93  -0.01   99999  99999    0.02 e
 9  rn 11.80 6.75 103.49 629528  62953 731.10  -0.01   99999  99999    0.02 e
C04   1  1
 1  rs  1.0  6.00  56.55 629528  62953  95.48     0    99999  99999    0.02 e
P04   1  1
 1  rn  1.0  6.00  56.55 629528  62953  47.54     0    99999  99999    0.02 e
S05 1 1
 1  rn  2.0  6.75 103.49 629528  62953 826.75  -0.01   99999  99999    0.02 e
******************************************************************************

                ***  Link Beam or Second Deck Block Info ***

Link beam or second deck option may be placed at any span or any
elevation relative to the superstructure (down is negative).

For each link beam indicate beam number; total number of elements;
nmber of segments; left end elevation; right end elevation.

For each link beam segment input the following:

        see Structural Data Block Info.

Data Specific to this bridge: Link Beams are NOT being used.
#   F   L    D      A      Ei     Ef     I      q      Mp    Mn      T  status
******************************************************************
LINK_BEAM_DATA
******************************************************************

                ***  Soil p-y Block Info ***

This section contains the p-y information. First the number of p-y curves
is specified in the analysis (max 50). Then For each p-y curve enter the
curve number, number of segments (2 for this version with the plateaue
as the third segment generated by computer), p1, y1, p2, y2.
Data Specific to this bridge:

There are two layers of sand.
The top layer is loose sand with layer thickness of 3'.
The bottom layer is medimum dense sand with layer thickness of 5'
Two p-y curves are used per layer.
*****************************************************************
PYS
NUMBER_OF_PYS   8
PY_NO.  NO._OF_SEGMENTS     P1       Y1       P2        Y2
```

```
  1         2           5.040     0.142   5.040     1.000
  2         2          28.800     0.154  28.800     1.000
  3         2          31.680     0.104  31.680     1.000
  4         2          95.360     0.108  95.360     1.000
  5         2           5.040     0.138   5.040     1.000
  6         2          28.800     0.154  28.800     1.000
  7         2          30.960     0.110  30.960     1.000
  8         2         126.240     0.108 126.240     1.000
******************************************************************

                ***  Soil t-z Block Info ***

This section contains the t-z information. First the number of t-z curves
is specified in the analysis (max 50). Then For each t-z curve enter the
curve number, number of segments (2 for this version with the plateaue
as the third segment generated by computer), t1, z1, t2, z2.

t-z curves are usually specified for muti-pile situation.

Data Specific to this bridge:
Curve 1 is applicable at 6" below Ground Level
Curve 2 is applicable at 2'-6" below Ground Level.
Curve 3 is applicable at 3'-6" below Ground Level.
Curve 4 is applicable at 7'-6" below Ground Level.
******************************************************************
TZS
NUMBER_OF_TZS   0
TZ_NO.  NO._OF_SEGMENTS     T1      Z1      T2      Z2
******************************************************************

                ***  Foundation Block Info for p-y application  ***

These p-y values are used to attach horizontal springs to the pile
nodes for lateral response of the pile in the soil-structure interaction
study.

This section contains the foundation information for the p-y applications.
A foundation location is defined as pile locations defined in the structural
input. As discussed earlier the portion of a column below ground is called
a pile.

For each foundation location (i.e. pile or column 1, 2, etc.) indicate:
location number; and the number of p-y applications.

Each soil layer is considered one p-y application in this example.
A soil layer may be subdivided into several segments, each considered
one application. You need to input one new line per each count of
application. Provide as many new lines as the number of
p-y applications with the following info:
Start & end depth of soil layer or sub-layer (measured from top of pile).
Starting p-y number at top of layer. End p-y number at bottom of layer
where linear interpolation is used for the generation of the intermediate
springs.

A factor is also used for the case of many actual piles represented by one
"model pile" in the 2-D modeling of wFRAME. Also the group reduction factors
typically used in soil-structure interaction problems for pile-groups may
be applied through this factor.

Data Specific to this bridge:

******************************************************************
FOUNDATIONS_PY
   |   NO. OF     |         |        |           |         | FACTOR
LOC| SOIL-LAYERS/ |START    |END    |START-PY  |END-PY  |FOR # OF
NO.| PY APPLIC.   |DEPTH    |DEPTH |  NO.      |   NO.  |  PILE
1       0
2        2
```

```
                     0.00     3.28    1          2        1
                     3.28     8.20    3          4        1
3       2
                     0.00    3.28     5          6        1
                     3.28   11.15     7          8        1
4       0

******************************************************************

                ***  Foundation Block Info for t-z application  ***

This section contains the foundation information for the t-z applications.
The general logic followed in this section is similar to the p-y
applications. These values are used to attach vertical springs to the pile
nodes for axial response of the pile in the soil-structure interaction
study.

For each foundation location (i.e. column 1, 2, etc.) indicate:
location number, and the number of t-z applications.
Each soil layer may be considered one t-z application or
a soil layer may be subdivided into several segments, each considered
one application. Provide as many new lines as the number of
t-z applications with the following info:
start & end depth of soil (measured from top of pile). Starting t-z number
at top of layer. End t-z number at bottom of layer where linear interpolation
is used for the generation of the intermediate springs.
A factor is also used for the case of many actual piles represented by one
"model pile" in the 2-D modeling of wFRAME.

Data Specific to this bridge: None

******************************************************************
FOUNDATIONS_TZ
   |   NO. OF     |         |       |          |         | FACTOR
LOC| SOIL-LAYERS/ |START    |END    |START-TZ  |END-TZ   |FOR # OF
NO.| TZ APPLIC.   |DEPTH    |DEPTH |  NO.     |   NO.   |  PILES
1       0
2       0
3       0
4       0
******************************************************************

        ***  Boundary node Block Info for spring application  ***

This section contains the boundary information where additional springs
may be attached to the extreme boundaries of the structure. The locations
are at the pile tips and at the abutments.

The boundary locations are identified according to the structural definition
listed earlier in the input file. The following possibilities exist:

For transverse analysis of say a 2 column bent (pin at base of columns) on
pile group the following assumptions may be made if the user does not wish
to model the piles explicitly. The pile group at each footing location may
be modeled as providing fixity or spring action in horizontal direction (the
user must estimate the spring value, otherwise fixity must be used).
Therefore, boundary locations 0 and 3 are the overhangs and they must be
released in all components (rx, ry, rz). The locations 1 and 2 will be modeld
at column to footing connection as fx, fy, rz. In general for the transverse
analysis of bents with "n" columns, locationns 1 and n+1 indicate the ends
of cap beam and it usually is free (rx, ry, rz).


For the transverse analysis of the above bent the user may decide to model
the entire pile groups at the two foundations. The piles must be numbered as
seen on the elevation view of the bent. This example will be presented later
due to the complexity of the situation.
```

APPENDIX – R

(*wFRAME* Input File – To Determine Superstructure Force due to Column Hinging Continues, Case 1)

```
For the longitudinal analysis of a 2 span bridge one may input two ficticious
column/pile combinations at the abutments with proper releases to model the
roller action of the seat abutment support. In this case release the top of
the ficticious column for moment (rs in the element) and model the bottom
with fx, fy, rz. This column will not carry a shear in the longitudinal push
and it will only carry the dead load at the abutment. Attach a spring at the
right abutment to model the passive resistance of the soil (sx plus a new
line with k1, del1, k2, del2).

For Location: enter 0 for left end of frame, 1 to xx for tips of piles,
and the last location is for right end of frame.
After boundary location number enter the following info on the next line:
Fixity code for each X, Y and Z directions on consecutive lines:
(rx=release x dir., fx=fix x dir., sx=spring code in x dir. etc.).
If a spring is defined, the next line must be included for the spring
with the following info.:
Number of segments, stiffness and displacements
at breakpoints of the multi-linear curve ((ki,deli) for i=1, 2...)
(Input only 2 segments for this version with the plateaue segment
generated by computer as the third segment).
End bearing at tip of compression piles may be modeled with these springs.

Data Specific to this bridge:

For this simple example only fixity in the Y-direction is provided because
the t-z(s) were not explicitly modeled. With t-z modeling the structure will
be floating is soil with releases at all boundary locations to represent the
real condition.

******************************************************************
BOUNDARIES
LOCATION    FIXITY_CODE   NO._OF_SEGMENTS       ki       del1     k2    del2
    0
            rx
            ry
            rz
    1
            fx
            fy
            rz
    2
            fx
            fy
            rz
    3
            fx
            fy
            rz
    4
            fx
            fy
            rz
    5
            rx
            ry
            rz
******************************************************************
```

APPENDIX – S

(*wFRAME* Output File – To Determine Superstructure Forces due to Column Hinging, Case 1)

```
09/27/2006, 00:57
Design Academy Example No: LRFD Superstructure (Right Push)

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


Node Point Information:

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

node    name    coordinates    -----------fixity --------
  #              X       Y     X-dir.     Y-dir.   Rotation
    1 S01.00     0.00     0.00   r          r          r
    2 S01.01     2.00     0.00   r          r          r
    3 C01.01     2.00    -1.00   r          r          r
    4 P01.01     2.00    -2.00   f 0.0000   f 0.0000   r
    5 S02.01    14.60     0.00   r          r          r
    6 S02.02    27.20     0.00   r          r          r
    7 S02.03    39.80     0.00   r          r          r
    8 S02.04    52.40     0.00   r          r          r
    9 S02.05    65.00     0.00   r          r          r
   10 S02.06    77.60     0.00   r          r          r
   11 S02.07    90.20     0.00   r          r          r
   12 S02.08   102.80     0.00   r          r          r
   13 S02.09   115.40     0.00   r          r          r
   14 S02.10   119.57     0.00   r          r          r
   15 S02.11   123.74     0.00   r          r          r
   16 S02.12   128.00     0.00   r          r          r
   17 C02.01   128.00    -3.38   r          r          r
   18 C02.02   128.00   -15.31   r          r          r
   19 C02.03   128.00   -27.24   r          r          r
   20 C02.04   128.00   -39.17   r          r          r
   21 P02.01   128.00   -41.22   s 2.7e+002  r          r
   22 P02.02   128.00   -43.27   s 8.3e+002  r          r
   23 P02.03   128.00   -45.32   s 1.3e+003  r          r
   24 P02.04   128.00   -47.37   f 0.0000   f 0.0000   r
   25 S03.01   132.26     0.00   r          r          r
   26 S03.02   138.53     0.00   r          r          r
   27 S03.03   144.80     0.00   r          r          r
   28 S03.04   161.60     0.00   r          r          r
   29 S03.05   178.40     0.00   r          r          r
   30 S03.06   195.20     0.00   r          r          r
   31 S03.07   212.00     0.00   r          r          r
   32 S03.08   228.80     0.00   r          r          r
   33 S03.09   245.60     0.00   r          r          r
```

```
   34 S03.10   262.40     0.00   r          r          r
   35 S03.11   279.20     0.00   r          r          r
   36 S03.12   285.47     0.00   r          r          r
   37 S03.13   291.74     0.00   r          r          r
   38 S03.14   296.00     0.00   r          r          r
   39 C03.01   296.00    -3.38   r          r          r
   40 C03.02   296.00   -15.33   r          r          r
   41 C03.03   296.00   -27.28   r          r          r
   42 C03.04   296.00   -39.23   r          r          r
   43 P03.01   296.00   -41.46   s 3.2e+002  r          r
   44 P03.02   296.00   -43.69   s 9.2e+002  r          r
   45 P03.03   296.00   -45.92   s 1.5e+003  r          r
   46 P03.04   296.00   -48.15   s 2e+003   r          r
   47 P03.05   296.00   -50.38   f 0.0000   f 0.0000   r
   48 S04.01   300.26     0.00   r          r          r
   49 S04.02   304.03     0.00   r          r          r
   50 S04.03   307.80     0.00   r          r          r
   51 S04.04   319.60     0.00   r          r          r
   52 S04.05   331.40     0.00   r          r          r
   53 S04.06   343.20     0.00   r          r          r
   54 S04.07   355.00     0.00   r          r          r
   55 S04.08   366.80     0.00   r          r          r
   56 S04.09   378.60     0.00   r          r          r
   57 S04.10   390.40     0.00   r          r          r
   58 S04.11   402.20     0.00   r          r          r
   59 S04.12   414.00     0.00   r          r          r
   60 C04.01   414.00    -1.00   r          r          r
   61 P04.01   414.00    -2.00   f 0.0000   f 0.0000   r
   62 S05.01   416.00     0.00   r          r          r


Spring Information at node points:

k's = k/ft or ft-k/rad.;  d's = ft or rad.
node   spring      k1      d1      k2      d2
  #    name
   21  P02X01       272.74  0.149         0.00  1.000         0.00 1000.000
   22  P02X02       828.36  0.105         0.00  1.000         0.00 1000.000
   23  P02X03      1326.91  0.106         0.00  1.000         0.00 1000.000
   43  P03X01       317.46  0.149         0.00  1.000         0.00 1000.000
   44  P03X02       919.77  0.110         0.00  1.000         0.00 1000.000
   45  P03X03      1476.21  0.109         0.00  1.000         0.00 1000.000
   46  P03X04      2038.47  0.109         0.00  1.000         0.00 1000.000


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


Element Information:
   element      nodes       depth
 # name fix    i  j      L   d   area    Ei      Ef      Icr      q      Mpp    Mpn  tol
status
  1 S01-01 rn   1   2   2.00 6.8 103.5 629528  62953 826.75  -0.01  99999  99999 0.02 e
  2 C01-01 rs   2   3   1.00 6.0 56.5 629528  62953  95.48  0.00  99999  99999 0.02 e
  3 P01-01 rn   3   4   1.00 6.0 56.5 629528  62953  47.74  0.00  99999  99999 0.02 e
  4 S02-01 rn   2   5  12.60 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
  5 S02-02 rn   5   6  12.60 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
  6 S02-03 rn   6   7  12.60 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
  7 S02-04 rn   7   8  12.60 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
  8 S02-05 rn   8   9  12.60 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
  9 S02-06 rn   9  10  12.60 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 10 S02-07 rn  10  11  12.60 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 11 S02-08 rn  11  12  12.60 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 12 S02-09 rn  12  13  12.60 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 13 S02-10 rn  13  14   4.17 6.8 109.6 629528  62953 778.93  -0.01  99999  99999 0.02 e
 14 S02-11 rn  14  15   4.17 6.8 109.6 629528  62953 778.93  -0.01  99999  99999 0.02 e
 15 S02-12 rn  15  16   4.26 6.8 115.6 629528  62953 826.75  -0.01  99999  99999 0.02 e
```

```
 16 C02-01 rn  16  17   3.38 6.0 56.5 629528  62953  95.48   0.00  99999  99999 0.02 e
 17 C02-02 rn  17  18  11.93 6.0 56.5 629528  62953  47.74   0.00  31988  34494 0.02 e
 18 C02-03 rn  18  19  11.93 6.0 56.5 629528  62953  47.74   0.00  31988  34494 0.02 e
 19 C02-04 rn  19  20  11.93 6.0 56.5 629528  62953  47.74   0.00  31988  34494 0.02 e
 20 P02-01 rn  20  21   2.05 6.0 56.5 629528  62953  47.74   0.00  31988  34494 0.02 e
 21 P02-02 rn  21  22   2.05 6.0 56.5 629528  62953  47.74   0.00  31988  34494 0.02 e
 22 P02-03 rn  22  23   2.05 6.0 56.5 629528  62953  47.74   0.00  31988  34494 0.02 e
 23 P02-04 rn  23  24   2.05 6.0 56.5 629528  62953  47.74   0.00  31988  34494 0.02 e
 24 S03-01 rn  16  25   4.26 6.8 115.6 629528  62953 826.75  -0.01  99999  99999 0.02 e
 25 S03-02 rn  25  26   6.27 6.8 109.6 629528  62953 778.93  -0.01  99999  99999 0.02 e
 26 S03-03 rn  26  27   6.27 6.8 109.6 629528  62953 778.93  -0.01  99999  99999 0.02 e
 27 S03-04 rn  27  28  16.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 28 S03-05 rn  28  29  16.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 29 S03-06 rn  29  30  16.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 30 S03-07 rn  30  31  16.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 31 S03-08 rn  31  32  16.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 32 S03-09 rn  32  33  16.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 33 S03-10 rn  33  34  16.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 34 S03-11 rn  34  35  16.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 35 S03-12 rn  35  36   6.27 6.8 109.6 629528  62953 778.93  -0.01  99999  99999 0.02 e
 36 S03-13 rn  36  37   6.27 6.8 109.6 629528  62953 778.93  -0.01  99999  99999 0.02 e
 37 S03-14 rn  37  38   4.26 6.8 115.6 629528  62953 826.75  -0.01  99999  99999 0.02 e
 38 C03-01 rn  38  39   3.38 6.0 56.5 629528  62953  95.48   0.00  99999  99999 0.02 e
 39 C03-02 rn  39  40  11.95 6.0 56.5 629528  62953  47.54   0.00  34440  31764 0.02 e
 40 C03-03 rn  40  41  11.95 6.0 56.5 629528  62953  47.54   0.00  34440  31764 0.02 e
 41 C03-04 rn  41  42  11.95 6.0 56.5 629528  62953  47.54   0.00  34440  31764 0.02 e
 42 P03-01 rn  42  43   2.23 6.0 56.5 629528  62953  47.54   0.00  34440  31764 0.02 e
 43 P03-02 rn  43  44   2.23 6.0 56.5 629528  62953  47.54   0.00  34440  31764 0.02 e
 44 P03-03 rn  44  45   2.23 6.0 56.5 629528  62953  47.54   0.00  34440  31764 0.02 e
 45 P03-04 rn  45  46   2.23 6.0 56.5 629528  62953  47.54   0.00  34440  31764 0.02 e
 46 P03-05 rn  46  47   2.23 6.0 56.5 629528  62953  47.54   0.00  34440  31764 0.02 e
 47 S04-01 rn  38  48   4.26 6.8 115.6 629528  62953 826.75  -0.01  99999  99999 0.02 e
 48 S04-02 rn  48  49   3.77 6.8 109.6 629528  62953 778.93  -0.01  99999  99999 0.02 e
 49 S04-03 rn  49  50   3.77 6.8 109.6 629528  62953 778.93  -0.01  99999  99999 0.02 e
 50 S04-04 rn  50  51  11.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 51 S04-05 rn  51  52  11.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 52 S04-06 rn  52  53  11.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 53 S04-07 rn  53  54  11.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 54 S04-08 rn  54  55  11.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 55 S04-09 rn  55  56  11.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 56 S04-10 rn  56  57  11.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 57 S04-11 rn  57  58  11.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 58 S04-12 rn  58  59  11.80 6.8 103.5 629528  62953 731.10  -0.01  99999  99999 0.02 e
 59 C04-01 rs  59  60   1.00 6.0 56.5 629528  62953  95.48   0.00  99999  99999 0.02 e
 60 P04-01 rn  60  61   1.00 6.0 56.5 629528  62953  47.54   0.00  99999  99999 0.02 e
 61 S05-01 rn  59  62   2.00 6.8 103.5 629528  62953 826.75  -0.01  99999  99999 0.02 e
bandwidth of the problem = 11
Number of rows and columns in strage = 186 x 33

---------------------------------------------------------
---------------------------------------------------------
---------------------------------------------------------

Cumulative Results of analysis at end of stage   6

Plastic Action at:
                      Lat. Force        / Deflection
Element/ Stage/ Code/  *g (DL=    4.2)  /    (in)
P03X01     1      2      238.3100          6.8034
P03X02     1      2      238.3100          6.8034
P02X01     2      2      320.4090          9.2453
P02X02     3      2      337.1791          9.7459
P03X03     4      2      346.8074         10.0355
C02-02     5     rs      367.7457         10.6771
C03-02     6     rs      370.2617         10.8498

node# name   ---------- GLOBAL ---------
             Displ.x  Displ.y  Rotation
```

```
    1 S01.00  0.90415 -0.00139  0.00070
    2 S01.01  0.90415  0.00001  0.00070
    3 C01.01  0.45208  0.00000 -0.45208
    4 P01.01  0.00000  0.00000 -0.45208
    5 S02.01  0.90415  0.00870  0.00068
    6 S02.02  0.90413  0.01687  0.00061
    7 S02.03  0.90411  0.02398  0.00051
    8 S02.04  0.90408  0.02951  0.00036
    9 S02.05  0.90403  0.03294  0.00017
   10 S02.06  0.90398  0.03374 -0.00006
   11 S02.07  0.90393  0.03137 -0.00033
   12 S02.08  0.90386  0.02532 -0.00064
   13 S02.09  0.90378  0.01504 -0.00100
   14 S02.10  0.90375  0.01063 -0.00112
   15 S02.11  0.90373  0.00572 -0.00124
   16 S02.12  0.90370  0.00017 -0.00136
   17 C02.01  0.89564  0.00016 -0.00338
   18 C02.02  0.77721  0.00012 -0.01551
   19 C02.03  0.54070  0.00007 -0.02351
   20 C02.04  0.23139  0.00003 -0.02771
   21 P02.01  0.17421  0.00002 -0.02805
   22 P02.02  0.11644  0.00002 -0.02828
   23 P02.03  0.05831  0.00001 -0.02842
   24 P02.04  0.00000  0.00000 -0.02846
   25 S03.01  0.90372 -0.00526 -0.00119
   26 S03.02  0.90375 -0.01190 -0.00093
   27 S03.03  0.90377 -0.01699 -0.00070
   28 S03.04  0.90383 -0.02372 -0.00013
   29 S03.05  0.90388 -0.02222  0.00028
   30 S03.06  0.90391 -0.01511  0.00054
   31 S03.07  0.90392 -0.00498  0.00064
   32 S03.08  0.90392  0.00555  0.00059
   33 S03.09  0.90390  0.01387  0.00038
   34 S03.10  0.90386  0.01736  0.00001
   35 S03.11  0.90381  0.01343 -0.00051
   36 S03.12  0.90379  0.00957 -0.00073
   37 S03.13  0.90377  0.00427 -0.00097
   38 S03.14  0.90375 -0.00020 -0.00113
   39 C03.01  0.89675 -0.00018 -0.00299
   40 C03.02  0.79212 -0.00014 -0.01395
   41 C03.03  0.57699 -0.00009 -0.02148
   42 C03.04  0.29240 -0.00004 -0.02558
   43 P03.01  0.23492 -0.00004 -0.02596
   44 P03.02  0.17671 -0.00003 -0.02623
   45 P03.03  0.11803 -0.00002 -0.02639
   46 P03.04  0.05907 -0.00001 -0.02647
   47 P03.05  0.00000  0.00000 -0.02650
   48 S04.01  0.90378 -0.00479 -0.00102
   49 S04.02  0.90380 -0.00845 -0.00092
   50 S04.03  0.90382 -0.01176 -0.00083
   51 S04.04  0.90389 -0.01976 -0.00053
   52 S04.05  0.90394 -0.02448 -0.00027
   53 S04.06  0.90400 -0.02632 -0.00005
   54 S04.07  0.90404 -0.02571  0.00015
   55 S04.08  0.90407 -0.02303  0.00030
   56 S04.09  0.90410 -0.01872  0.00042
   57 S04.10  0.90412 -0.01317  0.00051
   58 S04.11  0.90413 -0.00679  0.00056
   59 S04.12  0.90413 -0.00001  0.00058
   60 C04.01  0.45207  0.00000 -0.45207
   61 P04.01  0.00000  0.00000 -0.45207
   62 S05.01  0.90413  0.00116  0.00058

element   node  -------- local -----------  ------ element ----------
#    name  fix    displ.x  displ.y  rotation  axial    shear     moment
  1 S01-01 rn   1  0.90415 -0.00139  0.00070       2.84       0.05       0.04
                2  0.90415  0.00001  0.00070      -2.84      -0.03      -0.08
  2 C01-01 rs   2 -0.00001  0.90415 -0.45208    -120.85       1.71      -7.90
```

		3	0.00000	0.45208	-0.45208	120.85	-1.71	-17.98
3	P01-01 rn	3	0.00000	0.45208	-0.45208	-120.85	-7.99	-11.26
		4	0.00000	0.00000	-0.45208	120.85	7.99	-1.32
4	S02-01 rn	2	0.90415	0.00001	0.00070	27.04	-120.88	0.10
		5	0.90415	0.00870	0.00068	-27.04	121.01	-1524.03
5	S02-02 rn	5	0.90415	0.00870	0.00068	73.86	-121.01	1524.04
		6	0.90413	0.01687	0.00061	-73.86	121.14	-3049.55
6	S02-03 rn	6	0.90413	0.01687	0.00061	120.42	-121.14	3049.55
		7	0.90411	0.02398	0.00051	-120.42	121.26	-4576.61
7	S02-04 rn	7	0.90411	0.02398	0.00051	166.47	-121.26	4576.55
		8	0.90408	0.02951	0.00036	-166.47	121.39	-6105.31
8	S02-05 rn	8	0.90408	0.02951	0.00036	212.95	-121.39	6105.28
		9	0.90403	0.03294	0.00017	-212.95	121.52	-7635.61
9	S02-06 rn	9	0.90403	0.03294	0.00017	259.32	-121.52	7635.55
		10	0.90398	0.03374	-0.00006	-259.32	121.65	-9167.53
10	S02-07 rn	10	0.90398	0.03374	-0.00006	306.21	-121.65	9167.56
		11	0.90393	0.03137	-0.00033	-306.21	121.78	-10701.15
11	S02-08 rn	11	0.90393	0.03137	-0.00033	352.99	-121.77	10701.21
		12	0.90386	0.02532	-0.00064	-352.99	121.90	-12236.28
12	S02-09 rn	12	0.90386	0.02532	-0.00064	399.28	-121.90	12236.33
		13	0.90378	0.01504	-0.00100	-399.28	122.02	-13773.04
13	S02-10 rn	13	0.90378	0.01504	-0.00100	432.17	-121.98	13772.78
		14	0.90375	0.01063	-0.00112	-432.17	122.03	-14281.39
14	S02-11 rn	14	0.90375	0.01063	-0.00112	446.68	-122.04	14281.40
		15	0.90373	0.00572	-0.00124	-446.68	122.08	-14790.22
15	S02-12 rn	15	0.90373	0.00572	-0.00124	463.89	-122.07	14790.26
		16	0.90370	0.00017	-0.00136	-463.89	122.11	**-15310.55**
16	C02-01 rn	16	-0.00017	0.90370	-0.00136	-130.40	802.48	37205.61
		17	-0.00016	0.89564	-0.00338	130.40	-802.48	-34494.98
17	C02-02 rs	17	-0.00016	0.89564	-0.00371	-130.34	801.37	34494.00
		18	-0.00012	0.77721	-0.01551	130.34	-801.37	-24933.64
18	C02-03 rn	18	-0.00012	0.77721	-0.01551	-130.33	801.39	24933.54
		19	-0.00007	0.54070	-0.02351	130.33	-801.39	-15372.98
19	C02-04 rn	19	-0.00007	0.54070	-0.02351	-130.33	801.39	15372.89
		20	-0.00003	0.23139	-0.02771	130.33	-801.39	-5812.36
20	P02-01 rn	20	-0.00003	0.23139	-0.02771	-130.33	802.05	5812.72
		21	-0.00002	0.17421	-0.02805	130.33	-802.05	-4167.29
21	P02-02 rn	21	-0.00002	0.17421	-0.02805	-130.33	762.34	4168.23
		22	-0.00002	0.11644	-0.02828	130.33	-762.34	-2605.56
22	P02-03 rn	22	-0.00002	0.11644	-0.02828	-130.33	673.95	2606.07
		23	-0.00001	0.05831	-0.02842	130.33	-673.95	-1223.58
23	P02-04 rn	23	-0.00001	0.05831	-0.02842	-130.33	596.89	1223.75
		24	0.00000	0.00000	-0.02846	130.33	-596.89	0.17
24	S03-01 rn	16	0.90370	0.00017	-0.00136	-324.17	-252.49	**-21895.81**
		25	0.90372	-0.00526	-0.00119	324.17	252.53	20820.30
25	S03-02 rn	25	0.90372	-0.00526	-0.00119	-303.88	-252.46	-20820.02
		26	0.90375	-0.01190	-0.00093	303.88	252.52	19236.85
26	S03-03 rn	26	0.90375	-0.01190	-0.00093	-280.84	-252.52	-19236.81
		27	0.90377	-0.01699	-0.00070	280.84	252.59	17653.28
27	S03-04 rn	27	0.90377	-0.01699	-0.00070	-238.12	-252.60	-17653.29
		28	0.90383	-0.02372	-0.00013	238.12	252.77	13408.15
28	S03-05 rn	28	0.90383	-0.02372	-0.00013	-175.94	-252.77	-13408.13
		29	0.90388	-0.02222	0.00028	175.94	252.94	9160.19
29	S03-06 rn	29	0.90388	-0.02222	0.00028	-113.56	-252.94	-9160.15
		30	0.90391	-0.01511	0.00054	113.56	253.11	4909.38
30	S03-07 rn	30	0.90391	-0.01511	0.00054	-51.34	-253.11	-4909.39
		31	0.90392	-0.00498	0.00064	51.34	253.28	655.75
31	S03-08 rn	31	0.90392	-0.00498	0.00064	10.76	-253.28	-655.75
		32	0.90392	0.00555	0.00059	-10.76	253.44	-3600.70
32	S03-09 rn	32	0.90392	0.00555	0.00059	72.98	-253.45	3600.70
		33	0.90390	0.01387	0.00038	-72.98	253.61	-7860.00
33	S03-10 rn	33	0.90390	0.01387	0.00038	135.11	-253.61	7860.00
		34	0.90386	0.01736	0.00001	-135.11	253.78	-12122.12
34	S03-11 rn	34	0.90386	0.01736	0.00001	197.11	-253.79	12122.11
		35	0.90381	0.01343	-0.00051	-197.11	253.95	-16387.14
35	S03-12 rn	35	0.90381	0.01343	-0.00051	239.82	-253.95	16387.14
		36	0.90379	0.00957	-0.00073	-239.82	254.02	-17979.58
36	S03-13 rn	36	0.90379	0.00957	-0.00073	262.44	-254.03	17979.48

			37	0.90377	0.00427	-0.00097	-262.44	254.09	-19572.36
37	S03-14	rn	37	0.90377	0.00427	-0.00097	282.47	-254.08	19572.30
			38	0.90375	-0.00020	-0.00113	-282.47	254.12	**-20654.71**
38	C03-01	rn	38	0.00020	0.90375	-0.00113	139.66	720.91	34201.37
			39	0.00018	0.89675	-0.00299	-139.66	-720.91	-31767.00
39	C03-02	rs	39	0.00018	0.89675	-0.00299	139.68	720.04	31764.00
			40	0.00014	0.79212	-0.01395	-139.68	-720.04	-23159.53
40	C03-03	rn	40	0.00014	0.79212	-0.01395	139.69	720.06	23159.42
			41	0.00009	0.57699	-0.02148	-139.69	-720.06	-14554.75
41	C03-04	rn	41	0.00009	0.57699	-0.02148	139.69	720.06	14554.75
			42	0.00004	0.29240	-0.02558	-139.69	-720.06	-5949.93
42	P03-01	rn	42	0.00004	0.29240	-0.02558	139.69	719.45	5949.87
			43	0.00004	0.23492	-0.02596	-139.69	-719.45	-4344.50
43	P03-02	rn	43	0.00004	0.23492	-0.02596	139.69	673.41	4345.39
			44	0.00003	0.17671	-0.02623	-139.69	-673.41	-2843.41
44	P03-03	rn	44	0.00003	0.17671	-0.02623	139.69	572.30	2843.25
			45	0.00002	0.11803	-0.02639	-139.69	-572.30	-1567.32
45	P03-04	rn	45	0.00002	0.11803	-0.02639	139.69	411.75	1567.28
			46	0.00001	0.05907	-0.02647	-139.69	-411.75	-649.24
46	P03-05	rn	46	0.00001	0.05907	-0.02647	139.69	291.26	649.38
			47	0.00000	0.00000	-0.02650	-139.69	-291.26	0.11
47	S04-01	rn	38	0.90375	-0.00020	-0.00113	-422.87	-114.45	**-13549.22**
			48	0.90378	-0.00479	-0.00102	422.87	114.50	13061.69
48	S04-02	rn	48	0.90378	-0.00479	-0.00102	-410.57	-114.36	-13061.47
			49	0.90380	-0.00845	-0.00092	410.57	114.40	12630.12
49	S04-03	rn	49	0.90380	-0.00845	-0.00092	-394.49	-114.36	-12629.92
			50	0.90382	-0.01176	-0.00083	394.49	114.40	12198.84
50	S04-04	rn	50	0.90382	-0.01176	-0.00083	-366.01	-114.35	-12198.75
			51	0.90389	-0.01976	-0.00053	366.01	114.47	10848.67
51	S04-05	rn	51	0.90389	-0.01976	-0.00053	-322.37	-114.46	-10848.68
			52	0.90394	-0.02448	-0.00027	322.37	114.58	9497.30
52	S04-06	rn	52	0.90394	-0.02448	-0.00027	-278.84	-114.58	-9497.36
			53	0.90400	-0.02632	-0.00005	278.84	114.70	8144.55
53	S04-07	rn	53	0.90400	-0.02632	-0.00005	-234.82	-114.69	-8144.57
			54	0.90404	-0.02571	0.00015	234.82	114.81	6790.48
54	S04-08	rn	54	0.90404	-0.02571	0.00015	-191.10	-114.80	-6790.45
			55	0.90407	-0.02303	0.00030	191.10	114.92	5435.10
55	S04-09	rn	55	0.90407	-0.02303	0.00030	-147.72	-114.91	-5435.07
			56	0.90410	-0.01872	0.00042	147.72	115.03	4078.38
56	S04-10	rn	56	0.90410	-0.01872	0.00042	-103.91	-115.03	-4078.36
			57	0.90412	-0.01317	0.00051	103.91	115.15	2720.33
57	S04-11	rn	57	0.90412	-0.01317	0.00051	-60.25	-115.15	-2720.31
			58	0.90413	-0.00679	0.00056	60.25	115.26	1360.86
58	S04-12	rn	58	0.90413	-0.00679	0.00056	-16.89	-115.26	-1360.86
			59	0.90413	-0.00001	0.00058	16.89	115.38	0.06
59	C04-01	rs	59	0.00001	0.90413	-0.45207	115.39	9.95	-6.26
			60	0.00000	0.45207	-0.45207	-115.39	-9.95	19.22
60	P04-01	rn	60	0.00000	0.45207	-0.45207	115.39	-7.53	-2.69
			61	0.00000	0.00000	-0.45207	-115.39	7.53	0.08
61	S05-01	rn	59	0.90413	-0.00001	0.00058	-2.50	0.02	-0.04
			62	0.90413	0.00116	0.00058	2.50	0.00	0.02

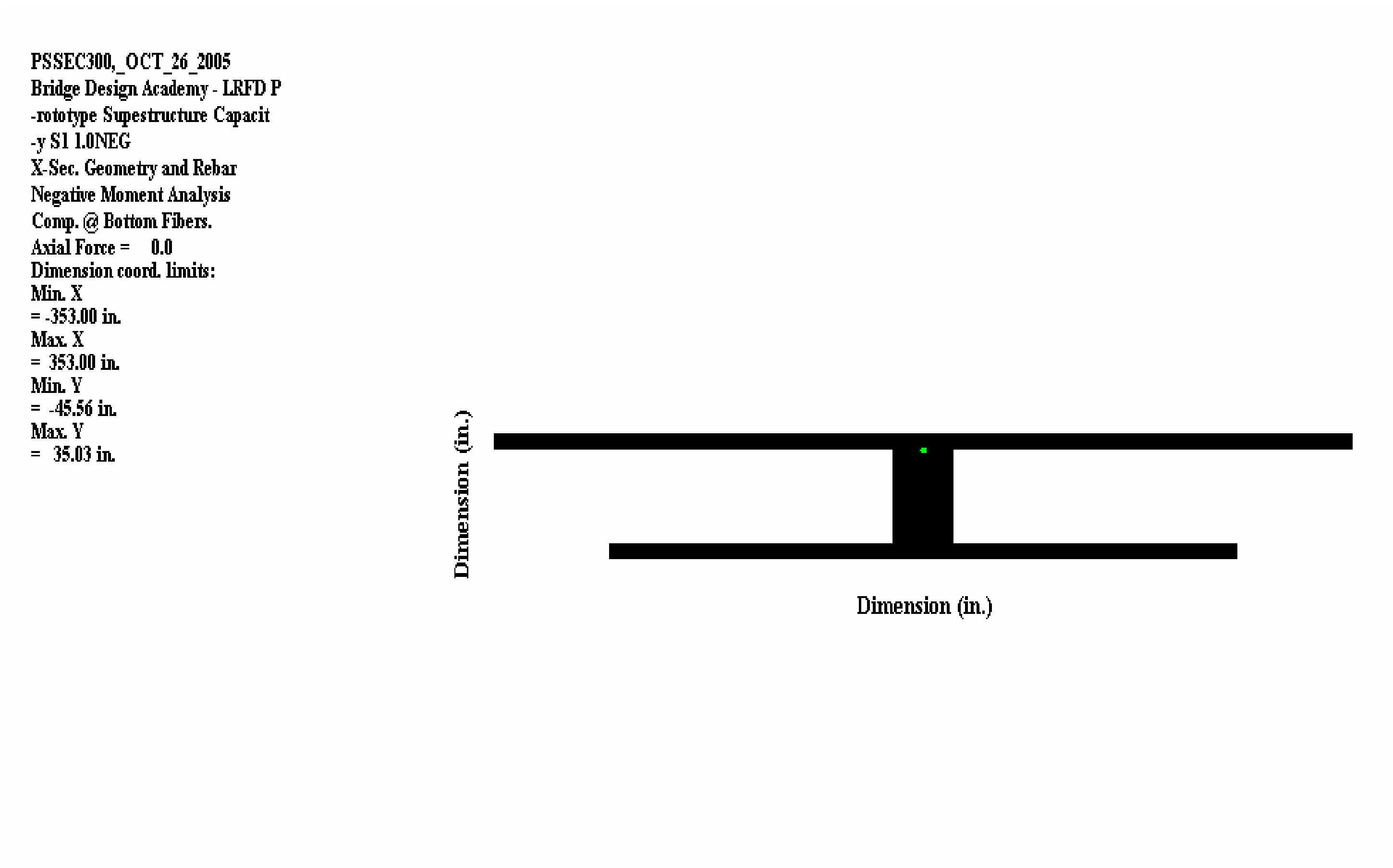
APPENDIX – T

(*PSSECx* Input File)

```
PSSEC300,_OCT_26_2005
Bridge Design Academy - LRFD Prototype Supestructure Capacity S1 1.0NEG
Number of different types of concrete
1
For each concrete type input:
Type number; Model code= 0 simple(unconfined/confined), 1 Mander's (unconfined)
strength f'c0 (ksi), strain ec0, strength fcu (ksi), ult. strain ecu, conc. density
1    1
5.200   .002       0.5    0.0025      150
Number of different types of P/S steel
1
For each type, 1st line for tensile parameters,2nd line for cmpressive parameters
type#;E;fy;strain hard. factor;fu;ult. strain;PS-code: 0 tendons, 1 otherwise
      E;fy;strain hard. factor;fu;ult. strain
1   28500    245    1    270   0.040   1
     0        0     0     0      0
Number of different types of mild steel
1
For each steel type input:
Type number;Model code= 0 simple, 1 complex
E(ksi);fy(ksi);strain hard. factor;fu(ksi);ultimate strain
1    1
29000    66   6.59   92   0.120
Number of Conc. Subsections
1
For each Subsec.:Subsection #,Section shape type, Concrete type, No. of fibers
Subsec. Dim.(in):(See Manual for input parameters.)
Subsec. Dim.(in):(See Manual for input parameters.)
Global coord. of the center of Subsec.: Xg, Yg
1  I-shaped,    1   200
706.0  48.0  517.0
81.0  9.125  8.25
0   -5.26
Number of P/S steel groups
1
For each group:group#;P/S type;x-coord.(in);y-coord.(in);area(in^2);P/S force
1    1     0    25.4412     38.28   6157
Number of mild steel rebar cages (rebar distributed around the perimeter)
0
cage#;steel type;cage shape;#of bars;x(in) of 1st bar(y=0);area(in^2)of bar
Number of mild steel groups (no logical pattern for distribution)
2   n
group#;steel type;x-coord.(in); y-coord.(in); area(in^2)
1   1    0   31.80  47.40
2   1    0  -42.13  34.76
Non P/S Axial load on mid-depth of section (Kips)(+ sign=compression)
0
Numerical Computation Factor (1 to 10)
5
Computer Graphics Card identifier: 0 none; 2 CGA; 3 Hercules; 9 EGA; 12 VGA
12
Output control: 0 short; 1 long output
1
X-Sec. plot control (0=no plot, 1=each stage, 2=every iteration of each step)
0
Analysis Control:  p - Positive moment, n - Negative moment
n
```

APPENDIX – U

(*PSSECx* Model for Superstructure)



APPENDIX – V

(Partial Output from PSSECx Run)

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                                         09-27-2006
********   SECx   ********

        DUCTILITY and STRENGTH of
Rectangular, T-, I-, Hammer, Octagonal, Circular, Ring,
     and Hollowed shaped Prestressed and Reinforced
      Concrete Sections using fiber models
            Ver. 3.00, OCT-26-2005
```


JOB TITLE: Bridge Design Academy - LRFD Prototype Supestructure Capacity S1 1.0NEG

Concrete Data, Complex Model, Mander's unconfined

Concrete Type		=	**1**
Compressive Strength (max.)	**(ksi)**	=	**5.200**
Strain at max. Strength		=	**.00200**
Strength at Ultimate Strain	**(ksi)**	=	**0.000**
Ultimate strain		=	**.00500**
Unit Weight (pcf)		=	**150.00**

Prestressing Steel Data

Material No.	Yield Strain	Hardening Strain	Ultimate Strain	Yield Stress ksi	Ultimate Stress ksi	Modulus of Elasticity ksi	
1	**0.00860**	**0.00860**	**0.04000**	**245.00**	**270.00**	**28500.00**	Tensile prop.
comp.	0.00000	0.00000	0.00000	0.00	0.00	0.00	Compressive prop.

Mild Steel Reinforcing Data

Material No.	Yield Strain	Hardening Strain	Ultimate Strain	Yield Stress ksi	Ultimate Stress ksi
1	**0.00228**	**0.01500**	**0.12000**	**66.00**	**92.00**

Rectangular, T-, or I-shaped section information

Depth of Section	(in.)	=	81.00
Top Flange width	(in.)	=	706.00
Top Flange thickness	(in.)	=	9.13
Bot Flange width	(in.)	=	517.00
Bot Flange thickness	(in.)	=	8.25
Web thickness	(in.)	=	48.00

Concrete fiber information

Fiber #	Material #	x (in)	y (in)	area (in^2)
1	1.0	0.00	-45.56	203.11
2	1.0	0.00	-45.17	203.11
3	1.0	0.00	-44.78	203.11
4	1.0	0.00	-44.38	203.11
5	1.0	0.00	-43.99	203.11
6	1.0	0.00	-43.60	203.11
7	1.0	0.00	-43.21	203.11
8	1.0	0.00	-42.81	203.11
9	1.0	0.00	-42.42	203.11
10	1.0	0.00	-42.03	203.11

```
   11    1.0      0.00    -41.63    203.11
   12    1.0      0.00    -41.24    203.11
   13    1.0      0.00    -40.85    203.11
   14    1.0      0.00    -40.46    203.11
   15    1.0      0.00    -40.06    203.11

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  188    1.0      0.00     30.06    292.83
  189    1.0      0.00     30.47    292.83
  190    1.0      0.00     30.88    292.83
  191    1.0      0.00     31.30    292.83
  192    1.0      0.00     31.71    292.83
  193    1.0      0.00     32.13    292.83
  194    1.0      0.00     32.54    292.83
  195    1.0      0.00     32.96    292.83
  196    1.0      0.00     33.37    292.83
  197    1.0      0.00     33.79    292.83
  198    1.0      0.00     34.20    292.83
  199    1.0      0.00     34.62    292.83
  200    1.0      0.00     35.03    292.83


       Prestressing Steel Fiber Data

 Fiber  Material      x          y         area    P/S force
   No.    No.        (in)       (in)       (in^2)    Kips

     1     1       0.00      25.44      38.28    6157.00

    Total P/S force on the section =    6157.0 kips
    Total moment due to P/S about point (0, 0)  =   13053.5 ft-kip

       Mild Steel Fiber Data

 Fiber  Material      x          y         area
   No.    No.        (in)       (in)       (in^2)

     1     1       0.00      31.80      47.40
     2     1       0.00     -42.13      34.76

Axial load at mid-depth of section (kip)(positive means compression) =     0.0

***************************************************
* Analysis Results ---   Negative Moment Capacity *
***************************************************

 Initial state due to P/S without non-P/S axial force:
 N.A. Loc.     Curvature     Conc. Strain @ max. compressed fiber
    -41.50    0.0000023         0.00017950

 Undeformed P/S element position w.r.t. reference plane
 P/S Fiber    Loc.(y)      Undef. pos.      Conc. Strain @ same loc.
      1      25.44     -0.0058006         -0.0001570


Force Equilibrium Condition of the x-section:

     Max.             Max.
     Conc.    Neutral Steel           Steel
     Strain    Axis   Strain Conc.    force       P/S     Net  Curvature Moment
step epscmax   in.    Tens.  Comp.  Comp. Tens.   force   force  in/in   (K-ft)
 0 -.00001  -41.50 -.00000   5923.   236.    -1. -6157.  -0.8 0.000002      -4.
 1 -.00001  -42.26 0.00000   5923.   235.     0. -6158.  -0.4 0.000002    -147.
```

```
 2 -.00001  -43.05 0.00000   5923.   236.     0. -6158.  -0.5 0.000002    -307.
 3 -.00000  -43.86 0.00000   5923.   236.     0. -6159.   0.3 0.000002    -486.
 4 -.00000  -44.70 0.00000   5924.   237.     0. -6160.   0.2 0.000002    -683.
 5 0.00000  -45.56 0.00000   5925.   237.     0. -6161.  -0.7 0.000002    -899.
 6 0.00010 9055.25 0.00000   5983.   237.     0. -6220.  -0.4 -.000000  -13142.
 7 0.00011  362.50 0.00000   5990.   237.     0. -6227.  -0.3 -.000000  -14634.
 8 0.00013  174.76 0.00000   5997.   237.     0. -6235.   0.8 -.000001  -16309.
 9 0.00014  110.77 0.00000   6006.   237.     0. -6244.   0.9 -.000001  -18186.
10 0.00016   78.67 0.00000   6017.   237.     0. -6254.  -0.1 -.000001  -20287.
11 0.00018   59.45 0.00000   6028.   238.     0. -6265.  -0.7 -.000002  -22643.
12 0.00020   46.72 0.00000   6041.   238.     0. -6278.  -0.3 -.000002  -25286.
13 0.00022   37.74 0.00000   6055.   238.     0. -6292.  -1.0 -.000003  -28243.
14 0.00025   29.97 -.00001   6079.   242.    -8. -6312.  -0.1 -.000003  -31443.
15 0.00028   14.77 -.00008   6224.   268.  -109. -6383.   0.4 -.000005  -33995.
16 0.00032    2.23 -.00020   6470.   296.  -269. -6496.  -0.7 -.000007  -36442.
17 0.00035   -6.69 -.00035   6806.   326.  -483. -6648.  -0.9 -.000009  -39119.
18 0.00040  -12.96 -.00055   7231.   359.  -751. -6840.   0.5 -.000012  -42153.
19 0.00045  -17.40 -.00078   7745.   395. -1072. -7069.   0.4 -.000016  -45615.
20 0.00050  -20.61 -.00105   8346.   435. -1445. -7336.   0.3 -.000020  -49549.
21 0.00056  -22.97 -.00136   9034.   480. -1872. -7642.  -0.4 -.000025  -53987.
22 0.00063  -24.75 -.00171   9811.   530. -2354. -7987.  -0.2 -.000030  -58960.
23 0.00071  -26.11 -.00210  10680.   587. -2893. -8372.  -1.0 -.000036  -64494.
24 0.00079  -28.00 -.00270  11447.   643. -3128. -8962.   0.2 -.000045  -69356.
25 0.00089  -30.83 -.00378  11856.   688. -3128. -9415.  -0.2 -.000060  -72107.
26 0.00100  -33.58 -.00545  11914.   718. -3128. -9504.  -0.3 -.000083  -72855.
27 0.00112  -35.35 -.00737  11979.   750. -3128. -9600.  -0.3 -.000110  -73616.
28 0.00126  -36.55 -.00953  12045.   785. -3128. -9702.   0.4 -.000139  -74391.
29 0.00141  -37.38 -.01192  12108.   825. -3128. -9805.   0.8 -.000172  -75173.
30 0.00158  -38.00 -.01460  12168.   871. -3128. -9910.  -0.3 -.000209  -75951.
31 0.00178  -38.43 -.01749  12351.   928. -3268.-10011.   0.0 -.000249  -77499.
32 0.00199  -38.78 -.02072  12537.   991. -3421.-10107.  -0.2 -.000294  -79072.
33 0.00223  -39.06 -.02433  12691.  1064. -3561.-10195.   0.4 -.000343  -80483.
34 0.00251  -39.26 -.02826  12802.  1151. -3688.-10266.   0.6 -.000398  -81685.
35 0.00281  -39.42 -.03259  12866.  1251. -3802.-10316.   0.6 -.000458  -82652.
36 0.00316  -39.52 -.03726  12865.  1373. -3902.-10336.  -0.3 -.000522  -83335.
37 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.
38 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.
39 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.
40 0.00000    0.00 0.00000      0.     0.     0.     0.   0.0 0.000000       0.
```

```
Prestress Tendon Strain on the x-section:

     Max.
     Conc.    Neutral P/S Steel  Strain
     Strain    Axis
step epscmax   in.   No. Strain  No. Strain  No. Strain  No. Strain  No. Strain
 0 -.00001  -41.50  1 -.005644
 1 -.00001  -42.26  1 -.005644
 2 -.00001  -43.05  1 -.005645
 3 -.00000  -43.86  1 -.005646
 4 -.00000  -44.70  1 -.005647
 5 0.00000  -45.56  1 -.005648
 6 0.00010 9055.25  1 -.005701
 7 0.00011  362.50  1 -.005708
 8 0.00013  174.76  1 -.005715
 9 0.00014  110.77  1 -.005723
10 0.00016   78.67  1 -.005733
11 0.00018   59.45  1 -.005743
12 0.00020   46.72  1 -.005755
13 0.00022   37.74  1 -.005768
14 0.00025   29.97  1 -.005785
15 0.00028   14.77  1 -.005850
16 0.00032    2.23  1 -.005954
17 0.00035   -6.69  1 -.006094
18 0.00040  -12.96  1 -.006269
19 0.00045  -17.40  1 -.006479
20 0.00050  -20.61  1 -.006724
```

APPENDIX – V

(Partial Output from *PSSECx* Run) – Continues

```
21 0.00056  -22.97  1 -.007004
22 0.00063  -24.75  1 -.007321
23 0.00071  -26.11  1 -.007674
24 0.00079  -28.00  1 -.008215
25 0.00089  -30.83  1 -.009201
26 0.00100  -33.58  1 -.010721
27 0.00112  -35.35  1 -.012473
28 0.00126  -36.55  1 -.014443
29 0.00141  -37.38  1 -.016626
30 0.00158  -38.00  1 -.019070
31 0.00178  -38.43  1 -.021706
32 0.00199  -38.78  1 -.024657
33 0.00223  -39.06  1 -.027944
34 0.00251  -39.26  1 -.031529
35 0.00281  -39.42  1 -.035483
36 0.00316  -39.52  1 -.039738
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
```

At step 36, prestress tendon fails at ε = 0.04 prior to concrete or mild steel

```
Recommended value of 'effective moment of inertia' based on
     initial slope of moment-curvature diagram (ft^4) = 213.8433

Yield pt. is defined as the First mild steel yields.
The first mild steel yields between the following Steps:    23  and     24
The computation of mild steel yield point IS within 2% tolerance.
The first P/S steel yields between the following Steps:    24  and     25
The computation of P/S steel yield point IS NOT within 2% tolerance.

              Curvature(rad/in)      Moments (ft-K)
Yield              0.000039            66935
Nominal          See force equilibrium table at concrete strain of .003
Ultimate           0.000000                0


end
```

APPENDIX – W

(Partial xSECTION output Including Transverse Overturning Effects – Compression Column)

```
09/18/2006, 12:42
************************************************************
*                                                          *
*                       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 BENT 2 COMP - LRFD



Concrete Type Information:

     ----------strains--------  --------strength--------
Type   e0    e2    ecc     eu     f0    f2    fcc    fu    E    W
 1 0.0020 0.0040 0.0055 0.0185  5.20  6.86  7.02  5.56  4280   148
 2 0.0020 0.0040 0.0020 0.0050  5.20  3.58  5.20  2.60  4280   148


Steel Type Information:

     -----strains------  --strength-
Type  ey     eh     eu    fy    fu    E
 1 0.0023 0.0150 0.1200 66.00 92.00 29000
 2 0.0023 0.0075 0.0600 66.00 92.00 29000

-----------------------------------
-----------------------------------
-----------------------------------

Force Equilibrium Condition of the x-section:

     Max.             Max.
     Conc.    Neutral Steel           Steel
     Strain    Axis   Strain Conc.    force       P/S     Net  Curvature Moment
step epscmax   in.    Tens.  Comp.  Comp. Tens.   force   force  rad/in  (K-ft)
 0   0.00000    0.00  0.0000     0     0     0     0      0.00 0.000000       0
```

APPENDIX – W

(Partial xSECTION output Including Transverse Overturning Effects – Compression Column Continues)

1	0.00037	-17.80	-0.0001	2261	242	-34	0	-0.80	0.000007	3225
2	0.00041	-14.06	-0.0001	2277	252	-57	0	2.24	0.000008	3586
3	0.00045	-10.70	-0.0002	2296	265	-89	0	1.34	0.000010	3950
4	0.00050	-7.72	-0.0003	2318	277	-127	0	-2.40	0.000011	4323
5	0.00055	-5.12	-0.0004	2355	294	-177	0	1.72	0.000013	4708
6	0.00061	-2.82	-0.0005	2394	311	-235	0	-0.26	0.000016	5116
7	0.00068	-0.72	-0.0006	2446	328	-305	0	-1.53	0.000018	5547
8	0.00075	1.07	-0.0007	2508	349	-388	0	-1.24	0.000021	6011
9	0.00083	2.60	-0.0008	2579	374	-483	0	-0.39	0.000025	6516
10	0.00091	4.05	-0.0010	2665	400	-596	0	-1.67	0.000029	7060
11	0.00101	5.26	-0.0012	2764	428	-721	0	0.61	0.000033	7653
12	0.00112	6.31	-0.0014	2874	459	-862	0	1.42	0.000038	8298
13	0.00123	7.21	-0.0017	2992	493	-1017	0	-1.48	0.000043	8995
14	0.00136	7.97	-0.0019	3129	533	-1192	0	0.77	0.000049	9753
15	0.00151	8.66	-0.0022	3280	580	-1390	0	-0.35	0.000055	10573
16	0.00167	9.42	-0.0026	3410	628	-1567	0	1.52	0.000063	11275
17	0.00184	10.30	-0.0030	3502	677	-1710	0	-1.54	0.000072	11805
18	0.00204	11.18	-0.0035	3570	728	-1826	0	1.68	0.000082	12227
19	0.00225	12.08	-0.0041	3619	780	-1930	0	-1.33	0.000094	12579
20	0.00249	12.88	-0.0048	3663	836	-2028	0	0.56	0.000108	12901
21	0.00275	13.69	-0.0056	3678	892	-2100	0	-0.08	0.000123	13145
22	0.00304	14.40	-0.0065	3710	932	-2171	0	0.72	0.000141	13346
23	0.00336	14.97	-0.0075	3745	960	-2235	0	-0.52	0.000160	13501
24	0.00372	15.49	-0.0086	3781	984	-2295	0	0.15	0.000181	13649
25	0.00411	15.90	-0.0097	3825	1012	-2368	0	-0.80	0.000205	13832
26	0.00454	16.16	-0.0110	3873	1041	-2444	0	-0.61	0.000229	13998
27	0.00502	16.23	-0.0122	3925	1055	-2512	0	-1.85	0.000254	14092
28	0.00555	16.28	-0.0135	3966	1071	-2567	0	-0.88	0.000282	14176
29	0.00614	16.44	-0.0151	4005	1085	-2619	0	0.92	0.000314	14312
30	0.00678	16.58	-0.0169	4048	1101	-2680	0	-0.98	0.000350	14472
31	0.00750	16.69	-0.0188	4095	1118	-2741	0	2.08	0.000389	14644
32	0.00829	16.80	-0.0210	4137	1138	-2804	0	0.98	0.000432	14810
33	0.00917	16.89	-0.0233	4180	1160	**-2872**	0	-1.99	0.000480	**14978**
34	0.01013	16.93	-0.0259	4222	1188	-2939	0	1.52	0.000532	15148
35	0.01120	16.94	-0.0286	4274	1198	-3003	0	-0.77	0.000589	15296
36	0.01239	16.93	-0.0316	4325	1211	-3065	0	1.11	0.000650	15442
37	0.01369	16.91	-0.0349	4370	1224	-3124	0	-0.15	0.000718	15567
38	0.01514	16.88	-0.0385	4414	1237	-3182	0	-0.48	0.000792	15678
39	0.01673	16.83	-0.0424	4453	1257	-3240	0	0.27	0.000874	15783
40	0.01850	16.77	-0.0467	4486	1279	-3294	0	0.00	0.000963	15865

```
First Yield of Rebar Information (not Idealized):

     Rebar Number 20
     Coordinates X and Y (global in.)  -3.85, -31.70
     Yield strain =  0.00228
     Curvature (rad/in)=  0.000056
     Moment (ft-k) =   10722


Cross Section Information:

     Axial Load on Section (kips) =  2470
     Percentage of Main steel in Cross Section =   1.44
     Concrete modulus used in Idealization (ksi) =  4280
     Cracked Moment of Inertia (ft^4) =    25.728


Idealization of Moment-Curvature Curve by Various Methods:


                  Points on Curve        Idealized Values
                  ===============   ============================
Method     Conc.                    Yield         symbol  Plastic
 ID      | Strain  Curv.   Moment | Curv.  Moment   for   Curv.
         | in/in   rad/in  (K-ft) | rad/in (K-ft)  moment rad/in
Strain @ 0.003   0.000138   13318 0.000070   13318   Mn  0.000893
```

APPENDIX – W

(Partial xSECTION output Including Transverse Overturning Effects – Compression Column Continues)

```
Strain @ 0.004   0.000198   13782 0.000072   13782   Mn  0.000890
Strain @ 0.005   0.000253   14088 0.000074   14088   Mn  0.000889
CALTRANS 0.00879 0.000460   14906 0.000078   14906   Mp  0.000885
UCSD@5phy0.00555 0.000282   14175 0.000074   14175   Mn  0.000888
```